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SEISMIC VULNERABILITY ASSESSMENT OF CLUSTERED BUILDINGS IN THE HISTORICAL CENTER OF TIMISOARA: FRAGILITY CURVES FOR OUT-OF-PLANE LOCAL MECHANISMS OF COLLAPSE

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SUMMARY

The evaluation of the seismic vulnerability of historical buildings is important in order to develop prevention strategies and to protect the existing cultural heritage. The vulnerability analysis gives a preliminary assessment of the possible damages that a building or an aggregate can develop during a seismic event.

The adopted methodology provides an immediate tool to evaluate the seismic vulnerability of an entire city centre, starting from easily detectable geometrical features.

In this thesis the vulnerability analysis of the historical city center of Timisoara is discussed. The city is situated in the Banat region in western part of Romania, which is considered as the second most important seismic zone of Romania, being subjected to shallow earthquakes of crustal type.

The buildings of the historical centre are organized in rectangular blocks composed by clustered buildings. 36 blocks of the city center and 4 blocks of the district of Iosefin are analyzed, for a total of 245 analyzed structural units.

The proposed methodology begins with a preliminary knowledge phase, in which an urban and historical research is performed. This analysis aims to define architectural and urban characteristics, urban evolution and recurrent constructive techniques, which are indeed essential to address the analysis and to choose the aspects to investigate.

The second phase consisted in the on-site activity, in which important data about the building geometry, organization and structure are collected. The data are organized filling forms which are suitable for the rapid survey of masonry buildings. The collected data are analyzed in order to identify the most common geometrical and structural characteristics.

The study continues with the typological analysis of the on-site data: for each building the most important characteristics have been analyzed and 8 macro-typologies have been defined. Each macro-typology has been subdivided in typologies according to the stories number and so defining a total of 33 typologies. Likewise each typology is divided in micro-typologies, in accordance with the wall thickness of the ground floor. Thanks to a map of the ground floor internal organization of the city center buildings, dated back approximately to the

1980, the analysis of the plan modules is possible. The combination of the on-site data and the plan modules analysis allow a complete knowledge of the main building characteristics that are relevant for the seismic behavior.

A seismic vulnerability assessment is performed with the software Vulnus, which evaluates the vulnerability of out of plane and in plane local mechanisms of collapse. The software is used in two different applications: the first one regards the study of three blocks of the historical center, with the purpose of defining the vulnerability level of the three aggregate building; the second application regards the study of four singular structural units, to compare the results obtained for two different level of information, which are the real case, obtained by detailed plans and sections, and the survey case, obtained using the city plan of 1980 and information collected on-site.

The final aim of this thesis is to define, for each typology, the vulnerability assessment for the out of plane mechanisms of simple overturning and vertical bending and consequently to provide a global vulnerability assessment of the entire historical center, with the possibility to extend the results to similar buildings and urban centers. Therefore the methodology goes through the analysis and the verification of these local mechanisms, up to define the capacity curve of each typology. The analysis is made considering a set of varying parameters which takes into account the uncertainty of information caused by a rapid and external survey. The validity of the entire process is checked comparing the results of four singular structural units, analyzed using three different levels of information: the real case, the rapid survey case and the typological case.

Finally for each typology the fragility curves are defined, reporting the probability of exceeding defined levels of damage for specific peak ground accelerations.

The typological analysis defines a preliminary vulnerability assessment starting from a rapid survey on large scale. The results can be extended to constructions with the same structural and geometrical characteristics of the analyzed ones, widen the assessment to an urban scale.

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1 TIMISOARA

1.1 GEOGRAPHICAL LOCATION

Timisoara (45°45′35″N, 21°13′48″E) is the capital city of Timis County and is considered the informal capital city of the historical region of Banat, in western Romania. It is considered the main social, economic and cultural center in the western part of Romania; it is located at a distance of 550 km from Bucharest, the capital city of Romania, and respectively 300 km and 170 km from Budapest, the capital city of Hungary, and from Belgrade, the capital city of Serbia-Montenegro. Timisoara is one of the largest Romanian cities with an area of 130.5 km^2 and a population of 319279 inhabitants (2446.58/km^2)[1]. Population is divided into ethnic groups of: 86.79% Romanians, while 5.12% were Hungarians, 1.37% Germans, 1.3% Serbs, 0.69% ethnic Romani, 0.18% Ukrainians, 0.17% Slovaks, 0.11% Jews and 0.76% others.[2]

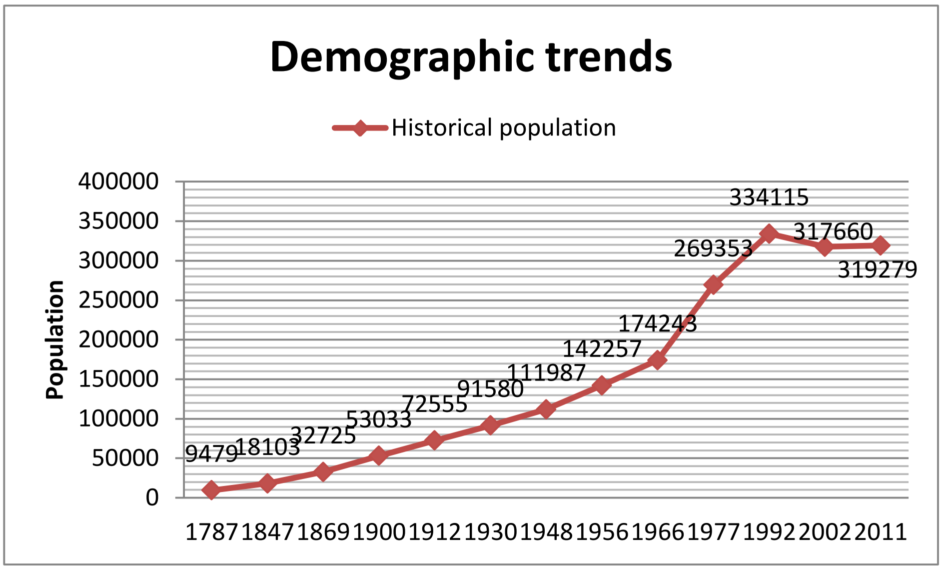

Fig. 1.1.1: Demographic trends graphic from 1787 to 2011

REFERENCE: Comunicat de presă - privind rezultatele provizorii ale Recensământului Populaţiei şi Locuinţelor – 2011 (Comisia judeţeană pentru recensământul populaţiei şi al locuinţelor, judeţul timiş, 2012)

[1] *Comunicat de presă - privind rezultatele provizorii ale Recensământului Populaţiei şi Locuinţelor – 2011*(Comisia judeţeană pentru recensământul populaţiei şi al locuinţelor, judeţul timiş, 2012)

[2] *Structura Etno-demografică a României* (Centrul de resurse pentru diversitate etnoculturală, 2002)

The north-east part of the city is the highest one, with an altitude of 95 m, and the west part is the lower, with 84 m of altitude[3].

Timisoara is situated on the southeast edge of the Banat plain, part of the Pannonian Plain near the Timis and Bega rivers, where the swamps could be crossed. With time the rivers of the area were drained, dammed and diverted. Due to these hydrographical projects of the 18th century, the city no longer lies on the Timis River, but just on the Bega canal, started in 1728, and all the surrounding marshes were drained of. However, the land across the city lies above a water table at a depth of only 0.5 to 5 m, a factor which does not allow the construction of tall buildings. The rich black soil and relatively high water table make this a fertile agricultural region[4].

Fig. 1.1.2: Geographical location of Timisoara

REFERENCE: (Google Earth 2014)

[3] *Date geografic-Reliefful* (Primaria municipiului Timişoara)

[4] *Premiere ale oraşului Timişoara* (Timişoara-info.ro, 2009)

Fig. 1.1.3: Region of Banat

REFERENCE: Banat (Ropenet.ro, 2008)

Fig. 1.1.4: Administrative division of Romania

REFERENCE:, History of Romania (Pop and Bolovan, 2006, p.841)

1.2 GEOMORPHOLOGY

The geological structure of Romania is determined by the position of the country between the structural zones of the Pannonian Depression, the Moldavian Platform, Scythian Platform and Moesian Platform. It is articulated around the Carpathian Mountains, formed during the alpine orogeny. The county of Vrancea is the most active seismic area of the country because is located in the connection of these structures[5].

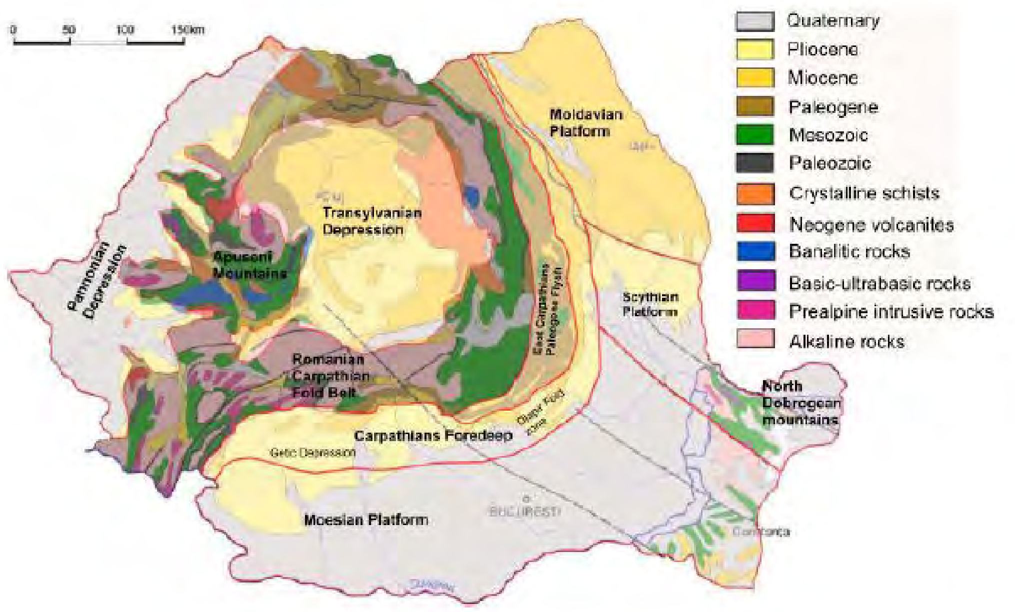

Fig. 1.2.1: Geological map of Romania and division of the major structural units

REFERENCE: Geology of Romania: potential co2 storage sites characterization. (Decarboni.se)

Alps, Carpathians and Dinarides are part of the system of Circum-Mediterranean orogeny and form a continuous and highly curved orogenic belt, that encircles the Pannonian Basin. The Alpine-Carpathian-Pannonian (ALCAPA) Mega-Unit slides to west against the Tisza-Dacia Mega-Unit, along the Mid-Hungarian Fault zone. The Tisza-Dacia Mega-Unit rotates anticlockwise against the Dinarides and the European Plate alongside respectively the Split-Karlovac Fault and the Timok

[5] *Geology of Romania* (Burchfiel et al., 2014)

and Cerna Jiu Faults. This rotational movement caused modifications of the Alpine-Carpathian-Dinaridic orogenic system.[6]

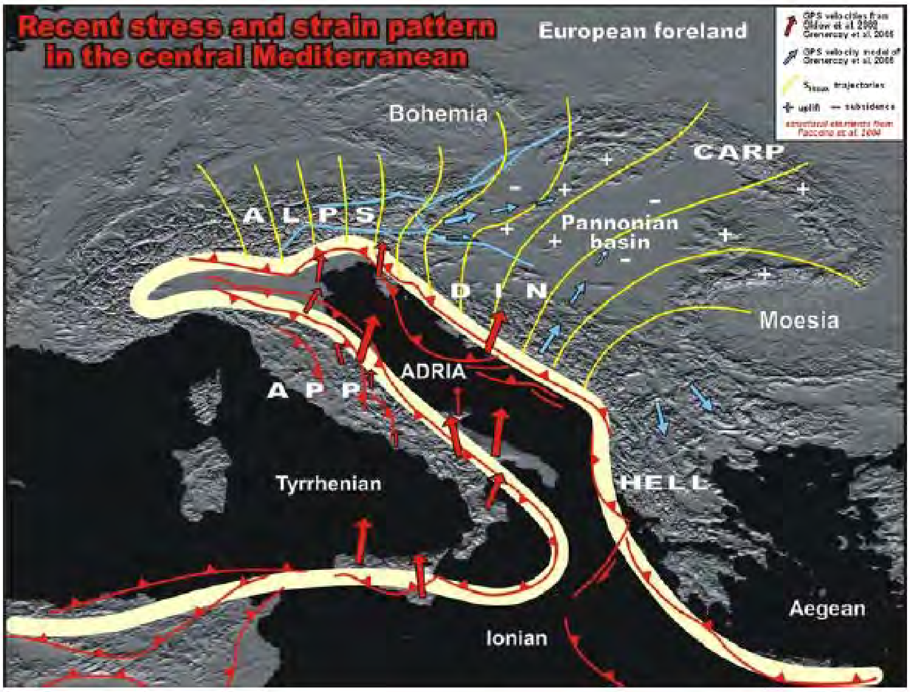

Fig. 1.2.2: Within the orogen: the Alps/Carpatian- Pannonian Basin System. Recent stress and strain pattern in the central Mediterranean.

REFERENCE: Intensity seismic hazard map of Romania by probabilistic and (neo)deterministic approaches, linear and nonlinear analyses
(Mărmureanu , Cioflan and Mărmureanu, 2011, p. 227)

[6] *A map-view restoration of the Alpine-Carpathian-Dinaridic system for the Early Miocene.* (Ustaszewski et al.,2008, p.1)

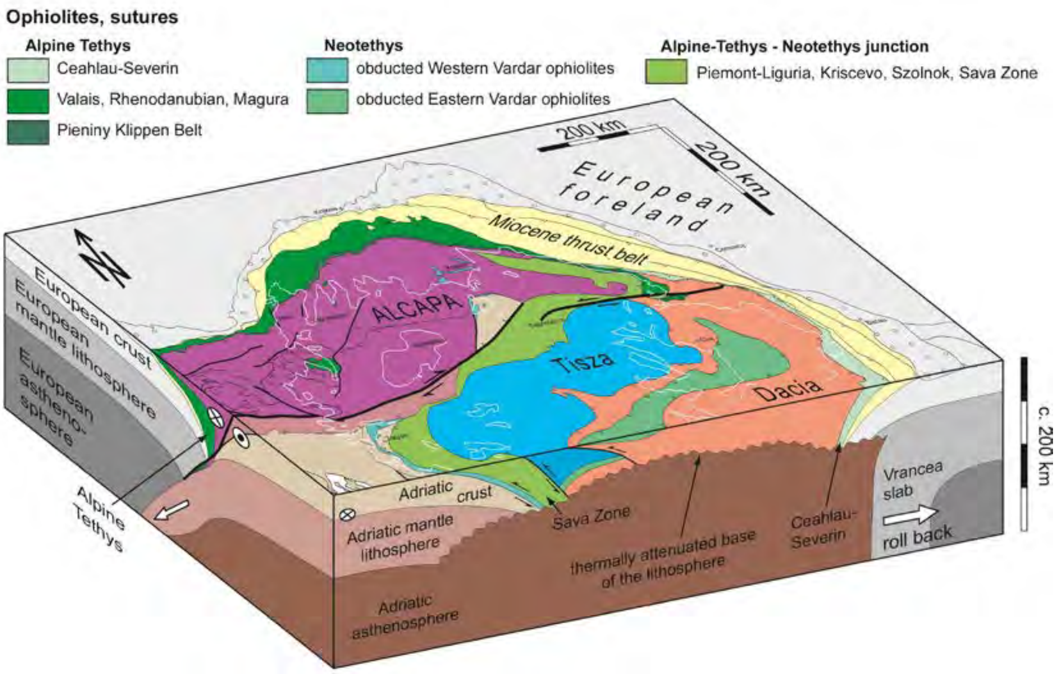

Fig. 1.2.3: Interpretative block diagram showing present-day lithospheric structures in the Eastern Alps, Carpathians and northern Dinarides.

REFERENCE: A map-view restoration of the Alpine-Carpathian-Dinaridic system for the Early Miocene. (Ustaszewski et al.,2008, p.18)

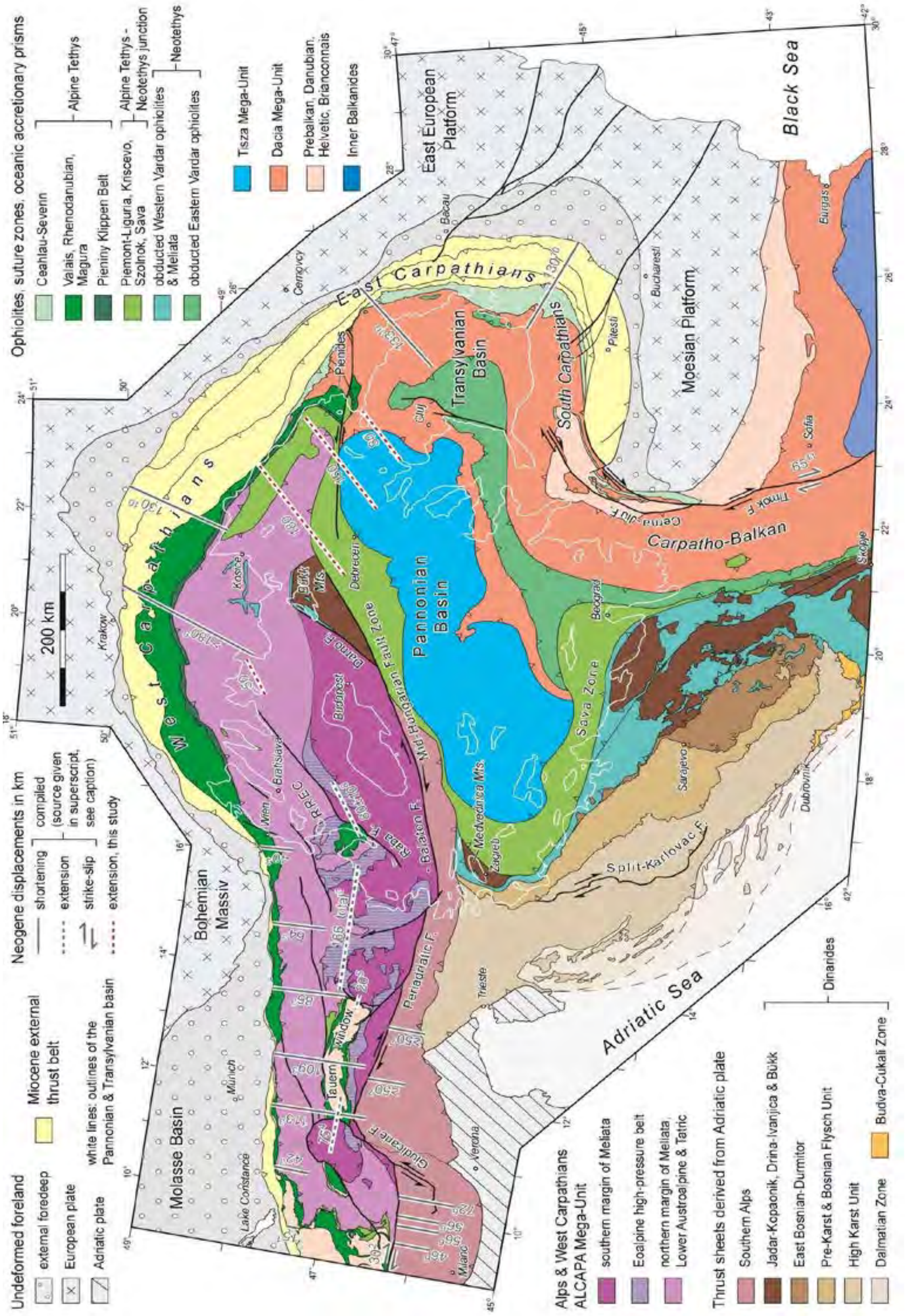

Fig. 1.2.4: Tectonic map of the Alps, Carpathians and Dinarides (simplified after Schmid et al. 2008), serving as a base for the Early Miocene restoration.

REFERENCE: A map-view restoration of the Alpine-Carpathian-Dinaridic system for the Early Miocene. (Ustaszewski et al.,2008, p.3)

1.2.1 Morphology

Two major morphological units are identified in the territory surrounding Timisoara: the area of the hills and the area of terraces and plain. Two hilly areas, constituted by crystalline schists, stand at the south-east of Timisoara plain: the banatic massifs of Borsa and Ocna de Fier – Bocsa Montana, and the northern extremity of the island of crystalline schists between Oravița et Bocșa Montana. These hilly area present a slightly accentuated slope and their height varies from 350 to 600 m. The main rivers, Timis, Berzava and Beregsau, dug a system of several terraces that, according to their relative altitude, determinate different levels; Timisoara is located in the so called "high terrace", with a relative altitude between 80 and 100 m.

1.2.2 Geological framework

Most of the territory near Timisoara is coated by recent deposits of Quaternary era, covering the formations of Pannonian Basin, where the crust has a thickness of 26–28 km and the transition limit between the upper to lower crust is at 15–20 km. In the south-east of the territory appear the crystal, eruptive and sedimentary formation of the western extremity of the Banat's mountains together with Neogene deposit of the Lugoj and Caransebes basins. These sedimentary formations and Quaternary ages cover unconformable a Proterozoic-Paleozoic crystalline basement and have thicknesses of about 1750 m in Timisoara area[7].

The bedrock of the land near Timisoara is formed by crystalline schists and by eruptive masses whose establishing, metamorphism and tectonic occurred during the Pre-alpine folds; later they were affected by the Alpine folds. During Quaternary new movements of subsidence occurred. They are especially visible in the territory at western part of the territory near Timisoara where a series of rivers such as Pogonis, Cerna, Bega, Timiş etc. converge; in the oriental zone rivers keep the diverging character of terraces. Finally, the recent tectonic causes the establishing of basalts, during the Quaternary, at north and south of Timisoara[8].

[7] *Site effects investigation in the city of Timisoara using spectral ratio methods.* (Oros, 2009, p.350)
[8] *Harta Geologica Scara 1:200,000, 24 Timisoara* (Dragulescu, Hinculov and Mihaila, 1968)

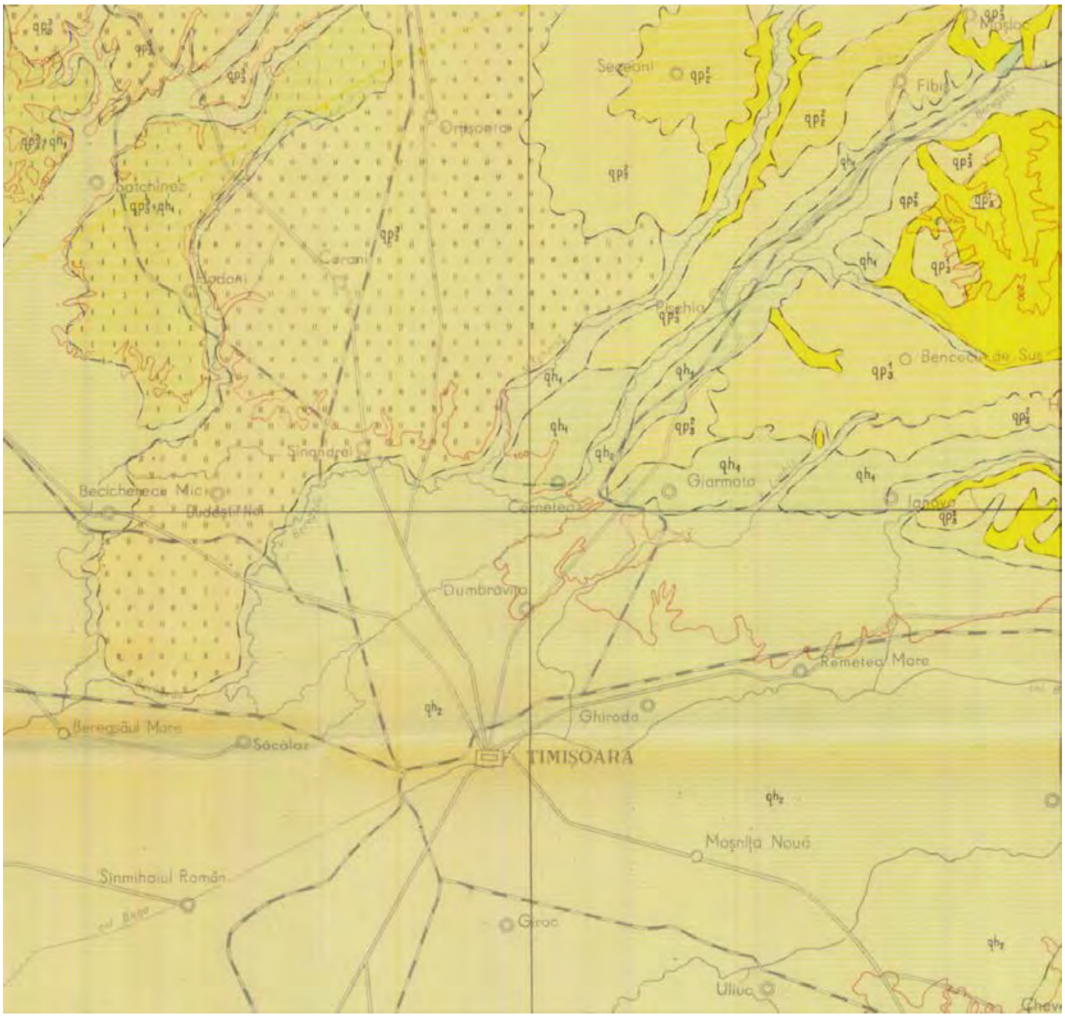

Fig. 1.2.5: Geological map, scale 1:100000 of Timisoara area

REFERENCE: Harta Geologica Scara 1:200,000, 24 Timisoara L-34-XXII (Comitetul de Stat al Geologiei – Istitutul Geologic, 1968)

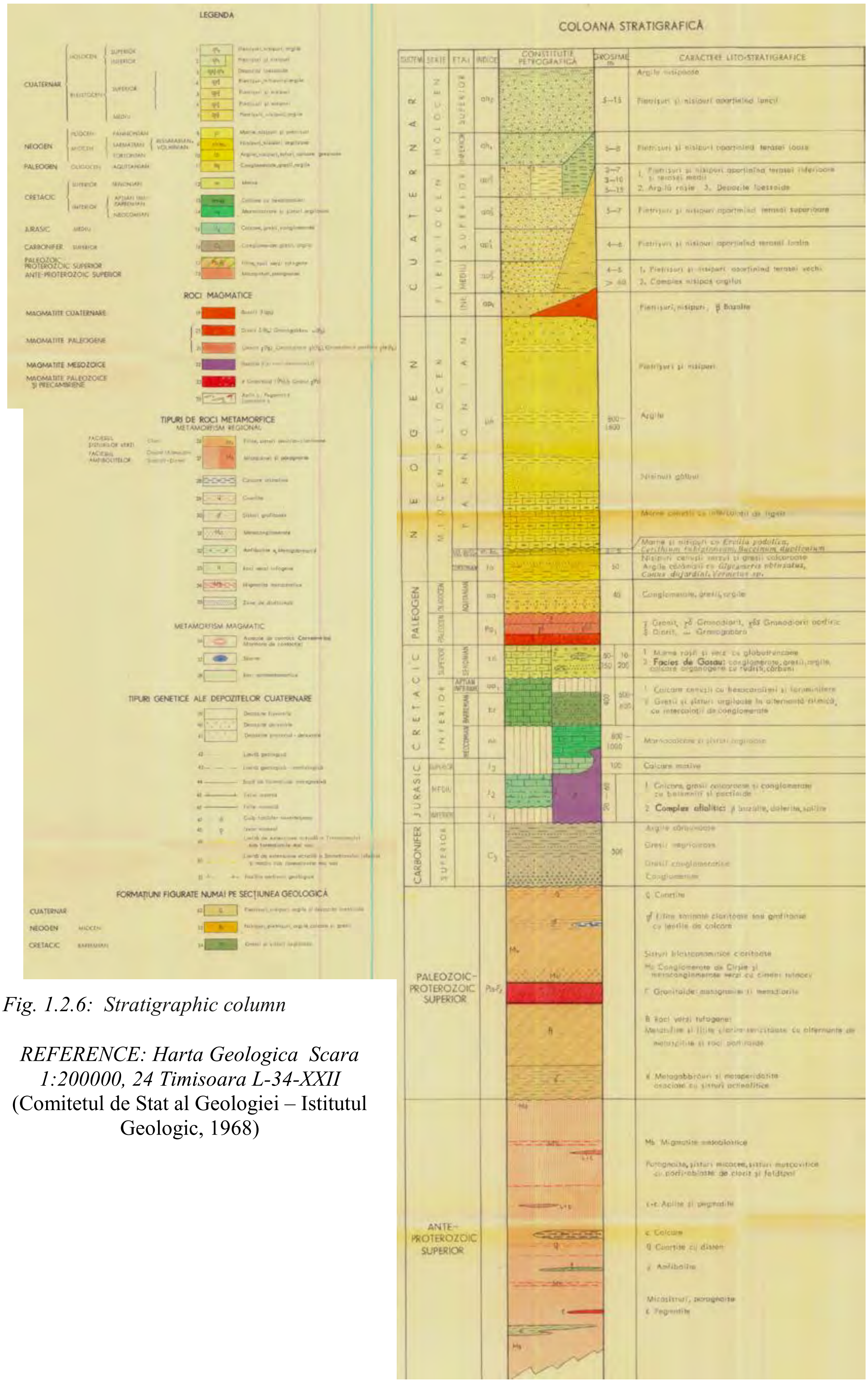

Fig. 1.2.6: Stratigraphic column

REFERENCE: Harta Geologica Scara 1:200000, 24 Timisoara L-34-XXII (Comitetul de Stat al Geologiei – Istitutul Geologic, 1968)

The Banat region is spread over the geotectonic units of Inner Dacides, Transylvanides and Middle Decides, until the unit of South Carpathians. The units are divided between them and on their internal by several faults and by the tectonic limits of nappes. Timisoara is located in the geotectonic unit of Transylvanides, is divided from the Inner Dacides at north by the tectonic limits of a nappe and from the Middle Decides at south by various fault lines.

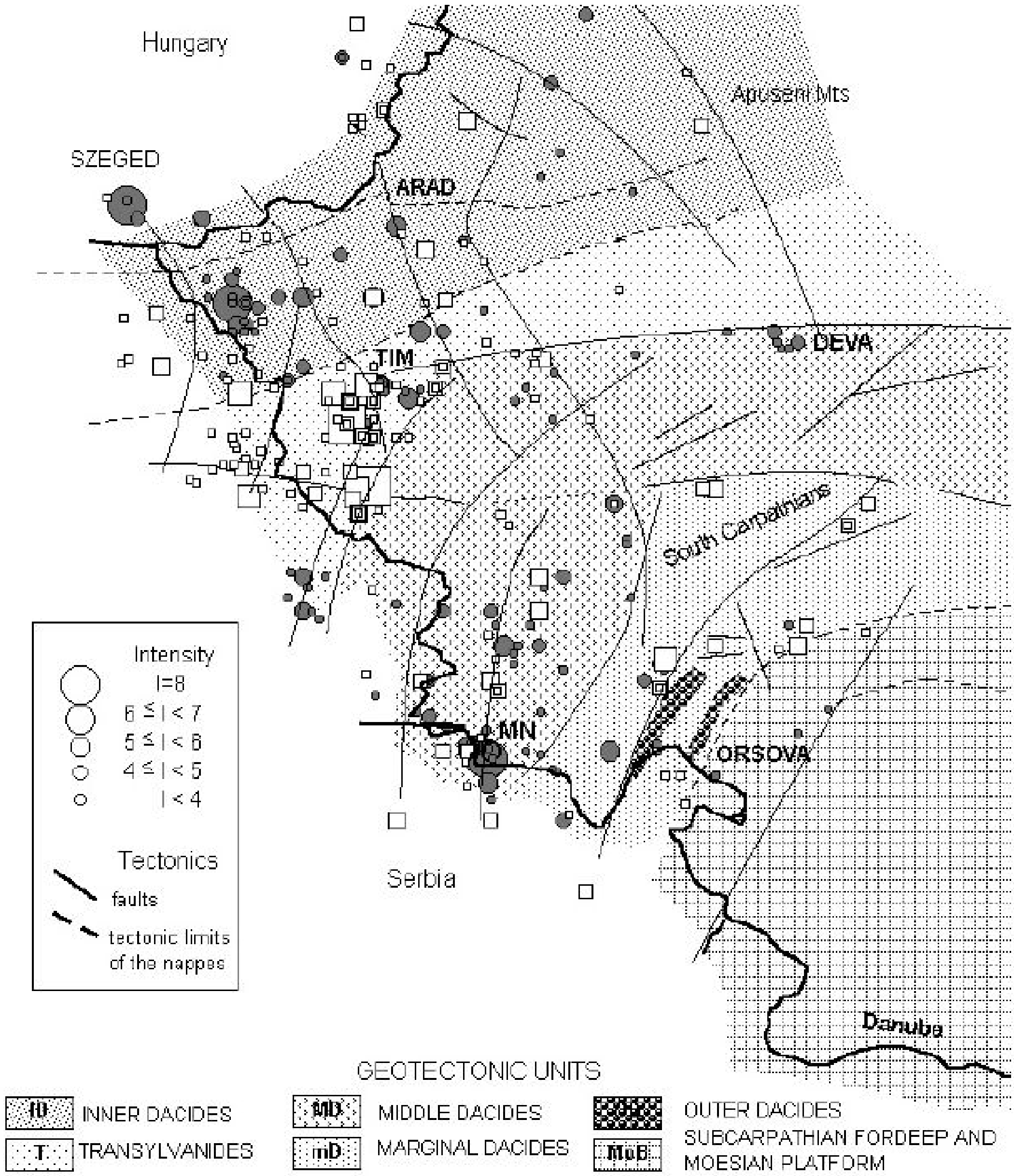

Fig. 1.2.7: Distribution of epicenters of earthquakes with $I_0 \geq VI$ on MSK scale produced in the seismic zome of Banat and surroundings areas

REFERENCE: Seismele din zona Banat – Timişoara (Marin, Roman and Roman, 2011, p.25)

1.2.4 Geotechnical setting

Bega River, crossing the city from east to west, affected the soil composition of the city territory. The geotechnical map of Timisoara shows, indeed, that clays and silty clays were mainly in the northern part of the city, while the dominant soil type in the southern part was a mixture of clay and sand, with a reduced compaction index in many areas. The southern part of city is also characterized by a high level of underground water, (until 1-2 m under the ground level), while in the northern part the level is considerately lower. Between a depth of 10 to 20 m it is possible to find the superior soil strata sedimented during the Quaternary period, composed by gravels, sands and clays, mainly sandy and silty soils, of alluvial derivation.

The soil layer between 120 and 150 m of depth is the one used to evaluate the dynamic characteristics of the soil in different sites of the city.

As it is possible to see in Figure 1.2.8, the yellow areas, concentrated in the south of the city, are composed by fine silty sands and fine and medium sands with a minimum thickness of 6 m. The combination of sand and high levels of underground water can provoke, in case of strong earthquakes, liquefaction phenomena with serious effects on the constructions. The north and west part of the city are dominated by very homogenous silty clays and a thickness higher than 10 m. The transition between these two areas is made by isles of clay and sand, organized in overlapping layers (Figure 1.2.9).

In the area around the walls of the citadel some old swamps and the Bega river bed have been drained and later filled for a thickness between 3 and 6 m. The nonhomogenity and the compaction degree cause a high seismic hazard.

The geotechnical map represents also the positioning of the inactive seismic fault lines in the west part of the city[9].

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[9] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Roman"* (Mosoarca et al., 2014, p. 4);
The influence of the local soil conditions on the seismic response of the buildings in Timisoara area. (Marin and Boldurean, pp.1-4)

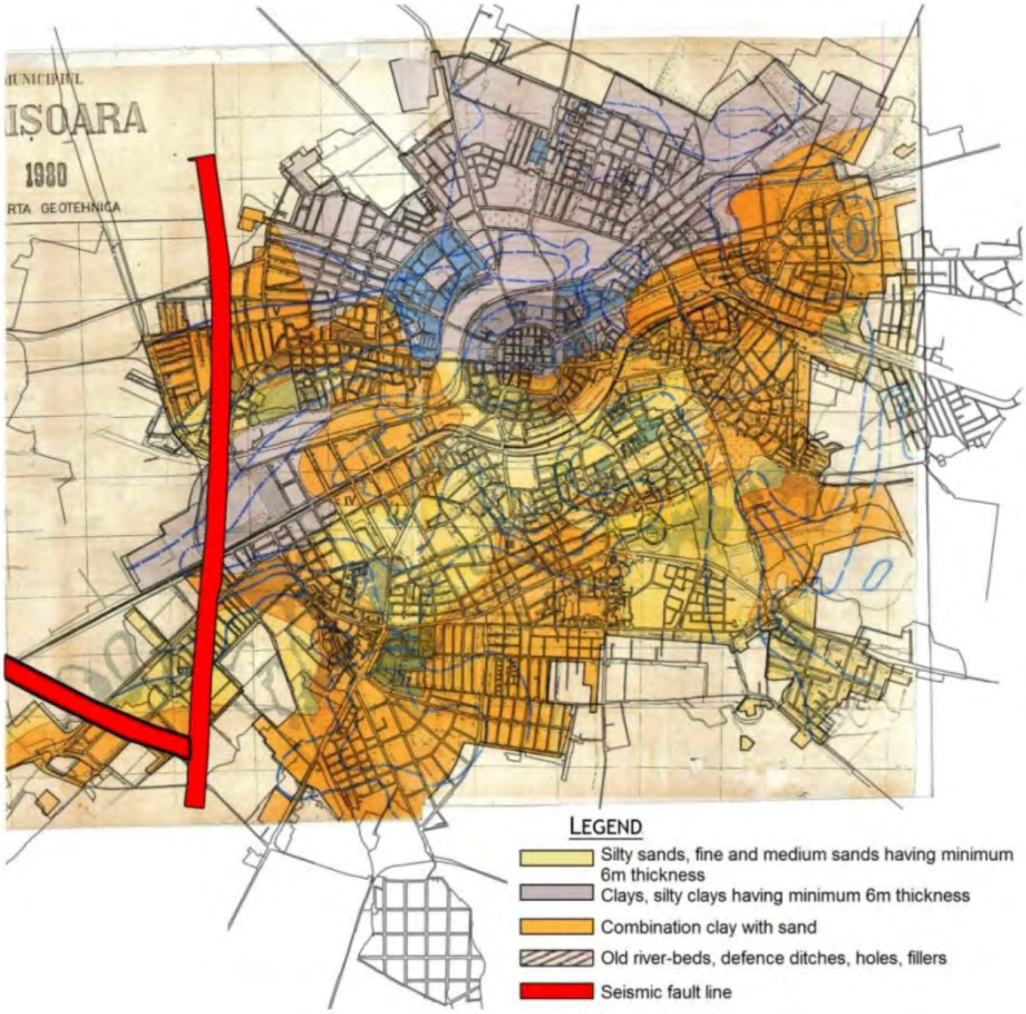

Fig. 1.2.8: Geotechnical map of Timisoara and location of seismic fault line

REFERENCE: Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania (Mosoarca et al., 2014, p. 4)

The two corings in Figure 1.2.9 were pulled put from two deep wells for the geological exploration and geotechnical drilling in the city. The one on the left represents the northern part of Timisoara (Torontal Way-Arad Way- Lipova Way) with a profile mainly clayely (86% of clay and 14% of sand), while the one on the right represent the southern part of the Bega Canal, characterized by sandy profile (32% of clay and 68% of sand).

These studies underline the following characteristics:

- the first 50-60 m of underground soils define the behavior of the soil at dynamic loads;
- the average density of the soil is ρ= 1,9-2,0 g/cm^3;
- the speed of the seismic waves is Vp = 350 m/s and Vs ≈ 200 m/s;

- the fraction of the critical damping is D = 12% for sands and 8% for clays and in the case of Banat area earthquakes M = 10^{-2} -10^{-1} %.[10]

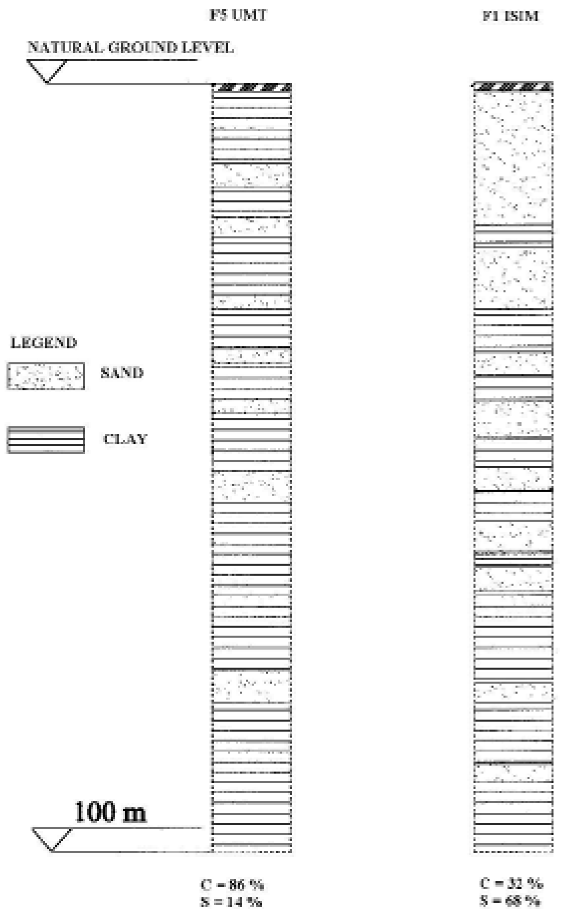

Fig. 1.2.9: Characteristic geotechnical profiles for Timisoara

REFERENCE: The influence of the local soil conditions on the seismic response of the buildings in Timisoara area. (Marin and Boldurean, p.4)

[10] *The influence of the local soil conditions on the seismic response of the buildings in Timisoara area*. (Marin and Boldurean, p.4)

1.3 SEISMICITY

The lithosphere, the external part of the planet, is composed by plates moving, colliding and pressing against each other. This movements caused deep conditions of effort and energy storage that, when the rocks exceeded the limit of their strength, can form deep cracks called faults and provoke the release of energy in the form of earthquakes. The strength of an earthquake can be define using two different measures scales: the magnitude, that transform the energy released into a numeric value of the Richer scale, and the macroseismic intensity, that measures the effects and express them in degrees of Mercalli scales.
Seismicity is a physical characteristic of the territory and considerates the frequency and force of earthquakes. It depends on three factors:

- The **seismic- hazard** "measures the probability that in a given area and in a certain time interval occurs an earthquake that exceeds a certain threshold of intensity, magnitude or peak acceleration (PGA). It depends on the type of earthquake, distance from the epicenter and geomorphological conditions. It's not possible to prevent the earthquakes or to modify their intensity or frequency. The knowledge of the hazard is useful in order to calibrate the interventions. The seismic classification determines the hazard and quantifies the reference actions in every area" [11].
- The **vulnerability** "expresses the probability that a certain type structure may suffer a certain level of damage as a result of an earthquake of certain intensity. The measure depends on the definition of damage, linked to the loss or reduction of functionality. The expected damage can be reduced by an improvement of the structural and non-structural characteristics of the buildings. The interventions are calibrated regarding to the hazard and to the expected performances"[12].
- The **exposure "**measures the presence of assets in risk and therefore the possibility of suffering a damage (economic, human life, cultural heritage,

[11] *Il rischio sismico* (Protezione civile nazionale)
[12] *Ibidem*

etc.…). Before the event it measures the quantity and quality of assets exposed, after the event it values losses caused by the earthquake. Exposition can be reduced by designing the territory, acting on the building distribution and density, on infrastructures, on the use destinations"[13].

1.3.1 In Romania

From seismic point of view, Romania is considered as a country having a large seismic risk. The seismicity of the country is focused in several epicentral areas: Vrancea, Fagaras-Campulung, Banat, Crisana, Maramures and Dobrogea de sud. Other epicentral zones of local importance can be found in Transylvania, in the area of Jibou and Tarnava river, in northern and western part of Oltenia, in northern Moldova and in the Romanian Plain, in particular for the west part of the country also the Hungarian areas of Szeged and Bekes and the Serbian areas of Alibunar, Srbsky Ittebej, Kikinda, Becej[14].

[13] *Ibidem*

[14] *Seismicity of Romania* (National Institute for Earth Physics, 2013)

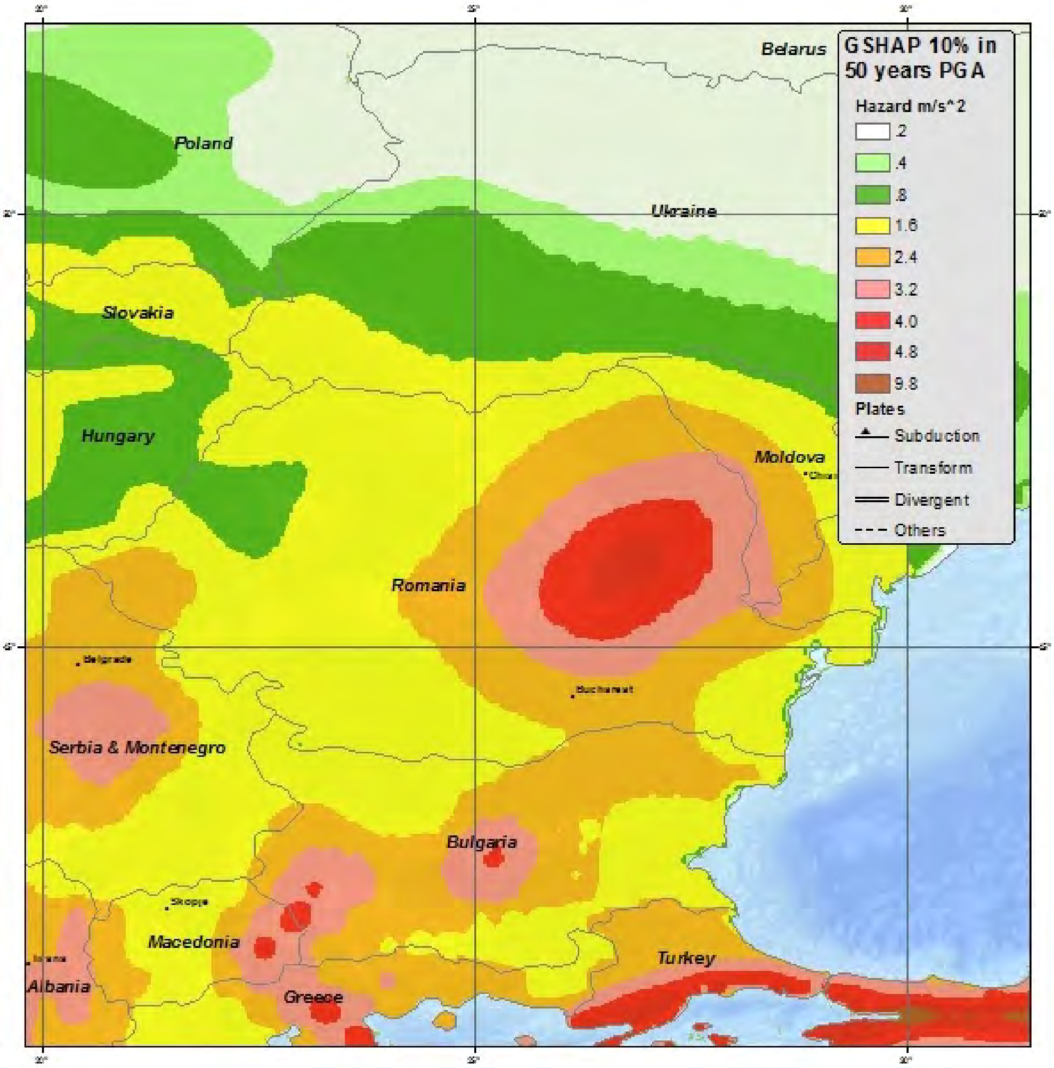

Fig. 1.3.1: Seismic hazard map in terms of peak ground acceleration (m/s^2) with 10% probability of exceedance in 50 years

REFERENCE: Romania-Seismic Hazard Map (USGS, 2005)

1.3.2 In Banat region

The western part of Romania is characterized by the contact between the Pannonian Depression and the Carpathian Orogen. In this part of Romania two distinct seismic areas can be defined on the basis of the seismicity distribution: Banat zone at south, and Crisana-Maramures zone at north[15]. In Timisoara, located in the Banat Seismic Region (RSB), the local earthquakes are perceived more intensely than the Vrancea ones.

[15] *Seismicity of Romania* (National Institute for Earth Physics, 2013)

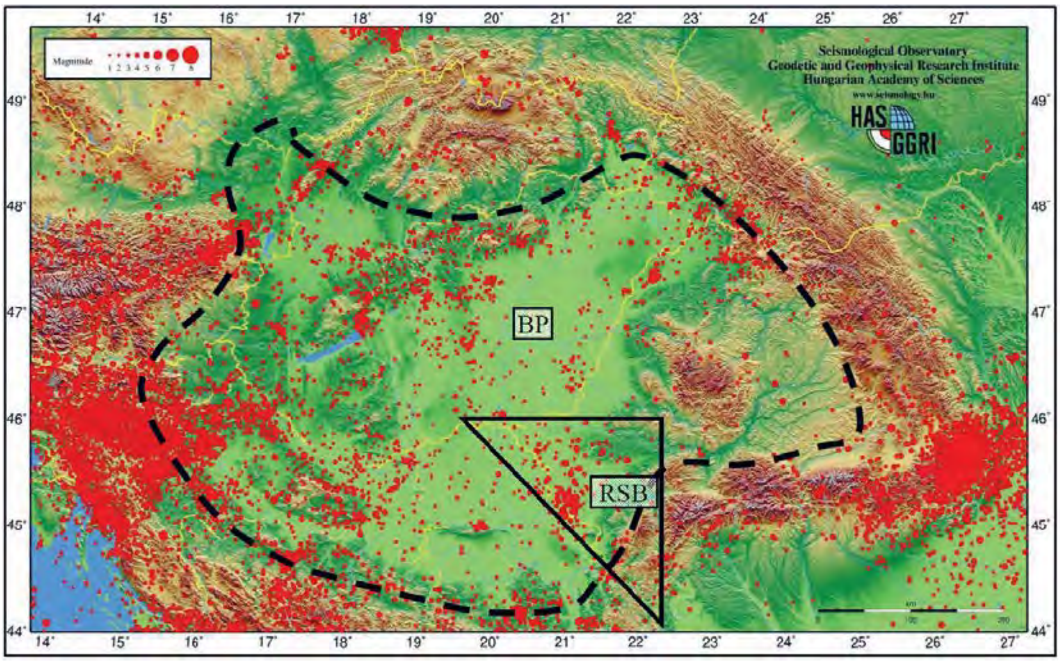

Fig. 1.3.2: Earthquakes in and around the Carpathian basin between 456 and 2006. Symbols are proportional to Richter magnitude, the triangle defines the Banat Seismic Region (RSB) and the dashed line represents the approximate limit of the Pannonian Basin (BP)

REFERENCE: Seismicity and seismic hazard in Hungary (Kovesligethy radò szeizmologai obszerbatorium, 2013)

Considering the energy and the number of seismic events, the Banat region is considered as the second most important seismic zone, being subjected to shallow earthquakes of crustal type. The earthquakes of this area are characterized by a small depth of the seismic source, between 5 and 15 km, however less than 30 km with a reduced surface of the epicenter area where the effects are greatest[16]. Earthquakes in Banat can be cataloged as monokinetic earthquakes, considering the division in two categories made by Prof. I. Atanasiu about seismic shocks. They are characterized by a relative small number of pre-shocks, followed by a large number of after-shocks[17].

The main faults have different orientations and depths, but the reverse and strike-slip faulting are predominant. Moreover the earthquakes of the region are

[16] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarca et al, 2014, p. 2)

[17] *Romanian seismology – historical, scientific and human landmarks* (Rădulescu, 2008, p. 6)

characterized by a strong directionality of the source, that is manifested by the elongated shape of isoseismals in the direction of the causative fault.

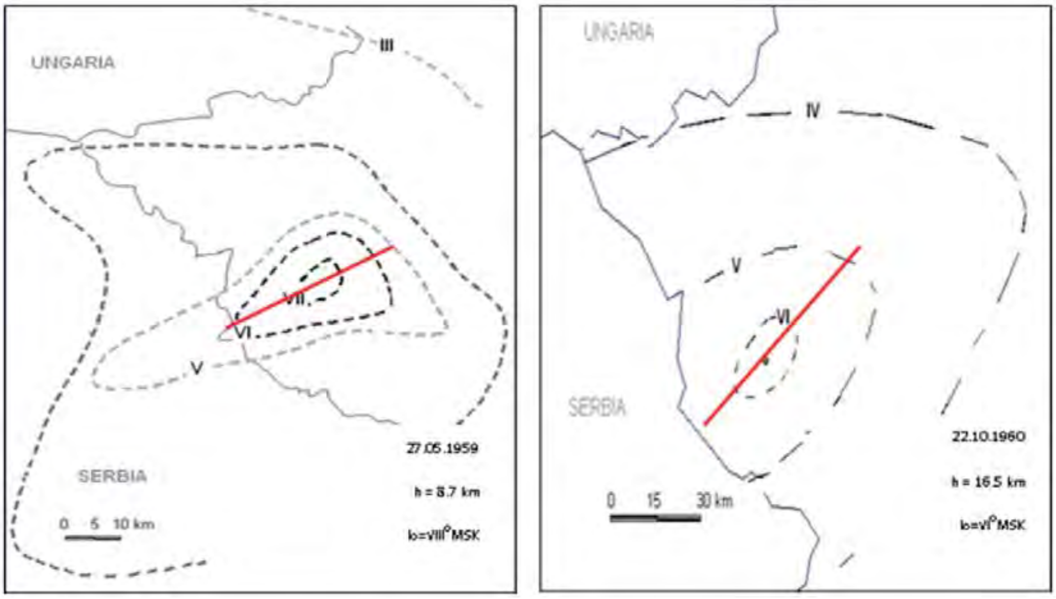

Fig. 1.3.3: Elongated isosmeismals in the direction of the seismic fault line (in red).
REFERENCE: Seismicitatea, seismotectonica şi hazardul seismic din zona Timişoara. (Oros, 2012, p.19)

The greatest earthquakes from this region have a seismic sources usually located at the intersection of seismic faults or near geological faults of different ages. The distribution of epicenters of the region recorded earthquakes shows that they can be clustered in three major areas, called "clusters", defined by important seismic events having M≥5.0: the first one near Timisoara in correspondence of Sag (M=5.4 on 27 May 1959) and Timisoara-Sacalaz (on 5 June 1443 and on 19 November 1879), the other two located at a critical distance from Timisoara, causing minimal effects on the buildings, one in Volteg-Barloc (M=5.6 on 12 July 1991 and M=5.5 on 2 December 1991) and the other between Lovrin and Vinga (M=5.3 on 30 August 1941 and M=5.2 on 8 July 1938). These clusters are concentrated on fault intersections, which had different principal directionalities: northeast-southwest, east-west and northnorthwest-southsoutheast[18].

[18] *Seismicitatea, seismotectonica şi hazardul seismic din zona Timişoara* (Oros, 2012, p.17)

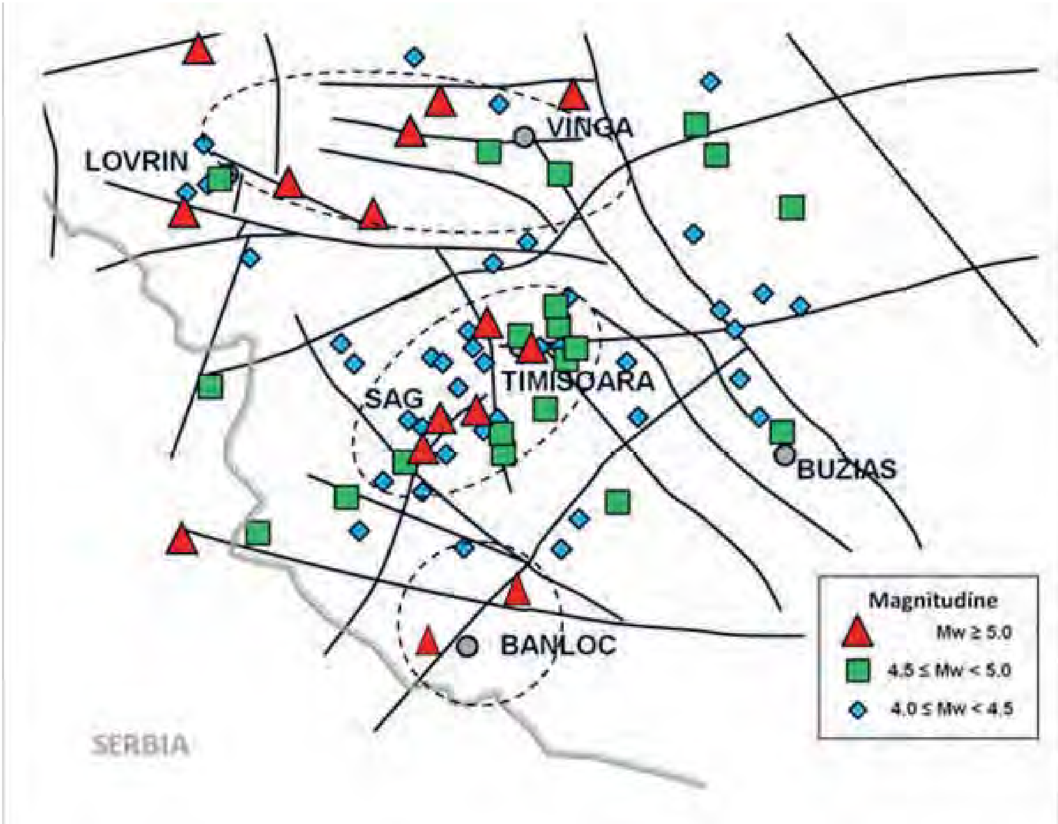

Fig. 1.3.4: Schematic representation of the seismotectonic of the area and relatives fault lines. REFERENCE: Seismicitatea, seismotectonica şi hazardul seismic din zona Timişoara. (Oros, 2012, p.18)

Regarding the intensity on MSK scale, Timis County includes zones having the mean earthquakes recurrence interval of 50 and 100 years, which can be evaluated in terms of peak ground acceleration, between ag=0.10g and ag=0.25g. As a result, a lot of existing buildings were not designed for seismic actions, or were designed for much smaller values of ground acceleration[19].

[19] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarca et al., 2014, p. 3)

City	No. of dwellings (2011 Census)	No. of inhabitants (2011 Census)	Seismic intensity (MSK)	T_c (sec)	a_g for MRI=100 years
Timisoara	124777	304467	VII	0.7	0.20g
Lugoj	14369	37321	VII	0.7	0.15g
Buzias	2273	6504	VII	0.7	0.15g
Deta	2209	5963	VIII	0.7	0.20g
Jimbolia	3615	10048	VII	0.7	0.20g
Sanicolau Mare	4079	11540	VII	0.7	0.20g
Ciacova	1608	5028	VIII	0.7	0.25g
Gataia	1839	5449	VII-VIII	0.7	0.15g
Recas	2679	7782	VII	0.7	0.20g

Fig. 1.3.5: Macro-seismic characteristics of most important cities from the Banat region in the year 2009

REFERENCE: Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romani" (Mosoarca et al., 2014, p. 4)

In the following table (Figure 1.3.6) the maximum intensities perceived in Timisoara are represented, as a result of Banat and Vrancea major seismic events. The detection station is situated in the street Calea Buziaşului, nr. 3-5, 3 km far from the center of Timisoara (in the south east direction). It is possible for Banat region to reach a maximum magnitude of M=6-6.5, corresponding to an intensity of $I_0 \geq$IX MSK. The crustal character of the region earthquakes and the rapid attenuation of the seismic energy limit the maximum effects of a big earthquake in a small area[20].

Date	Deph (km)	Epicentral distance (km)	Magnitudo Ms	Intensity in Timisoara (MSK)	Epicentral Intensity (MSK)
Vrancea earthquakes					
26/10/1802	150	415	7.5	5	9
10/11/1940	133	423	7.8	4	9
04/03/1977	109	396	7.2	5	8-9
30/08/1986	133	407	7	3	8
30/05/1990	89	438	6.7	3	8

[20] Seismele din zona Banat – Timişoara. (Marin, Roman L. and Roman O., 2011, p. 26)

31/05/1990	79	437	6.1	2	7
Local earthquakes					
10/10/1879	10	121	5.3	3	8
17/07/1991	11	45	5.7	6	8
02/12/1991	9	36	5.6	5-6	8
24/03/1996	23	23	4.8	6.6	7.1

Fig. 1.3.6: Intensity observed in Timisoara, street Calea Buziaşului, nr. 3-5, based on macroseismic maps.

REFERENCE: Seismele din zona Banat – Timişoara.
(Marin, Roman L. and Roman O., 2011, p. 25)

Banat is monitored by three accelerograms: two of them are situated in Timisoara (Tram Factory and IAEM) and the other one in Barloc (Central Station). These stations are property of the National Seismic Network of the National Institute for Earth Physics in Bucharest and they record earthquakes until M<1. Records and response spectra indicates a low seismic activity during last years.

1.4 HISTORICAL EARTHQUAKES

1.4.1 In Romania

One to five shocks with a magnitude higher than 7 occur in the country each century and are felt over a very large territory, from the Greek Islands to Scandinavia, and from Central Europe to Moscow[21].

Remarks on damages and human victims produced in Romania were mentioned in many medieval chronicles of the XVIII century. The oldest estimated seismic event occurred in Vrancea on 29 August **1471**, with an evaluated magnitude of 7.3. Other strong shocks of M=7.3 occurred on 9 August **1679** and they produced great damages and many churches and houses collapsed. On XVIII century the major earthquakes occurred on 11 June **1738** (M= 7.5), causing very wide extent heavy damage at Iasi and Bucharest; 5 April **1740**, with M=7.3 in epicentral area; 6 April **1790** with M=6.8. The XIX century opens with “The Big Earthquake of God’s Friday” on 26 October **1802**, that was considered the strongest earthquake of Romania with M=7.7.

The 1802 earthquake in Vrancea region lasted 2 minutes and 30 seconds and produced great damages, especially in Bucharest, where was demolished the Tower of Coltea and numerous churches, and major destructions were also produced in Transylvania and in Moldavia. It was followed by the one on 26 November **1829,** characterized by M=7.3 in epicentral area and felt over a very large area from Tisa to Bug and from Mureş to the Danube, with heavy damages in Bucharest. Other major earthquakes were registered on 23 January **1838**, felt over a wide area, in Romania, Hungary, Ukraine and Balkan Peninsula: they caused very heavy damages in Wallachia and southern Moldavia with a magnitude of 7.3[22], on 17 August **1893**, 31 August **1894** and 6 October **1908**, all in Vrancea with a magnitude of 7.1.

One of the most important earthquakes of XX century occurred on 10 November **1940** in Vrancea region, at 03.39: it lasted 45 seconds, with M= 7.4 on the Richter

[21] *Seismic Hazard of Romania: Deterministic Approach* (Radulian et al., 2000, p. 221)

[22] *Romanian seismology – historical, scientific and human landmarks* (Rădulescu, 2008, p. 1)

scale. The death toll was estimated at 1000 dead and 4000 wounded, mostly in Moldova, but the exact number of victims was not known because of the war context in which it was produced. The earthquake was felt on more than two million square kilometers and it devastated Wallachia and part of Moldavia. Shocks were felt up to the east of Odessa, Krakow, Poltava, Kiev and Moscow where it caused some damages (estimated intensity V-VI). Northwards the macroseismic area spread up to Leningrad, to East it spread over the Tissa river and to south-southwest in Yugoslavia, throughout Bulgaria and further to Istanbul. In Romania two areas of maximum intensity have been identified: the region between Panciu, Focsani and Beresti and the region in the Romanian Plain between Campina and Bucarest. It is believed that in the two regions the level of intensity of the earthquake exceeded everywhere VIII grade on Mercalli-Sieberg scale, approaching more the IX grade, which apparently has been exceeded for numerous villages in these regions. In Panciu the maximum value was recorded, with an intensity estimated on X grade. In Vrancea however, the intensity was lower, between grade VI and VII-VIII[23].

Another important earthquake occurred at 21:22 on 4 March **1977**. It had a magnitude of 7.2 on the Richter scale and a duration of about 56 seconds, causing about 1570 of victims, of which almost 1400 only in Bucharest. It was felt throughout the Balkans, with the epicenter in Vrancea region at a depth of 94 kilometers. About 35000 buildings were damaged, and the total damage was estimated on more than two billion dollars. Most of the damage was concentrated in Bucharest, where about 33 large buildings collapsed, so after the earthquake, the Romanian government imposed tougher construction standards. The shock wave was felt in almost all countries in the Balkan Peninsula, as well as Soviet republics of Ukraine and Moldavia, even if having a lower intensity. The seismic event was followed by aftershocks of lower magnitude, of which the strongest occurred on the morning of 5 March 1977, at 02:00, at a depth of 109 km, with a magnitude of 4.9 on the Richter scale, while the others did not exceed M=4.3 or M=4.5[24] [25]. The earthquakes caused damages to many

[23] *Cutremul din 10 noiembre 1940 – date sintetice* (Inforix

[24] *Cutremurul din 4 martie 1977 - 55 de secunde de cosmar* (Ilie)

architectural monuments and the Ceausescu's regime used this pretext to demolish a series of buildings. Once begin, the intention to eliminate the biggest number of architectural monuments and churches possible has been extended to art collections, with the pretext of safety of the works[26].

In the latest years of XX century two major earthquakes occurred: the first one on 30 August 1986 at 21:28 in Vrancea region, and the second one between 30 and 31 May of 1990, in Vrancea region too.

The **1986** earthquake killed more than 150 people, injured over 500, and damaged over 50000 homes, with M=6.5 on the Richter scale[27]. The seismic source was located at a depth between 131 and 148 km, as revealed by the location of aftershock hypocenters. The strongest aftershock occurred in the morning of 2 September 1986, at 5:00, at 143 km depth, with M=5, and was felt in Bucharest with an intensity of about III-IV degrees on the Mercalli scale. In total, 77 aftershocks were recorded with M≥3.2 on the Richter scale, of which 19 exceeded the value of 4.0[28]. The **1990** earthquakes were on 30 and 31 May 1990 measuring 6.7, 6.2 and 6.1 on Richter scale, on two consecutive days, at a depth of 89 km. Severe damages in the Bucharest-Braila-Brasov area and dozens of casualties in Romania, Moldova, Ukraine and Bulgaria were reported. The last two quakes occurred about 2.3 seconds apart and were followed by a series of weaker replicas: during the first 17 hours from the main shock about 80 replicas were recorded. Analysis of seismograms showed that the strongest replica occurred on 31 May, at 3:18, measuring 6.1 on Richter scale. It was felt in the epicentral area with an intensity of about VI degrees on the Mercalli scale, and in Bucharest with an intensity of about V degrees on the Mercalli scale. Just three seconds apart, another replica of M=6.2 struck the Vrancea County.

Since then, just one earthquake in **2004** touched M=6, while all the other major events measure from 5 to 5.5 degrees[29].

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[25] *Significant Earthquakes of the World – 1977* (USGS, 2005)

[26] *La triste sorte delle chiese di Bucarest negli anni del comunismo* (Cultura romena, 2008)

[27] *Significant Earthquakes of the World – 1986* (USGS, 2005)

[28] *Seismicity of Romania* (National Institute for Earth Physics, 2013)

[29] *Significant Earthquakes of the World – 1990* (USGS, 2005)

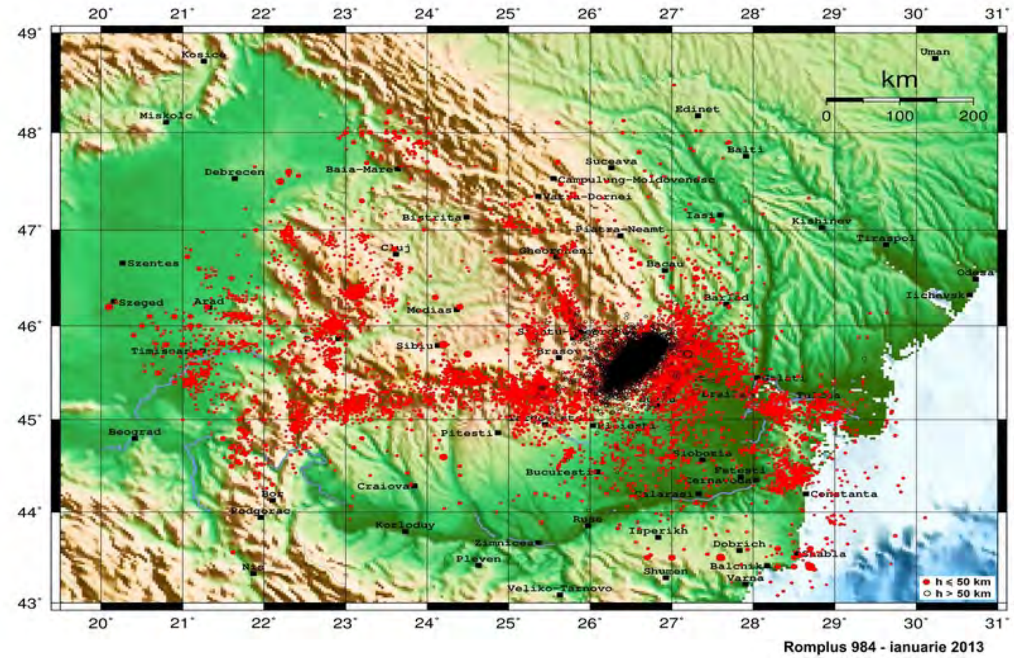

Fig. 1.4.1: Epicenters of the earthquakes occurred on the Romanian territory between 984 and 2013.

REFERENCE: Seismicity of Romania (National Institute for Earth Physics, 2013)

1.4.2 In Banat

From 984 until today in the wart part of Romania occurred 65 earthquakes with $I_{max0}\geq6$ MSK. From 1776 to 2000 occurred a large number of earthquakes with a I_0>VI, in particular 35 events with I_0>VI, 23 with I_0>VII and 7 with I_0>VIII. During the XX century the seismic activity focused on the center of Timis region: in 1991 the earthquake of Baile Herculane occurred with an intensity of I_0>VIII and M=5.6, followed by the events in Banloc –Voiteg on 12 July and 2 December of the same year, both with I_0=VIII and magnitude respectively of M=5.7 and M=5.6. These seismic events caused a revaluation of these areas, considered before with a maximum intensity of VI, and an updating of the normative values for this specific region[30].

Earthquakes of VI and VII intensity on the MSK scale of 1802, 1838, 1940, 1977, 1986 and 1990 recorded in Vrancea region, the most important seismic region

[30] *Seismele din zona Banat – Timişoara* (Marin, Roman and Roman, 2011, p.25)

from Romania, were also felt in the Banat region. Being close to the border, earthquakes from Serbia can also affect Timis County and the city of Timisoara[31].

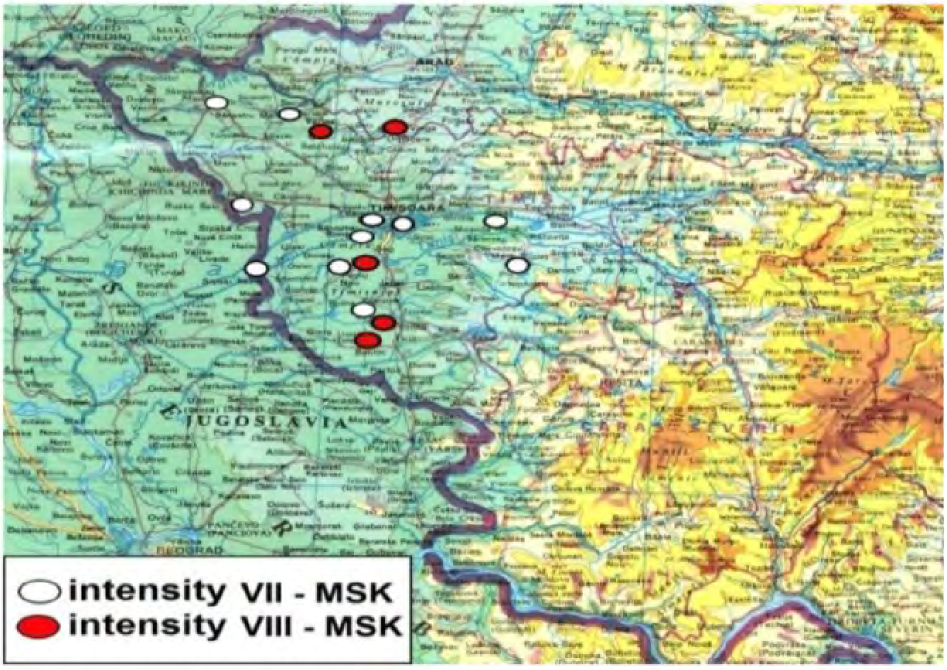

Fig. 1.4.2: Earthquakes from Banat region

REFERENCE: *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarcaet al., 2014, p. 2)

In the Table 1.4.1 data regarding strong earthquakes which occurred in the Banat region during the XVIII and XIX centuries there are given[32].

Epicentre zone	Maximum recorder intensity	Magnitude	Year
Periam – Varias	VII		1859
Sanicolaul Mare	VII		1879
Moldova Noua	VIII		1879
Timisoara (Mehala)	VII		1879
Carpinis	V		1889
Recas	V		1896

[31]*Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarcaet al., 2014, p. 3)

[32] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania"*(Mosoarcaet al., 2014, p. 3)

Barateaz	VII		1900
Recas	V		1902
Rudna – Ciacova	V		1907
Banloc – Ofsenita	VII - VIII		1915
Jimbolia – Bulgarus	VII		1941
Sanandrei – Hodoni	V		1950
Sag – Parta	VII		1959
Sanmihai – Sacalaz	VI		1973
Ortisoara		5.6	17 April 1974
Liebling – Voiteg	VIII	5.7	12 July1991
Baile Herculane-Mehadia	VIII	5.6	18 July 1991
Liebling – Voiteg	VIII	5.6	2 December 1991
Comeat		4.6	2 March 1992
Ivanda		4.6	19 December 1992
Rudna-Crai Nou		4.7	14 October 1994
Dinias-Peciu Nou		4.8	24 March 1996

Tab. 1.4.1: Zones with most important earthquakes, intensities, magnitude(when known) and year of occurrence.

REFERENCE: *Vulnerabilitatea seismică a zonelor de locuit din Timişoara.* (Budău, 2014); *Earthquakes Archive Search* (USGS, 2005).

The most important seismic events are: earthquakes occurred between October 1879 and April 1880 in Moldova Noua area; an earthquake occurred at a depth of 5 km near Timisoara city, on 27 May 1959, M=5.6, followed by two shocks occurred in 1960; earthquakes in Banloc, 12 July 1991, M=5.6, with a depth of 11 km, and Voiteg, 2 December 1991, M = 5.6, depth of 9 km, followed by a large number of aftershocks[33].

[33] *Seismicity of Romania* (National Institute for Earth Physics, 2013)

1.5 HISTORY OF TIMIȘOARA

1.5.1 Etymology[34]

History of Timisoara is documented from more than 730 years. All the variants of its name derive from Timis River, which flows into the Danube near Belgrade.
In the 101-103 and 105 B.C., the Romans under the Trajan Emperor conquered Dacia with two bloody wars and Banat was called "Dacia Ripensis".
It is still believed that the actual location of Timisoara corresponds to the Dacian village called "Zambara".
The Historiographer Ptolemy, mentioned this name during the second century B.C. It is not possible to know exactly the place where Romans founded a city called "Tibiscum", but it is probable that this city, named in another document as "Municipiu", was the ancent Timisoara.
During the barbarian invasions, especially by the Avars, "castrum Zambara" ruined and on its place grew up "Beguey", (the name is taken from the near river Beghei).
The place became an important military center, it was chosen for strategic reasons and it is situated at the confluence of Timis and Beghei rivers.
In 1212 "The city of Timis" (Castrum Temesiensis) is mentioned in a document of King Andrew II. He fortified Timişoara and the town became a "Castrum".

1.5.2 Antiquity[35]

Although the first document about the existence of Timisoara is dated on the XII century, the first traces of the human presence in the city dated back to the Neolithic Age. The region near the rivers of Mures, Tisa and Danube, was very fertile and offered perfect conditions for food and human settlement yet in 4000 BC. Archaeological remains attested the presence of a population of farmers, hunters, artisans, whose existence was favored by mild climate, fertile soil,

[34] *Timişoara multiculturale, tra sviluppo storico e articolazione etnica* (Cionchin, 2014)

[35] *Istoria Timişoare* (Munteanu and Leşcu);
Istoria Timisoarei (Enciclopedia Romaniei , 2014)

abundant water and forests. The discovery of tombs and vessels supported the hypothesis that, in the Bronze Age, there was a stable settlement.

1.5.3 Dacian and Roman period [36]

During Dacian period there was a demographic and economic progress, a specialization of crafts and expanded trade relations. Certainly, the city was inhabited during the Roman period, as in the following centuries, confirmed by the presence of late Roman Era, discovered after an explosion in 1854 in Mehala. Based on these materials some historians identified the correspondence between Timisoara and Zambara city, mentioned in the Tabula Peuntingeriana, but there were not inscriptions or other Roman monuments attesting the existence of the Zambara camp in the current territory of Timisoara.

Bronze coins and other physical evidences found in cemeteries and rural constructions indicated a continuity in dwelling, maintaining contacts with Roman and Byzantine civilizations.

Fig 1.5.1 Roman Dacia

REFERENCE: History of Romani (Pop and Bolovan, 2006, p.822)

[36] *Istoria Timişoare* (Munteanu and Leşcu);
Istoria Timisoarei (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005)

1.5.4 Committee of Timiş[37]

It was possible to find direct and indirect medieval evidences about the existence of the Thymes Castrensis or Castrum regium Themes in documents from 1177 to 1266; in 1175 the "Committee of Timis" was mentioned, which was a territorial administrative division of the Kingdom of Hungary, but the sources did not specify what was the economic and administrative center. During this period, the city occupied a rectangular area, the fortifications were surrounded by a moat fed by the river. Around the year 1030 the Magyars conquered the territory, later called Banat, the Kingdom finished in 1301 after the death of the last Arpadian King. During this period, there was many conflicts between Tatars, Byzantines and Ottomans.

In 1307 King Carlo Roberto D' Angiò ascended the throne and decided to build a stone fortress, much stronger than the old one and in the period between 1315-1323 the town became the royal residence. The royal palace was built by Italian craftsmen, and was organized around a rectangular court having a main body provided with a dungeon and a tower.

Timisoara remained the capital until 1325, then moved to Visegrad and finally to Buda.

Timisoara lost its political and administrative court but gained military importance; it was considered as one of the most important resistance outbreacks against the Turks.

In 1394, the Turks were defeated by the Wallachian in the Battle of Ruins. Later Turks defeated Christians in Nicopolis battle and then they devastated Timisoara and Banat region.

In 1440 John Hunyadi, who was considered the defender of Christianity, arrived in Timisoara and transformed the city into a permanent military encampment and moved there with his family. In 1443 an earthquake destroyed part of palace and of fortifications and many buildings.

[37] *Istoria Timişoare* (Munteanu and Leşcu);
Istoria Timisoarei (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005)

In 24 January 1458 Matthias Corvinus, the younger son of John Corvin who died in clashes against Turks, was elected king of Hungary.

An important event in Timisoara history was the Gheorghe Doja revolution. On 10 August 1514 he tried to change the course of Bega river to be able to enter more easily into the city, but he was defeated by attacks from both inside and outside the city.

The fall of Belgrade in 1521 and the defeat at Mohacs in 1526, caused the division of the Hungarian kingdom in three parts and the Banat became the object of contention between imperial and Turkish Hungarian nobility. After the death of Zápolya, Habsburgs obtained Transylvania and Banat, with Timisoara. This situation caused the Turks attack.

On 13 October 1551 an Ottoman army besieged Timisoara but on 27 October 1551 withdrew in Belgrade. Timisoara had to negotiate the surrender of the city.

After several battles against Turks, Timisoara fell into their hands and on 26 July 1552 164 years of Ottoman domination started. The city acquired not only an oriental character, but also hold an important place in Ottoman campaigns in Central Europe.

1.5.5 Ottoman domination[38]

After the conquest by the Turks Banat was organized as vilayet[39]. It included six sanjaks[40]: Timisoara, Lipova Cenad, Gyula, Moldova Veche and Orsova. In 1552 Timisoara became the new capital of the Ottoman province and, for more than 150 years, togheter with Belgrado, became a real military center.

Due to the strategic importance of the city, yet protected by marshes and natural fortifications, the Turks dig deep trenches around Timisoara to improve reinforcements.

[38] *Istoria Timişoare* (Munteanu and Leşcu);
Istoria Timisoarei (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005);
*Cultura otomană a vilayetului Timişoara (1552-1716) (*Feneşan, 2004*,* pp. 25- 73)

[39] The first-order administrative division of the Ottoman Empire. (Enciclopedia Treccani)

[40] Interior divisions of a valayet. (Enciclopedia Treccani)

For more than a century Timisoara was no longer the target of attacks or battles, but it had an important role to maintain under control Hungary, Transylvania and Romania.
Turks arrival did not transform the city into a Muslim center and the urban structure of the city remained the same: two distinct parts, the fortress and the city, with its suburbs.
The population was composed mostly of Christian people, most Romanian, Serbian, craftsmen and merchants Armenians, Greeks, Macedonians, Hebrew, while Muslims formed generally the privileged stratum.
In 1594 a Christian uprising interested Banat against the Ottoman power. Followed a strong offensive in Transylvania, the Christian army conquered Bocsa, Cenad, Nadlac, Pancota, Arad, Faget, Lipova and Vrsac, but Timisoara remained untouched.
Another attempt to retake the city took place in 1596, when an army of Sigismund Bathory, began the siege of the city. After 40 days of fruitless efforts, the besiegers retired.
Austrians knew the importance of Timisoara, that was attacked in 1695 and 1696. On the other hand also Turks understood the centrality of Timisoara and the fortifications reconstruction in 1703 operated by new sultan Ahmed II, demonstrated the importance of the city in Ottoman plans.
After 17 years there was the Austrian-Turkish war. On the 5 August 1716 Prince Eugene of Savoy conquered Timisoara. After a siege of 48 days, accompanied by repeated bombings, which destroyed much of the buildings of the city, the Ottoman garrison surrendered. On 12 October 1716 Turks surrendered and left the town. On October 1716, Prince Eugene of Savoy made his triumphal entry into a city hardly hit by a violent siege, "Gate of Prince Eugen" remembered that day.

Fig 1.5.1 Castle and the city of Timişoara as they appear in a lithograph from Turkish period
REFERENCE: *Istoria Timisoarei* (Enciclopedia Romaniei , 2014)

1.5.6 Habsburg rule[41]

The Peace of Passarovitz of 1718 enshrined that the Turks lost Banat, Oltenia and part of Serbia. Finally, on 28 June 1719, the King signed a decree that recognized Banat administration and established its headquarters in Timisoara, which became the capital of an important province of the Habsburg Monarchy and the residence of the main administrative structures.

Count Mercy has a strong impact on the organization and future development of the city; he created a large administration and imposed a new mentality for the city life. At the end of the XVIII century, Timisoara was considered one of the most beautiful and clean cities in Europe.

For more than 130 years, until the 1848 revolution, Timisoara experienced a quiet period without military action or serious upheaval and unrest. A modern city with architectural structure and economic life grew up: cultural, religious, healthcare institutions and a rapid population growth.

[41] *Istoria Timişoare* (Munteanu and Leşcu);
Istoria Timisoarei (Enciclopedia Romaniei , 2014)

From 1738 to 1739, the old district of Palanca Mare was devastating by a fire that made also a large number of victims. The city needed a new cemetery because of cholera and plague outbreaks.

In 1781 the Emperor Joseph II proclaimed Timisoara as "free royal city". King Leopold II renewed it in 1790. After Banat conquest, Viennese authorities started a long process of colonization. Inhospitable climate caused the death of many immigrants for malaria, and only the immigration process ensured the population growth. As a result, the share of German Catholics came to a point of the 50% of the total population.

The need to ensure good living conditions led to the reorganization of all the villages in the Banat, and the construction of new ones.

From the beginning, the administration showed a clear interest in urban development, the first action taken by the authorities after the conquest, was to repair walls and buildings, partly destroyed during the siege. Initially water played an important role in the development of the settlement, but in the XVIII century, the marshes were considered the main source of pestilence. Between 1728 and 1732, Bega river was regulated, creating a navigable channel between Timisoara and the lower part of Romania. Thus, the city was connected to Tisza and Danube Rivers and the transport by water with Central Europe became possible, before the advent of the railway.

1.5.7 Revolution of 1848-1849[42]

The Timisoara attack, held from 11 June to 9 August 1849, was one of the most important battle of the Hungarian Revolution.

On 18 March the mayor Johan Preyer proclaimed a popular assembly in front of the City Hall (Old City Hall today); it was a massive participation of strong freedom manifestation but the next day some local leaders tried to transform it in a revolution manifestation. Hungarians raised the banner of rebellion and separation from Austria, but the Citadel remained faithful to Vienna.

[42] *Timişoara sub asediu în timpul Revoluției Maghiare din 1848-49* (Both);
Istoria Timişoare (Munteanu and Leşcu)

The intensification of clashes between Hungarian revolutionaries and central authorities in Vienna generated tensions in Timisoara and on 3 October 1848 the Court of Vienna published an imperial rescript ordering the dissolution of the Hungarian Diet and cancelled virtually all previous concessions made to Hungarians.

The official rupture between officials in Vienna and Pest, generated complications in Timisoara and, on 10 October, General Rukavina introduced the curfew. In 1848, there was 3000 people living Timisoara and 14000 in the periphery (Fabric, Prince Charles Maier Mehala).

Due to this tense situation, the War Council and the Political - Administrative Committee were constituted in Timisoara, consisting of 14 members, between Germans, Serbians and Romanians (including Andrei Mocioni). From October 1848 Timisoara became the main coordination center against Hungarian revolutionaries in the Banat region.

On April 25, 1849 Hungarians besieged Timisoara, gradually occupying Fabric, Mehala, Freidorf, and they cut off the water to the Citadel. Inside the fortifications there were almost 15000 people (military and civilian); the City was bombarded with violence and the situation was desperate. There was no water, scant food and a typhus epidemic broke out. The siege went on for 114 days, until 9 August 1849; Timisoara lived the most dramatic days in the entire modern history and the revolution concluded with over 3500 victims and hundreds of buildings destroyed.

1.5.8 Voivodeship of Serbia and Banat of Temeschwar[43]

The Serbian Voivodeship together with the Banat of Temeschwar became a province of the Austrian Empire existed between 1849 and 1860.

The Habsburg Monarchy recognized to Serbs the right of territorial autonomy and Timisoara was designated as the residence of the province governor. The province is composed by Banat and Backa regions and northern Syrmian municipalities of Ilok and Ruma.

[43] *Evolutia istorica a aorasului* (E-Patrimonium Timiensis, 2005)

The city, as the capital of an imperial province, took advantage on economic privileges and regulations: the empire accelerated the rebuilding of the city after the partial destruction during the revolution.
The Empire developed thanks to the removing trade barriers to trade and the abolition of internal customs; the economical development was felt in Banat.
During that period in Timisoara many manufactories grew up and from 1845 to 1858 a prerequisites for broader economic development was introduced.
The administrative framework Banat of Timisoara was reincorporated in Hungarian kingdom in 1860.

1.5.9 The Kingdom of Hungary from 1860 to 1918[44]

On 1859 there was the unification of the Romanian Principalities and the end of the war between Austro-Franco-Piedmonts, with the defeat of Austria. Austria and Hungary were negotiating the control of the Banat territory, with the intention to place it under Hungarian administration. Andrei Mocioni, a Romanian leader, asked audience to the king to know the motivations about this choice, as the population was predominantly of Romanian nationality.
On the 27 December 1860, Emperor Franz Joseph I decided the annexation of Banat to Hungary.
Now cities were not military communities, they became frontier municipalities with mayors and municipal councilors. Firs, the city adopted a Magyarization policy; in this period, Timisoara had a fast economic and demographic development. Institutions invested significant sums for the local industry growth. Timisoara, thanks to Bega channel, was connected to Tisza and Danube river system, and was connected with important cities in Western Europe using the railways. In this period horse tram, telephone, public lighting were introduced and roads were paved. Timisoara lost its military importance and needed space for expansion so the old city and the suburbs were connected.

[44] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014);
Ibidem

1.5.10 World War the First (1914-1920) [45]

From 1914 to 1918, 12832 people left Timisoara for barbarian fronts, many didn't returned.

In September 1916 the curfew was introduced, rights and freedoms were suppressed and existing factories were used to weapons production. Life became difficult because of the massive increase of prices, the lack of food caused insufficient rationing for families. On 2 December 1917 over 4000 people gathered the streets asking the immediate conclusion of war.

The monarchy collapse caused social unrest and in the autumn of 1918 Hungarians, Swains, Romanians, Serbs and Jews created their own military advice in Timisoara.

Local political leaders and Hungarian officers formed “Sfatul poporului din Banat” ("Counsil of Banat people") and then, on the 31 October Otto Roth proclaimed the independence of the Republic of Banat, while the counsel of the Romanian officers tried to make the union of Romania with the Banat.

Serbs occupied the entire region; causing the intervention of French Army units, arrived in Timisoara on February 1919; Serbs lived Timisoara on July 1919.

On 28 July Aurel Cosma, designated Timis prefect, instituted Romanian administration in Banat, and on 3 August 1919 entered in Timisoara.

Swabians and Banat asked Romanians to maintain the old boundaries, but Banat was divided in three parts durign the peace conference in Trianon on the 4 June 1920. Two thirds of the territory of Banat took part of Romania and one third of the territory entered into the Serb-Croat-Slovene Kingdom; a small part remained in the composition of Hungary.

[45] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasulu. (E-Patrimonium Timiensis , 2005)

Fig 1.5.2 Romania during World War I

REFERENCE: History of Romani (Pop and Bolovan, 2006*,* p.837)

1.5.11 Interwar Period (1919 -1947) [46]

After the increase of territory, Romania became a multiethnic state and Timisoara itself developed a unique character.

In the years following the unification, Timisoara became an economic, financial and administrative center and it permitted Germans, Hungarians, Serbs to join social and political structures of Romanian system.

The 1923 Constitution represented an example of freedom and Timisoara was the model of non-discriminatory and effective participation of people in town activities.

During the period between the two World Wars, industries, trades, born schools and cultural associations developed, confirming the cultural values of Banat.

The city population defended integrity, values and democratic institutions of Romanian national state, indeed thousands of people participated to the antifascist meeting on 24 May 1936.

[46] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005)

The dictatorial regime, settled in 1938, annihilated the politic freedom of the city and in 1940 a totalitarian state of the extreme right took the lead. This occurrence had disastrous consequences and caused the deportation of a large number of Hebrew in Transnistria camps.

Fig 1.5.3 Interwar Romania

REFERENCE: History of Romani (Pop and Bolovan, 2006, p.838)

1.5.12 Second World War[47]

In the Second World War Romania entered alongside Germany, participating in the Reich campaign against the Soviet Union.

Timisoara was one of the cities that hosted refugees from Bessarabia, Bukovina and Moldova. Meanwhile the allied forces bombed Romania, in Timisoara Anglo-American bombing occurred between June 16 to July 3, 1944, causing great destructions in the districts of Iosefin and Mehela.

[47] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005)

After the coup of 23 August 1944, Timisoara lived difficult times; due to its roads and rail hub, with significant industrial potential, the city had strategic value for Hitler's military forces.

On 26 August 1944, the German Command in Timisoara was surprised by a Romanian soldiers attack and they surrendered without resistance. Hostilities continued on September of the same year, Wehrmacht units tried to take possession of Timisoara. On September 16, German tanks entered a lot of cities like Mehala, Freidorf and Fratelia, surrounding Timisoara. Romanian defenders resisted the enemy, the battles continued in the following days but on September 1944, Soviet troops entered the city.

Fig 1.5.4 Romania during World War II

REFERENCE: History of Romani (Pop and Bolovan, 2006, p.839)

1.5.13 Timisoara under socialist [48]

Entered into the sphere of influence of the USSR, Romania did not receive funds for reconstruction and was forced to pay reparations to the USSR, due to its position behind the “Iron Curtain”. This situation affected the population, giving

[48] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014)
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005;

rise to crisis. Romania was isolated for four decades, in particular Timisoara, a city having rich traditions: this isolation was deeply felt. This feeling bring the city to became a center of anti-communist manifestation

Since January 1945, 75000 Germans from Transylvania and Banat were sent to forced labor in the USSR. In Romania labor camps were established and Germans were deported there from 1946 to 1948.

Many Timisoara factories were obliged to product weapons for the Soviet Union and this situation caused the decline of Timisoara industry during the second half of the decade.

In 1950 Timisoara was losing its city status, under an alien regime that caused resistance movement. There were many insurrections attempts in the main cities of Banat, but the Soviet Regime suppressed them.

1.5.14 Student revolt of 1956[49]

The first anticommunist resistance movement were made by small groups of students and started on 23 October 1956. Their claims were the withdrawal of Soviet troops, the return to democratic freedoms, the abolition of Marxism-Leninism studies and the removal of Russian language from university curricula. The movement ended when authorities arrested 2000 students and expelled them from University, sending their leaders to prison. Cluj, Bucureşti and Iaşi students followed Timişoara example but on 5 November, the Russian army crushed the revolution in Hungary.

1.5.15 1989 Revolution[50]

After Gheorghe Georghiu-Dej's death, Nicolae Ceauşeascu became the General Secretary of the Romania's Comunist party. As first, he proclaimed the Romania's Socialist Republic and in 1967 he was appointed Chairman of the Council of State. He became popular for his policy of independence from URSS and during that period Romania had a rapid economic growth and improved living

[49] *Istoria Timisoarei* (Enciclopedia Romaniei , 2014)

[50] *Primii paşi ai României după 1989 către o integrare europeană şi euroatlantică* (Victor);
Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005);
Istoria Timişoare (Munteanu and Leşcu);

standard. Unfortunately, this liberalization period finished in the late Seventies when Ceausescu's personality cult started, Romania was isolated and the living standard drops significantly. At the end of the eighties, the regime was collapsing; the arbitrary eviction of pastor Laszlo Tokes from the Reformed Church of Timisoara planned for 15 December 1989 became a pretext for a popular uprising, which then became a revolution.

On 15 December many parishioners gathered in front of the church to prevent the eviction of the Shepherd. The crowd attracted a large number of people, students cried for the first time “Down Ceausescu!” “Down communism!” the Chairman sent police to disperse the crowd, the police arrested 930 people. In the afternoon of 17 December, the first martyrs of the Revolution of Timisoara dropped. In the following days, the resistance continued and authorities try to cover up the number of victims burning and burying the bodies.

On 20 December Timisoara revolutionary leaders asked Ceausescu and Government resignations, free elections, clarifing the oppression of Timisoara; criminal responsibility for those who gave the order to shoot people; immediate release of political prisoners.

On December 21, Ceausescu organized a big manifestation in Bucharest, against the "Hungarian hooligans" of Timisoara; the meeting turned into an anti-Ceausescu and anti-communist movement and in the same day revolution broke out in the largest cities of the country.

Few hours later, on 22 December 1989, at noon, Ceausescu took refuge in Bucharest. The situation was confused, several groups wanted to take power; in the evening of December 22 a group led by Ion Iliescu and Petre Roman, who organized the National Salvation Front, took the responsibility to lead Romania towards democratization.

December 22 was declared Romanian Revolution Victory Day, with a total 1104 dead and 3352 injured.

1.5.16 Romania today[51]

On 22 December an identified force declared as counter-revolutionary opened fire on civilians and military units in several cities, creating panic and confusion; this justified the summary judgment and execution of Ceausescu.

Thanks to political confusion, Iliescu and the NSF won the 1990 parliamentary elections, and two years later, the presidential election. NSF decided to dismantle public enterprises and they arrested more than 600 people, with the suspicion of terrorist attacks. When Iliescu privatized the first state enterprises and pushed through drastic austerity measures, he encountered violent resistance. In 1993, the government cut the subsidies for goods and services and thereby provoked a major strike movement.

In 1996, an opposition alliance of Christian Democrats, Social Democrats and National Liberals took over the government under the leadership of Emil Constantinescu and this was seen as a “real change”. At the same time, ultra-right figures increase their political influence, attempting to stoke up ethnic and racial tensions. There were violent clashes between the two camps. Romania stood on the brink of an ethnic civil war.

Since the fall of Ceausescu, right-wing and Socialist alternated in the government of the country. With the intention to enter in the European Union, rigorous austerity measures have been carried out and the last state-owned enterprises were privatized.

Romania have lived a prolonged economic crisis; the promises of prosperity and democracy have not been fulfilled and the perception of corruption in the policy system of the country is increased.

Today the Romanian people believes that the events of 1989 were a mistake.

[51] *Romania: Twenty years after the overthrow of Ceausescu (*Toma and Salzmann, 2009); *History of Romani* (Pop and Bolovan, 2006, pp. 667-696)

1.6 URBAN EVOLUTION

1.6.1 From XII to XIV century – Committee of Timis[52]

The first documentation about city organization dates back to XII century, when Timisoara belonged to the Kingdom of Hungary. The city developed on the top of one of the dry islands that emerged from the marshes that strongly characterized the territory and was organized around two principal areas: the "royal fort", strengthened with palisades, and the civil area, composed by rural wood construction.

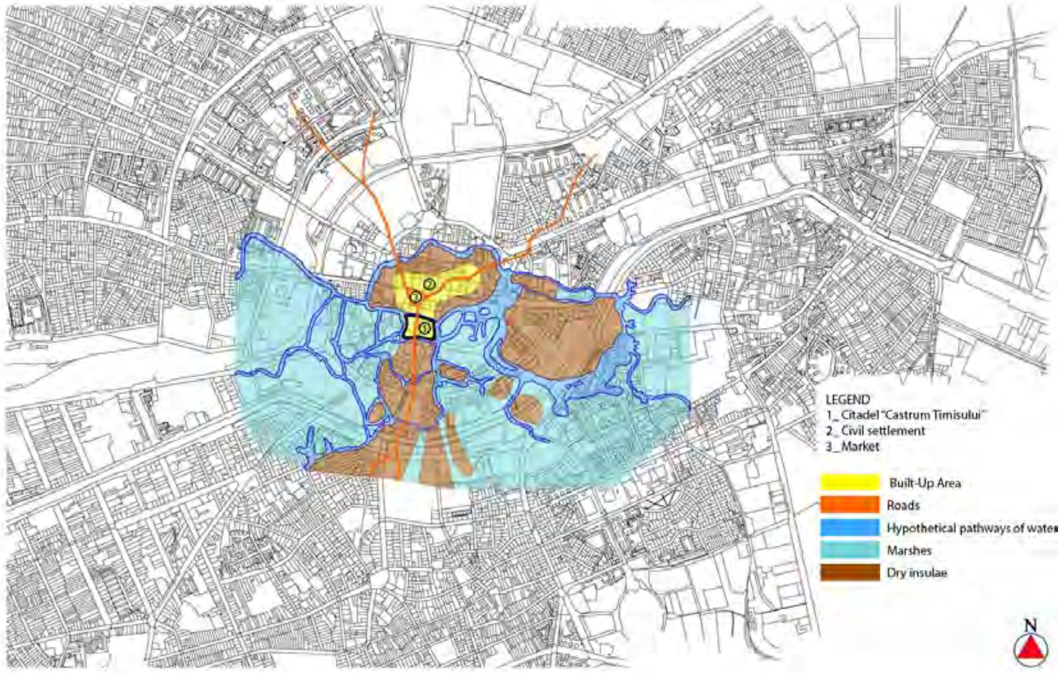

Fig. 1.6.1: Planimetric evolution of urban structure during XII and XIII century, until 1300.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 1)

1.6.2 During XIV and XV century – Committee of Timis[53]

After having been royal residence from 1315 to 1323, Timisoara obtained the state of "city". It was composed by four units:

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[52] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent* (Primaria Municipiului Timişoara, Plansa 1)

[53] *Ivi* (Plansa 2)

- the castle, that after king's departure became the administrative, politics and military center of Timis County;
- the original city, that together with the civil area formed the "proper city", hosting the principal urban functions;
- the east island, suburban area;
- the south island, that probably hosted functions and services related with the castle.

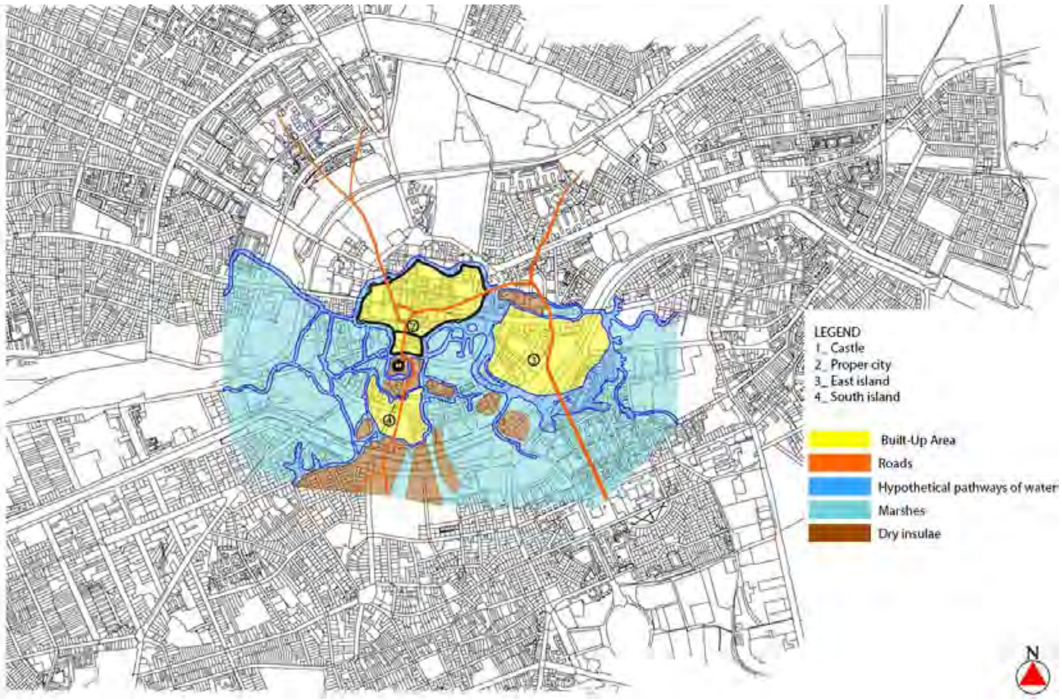

Fig. 1.6.2: Planimetric evolution of urban structure during XIV and XV century.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 2)

1.6.3 During XVI and XVII century – Ottoman Empire

After the conquer by the Turks in 1552, Timisoara became the capital of Ottoman vilayet and an important military center.
The urban structure remained similar to the previous one and it is composed by four units:

- the castle, which is the political and military headquarter;
- the proper city, which is the center of principal city functions;
- the district of Palanca Mare, so-called at the end of XVII century when it was fortified;

- the district of Palanca Mica, so-called at the end of XVII century when it was fortified too.[54]

The perimeter wall (5-6 km), having a thickness of 50-60 cm and five big openings, was equipped with battlements and small openings used for the 200 defensive cannons. Inside the city walls there were 1200 houses made of wood and mud, four mosques, four monasteries, seven schools, three inns, two bathrooms, four hundred shops and a bazaar. The town outside the city was composed of ten slums, included 1500 homes, each one having a yard and a garden, with separate entrances for carts and people. There were 10 places of worship, shops, but the bazaar lacked. The streets were all paved with planks. Timisoara was a beautiful city, well organized and fortified[55].

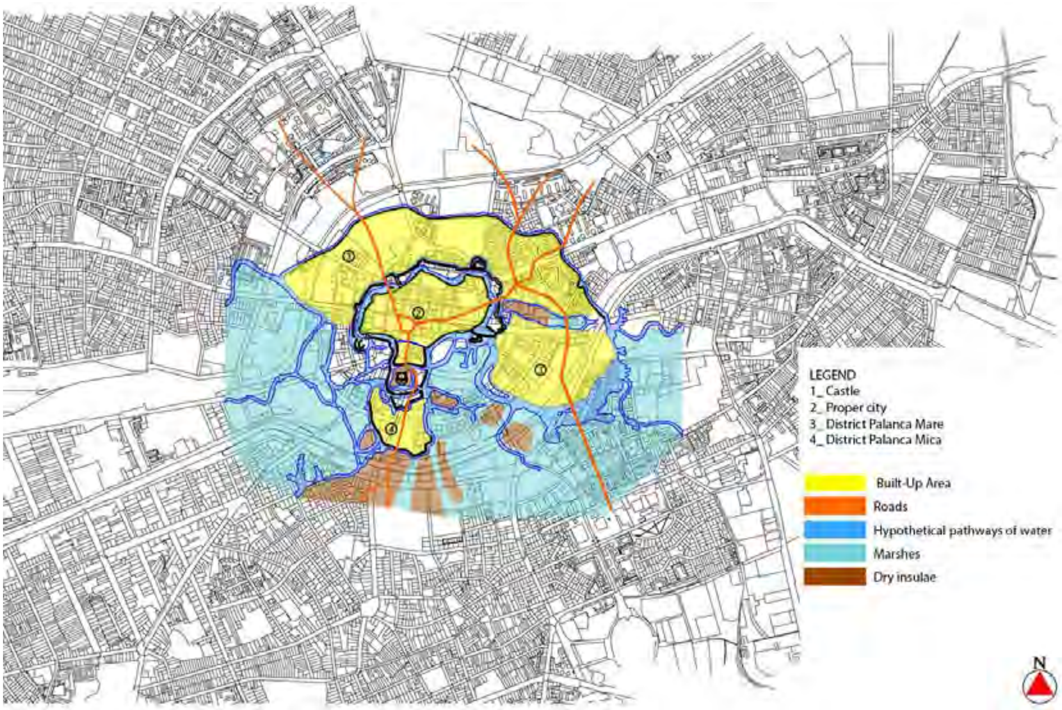

Fig. 1.6.3: Planimetric evolution of urban structure during XVI and XVII century.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 3)

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[54] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 3)

[55] *Istoria Timişoare* (Munteanu and Leşcu)

1.6.4 First half of XVIII century – Habsburg Empire

After the peace of 1718, Timisoara became the capital city of Banat, under Habsburg domination. Prince Eugene of Savoy thought that Timisoara needed to be transformed in a modern city and considering this aim, Count Mercy on 12 July 1717 submitted to the Aulic Chamber in Vienna a "Project of organization in Banat".

The project was approved and they created a “organizing committee of the Country Banat" which will operate under the leadership of Count Mercy.

Count Mercy had a strong impact on the organization and future development of the city; he created a large administration and imposed a new mentality for the city life. Besides repairing walls and buildings partially destroyed during the siege, he issued the “Building Regulation” for the town, ordering the demolition of all the existing buildings, their reconstruction obligatorily in bricks and their organization within a new rectangular street network. The continuous front of the buildings to the streets constitutes rectangular sections called “squares. Usually a “square” is occupied by an aggregate, but it can be also free of buildings and became a proper square: Union Square (old Dome Square) and Liberty Square (old Parade Square) occupied respectively two and one block of this net, focusing in them the city life. The buildings that overlook the first square were a church, the government palace and some private houses, while in the second one the buildings were reserved for military purposes. Besides representative buildings were built in the city: the Dome of the Roman Catholic and Episcopal Palace, the Orthodox Church in Union Square, the City Hall, and the Barrack Transylvania. After the new plan of 1723 the works focused on the development of the Fortress and on draining marshes. In the following forty years this improvement brings to the construction of several bridges, many public buildings and to the water network betterment [56].

The existing fortress no longer corresponded to the new technologies of war so they built a new system of defense from 1723 to 1765. The new area that had to be included within the walls was twice larger than the one of the medieval city. Small fragments of the fortifications are still visible today in Timisoara. The

[56] *Timisoara 2020 overall vision: a case study* (Tadi, 2007)

organization of the buildings, organized in perpendicular streets, reflected a military reason. The lack of space inside the fortifications, caused the displacement of the functions of productions out to the new district of Fabric, that raised at east of the actual Dacilor Street. in the north and in the east of the city new districts were born, abandoned few years later. Meanwhile, the regularization of Bega and Timis course and the draining of the marshes were changing the city's image[57].

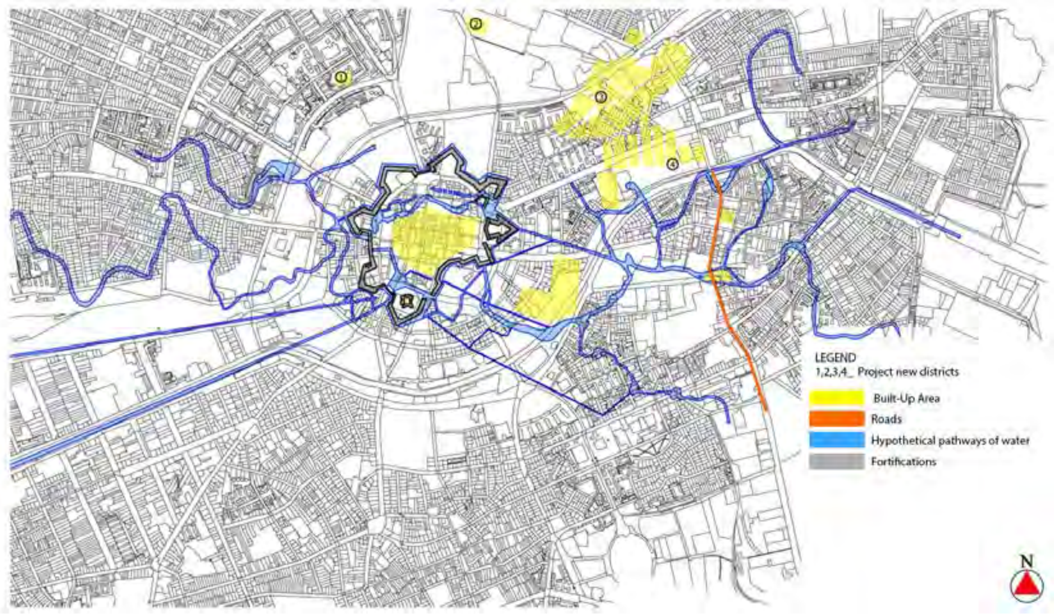

Fig. 1.6.4: Plan of Timisoara in 1734.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 9)

1.6.5 City in 1750 - Habsburg Empire[58]

The surface of the fortified city was not enough to accommodate all the functions the city needed, so new suburbs were designed and constructed starting from 1744:

[57] *Istoria Timisoarei* (Enciclopedia Romaniei, 2014);
Istoria Timişoare (Munteanu and Leşcu);
Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 9).

[58] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 11)

- the suburb of “Fabric Rascian”, for Orthodox, Serbs and Romanian residents;
- the suburb of “Fabric German”, for catholic residents, with residential, commercial and craft functions;
- the suburb of “Maierele Germane”, for German residents and having residential and agricultural functions;
- the suburb of Mihala, for Orthodox residents and having residential and agricultural functions too.

In the actual district of Elisabetin there were just four scattered constructions, called after 1750 “Mahiarele Vechi”.

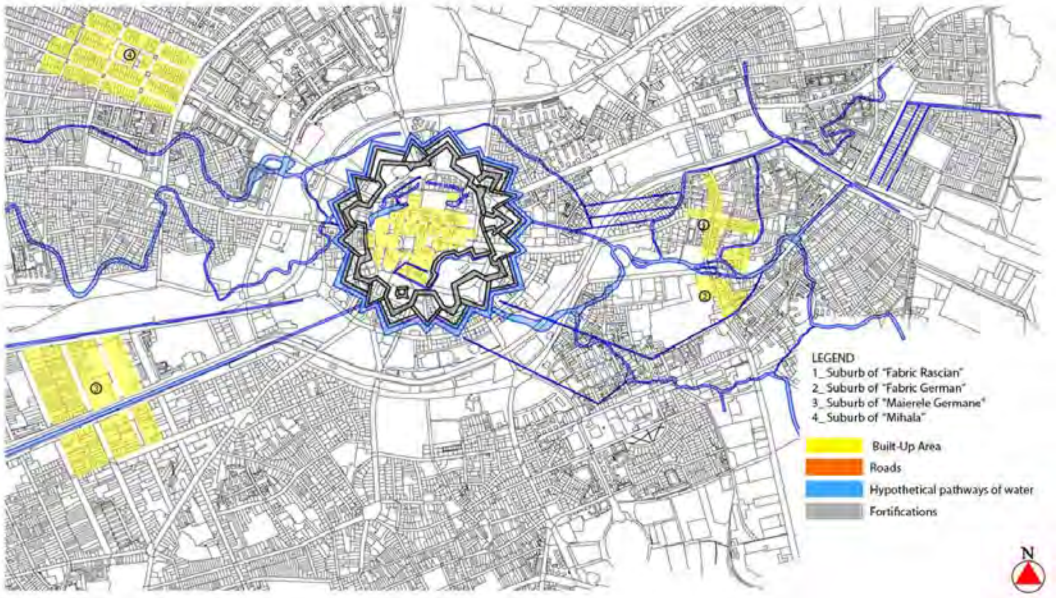

Fig. 1.6.5: Plan of Timisoara in 1750.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 11)

1.6.6 Second half of XVIII century - Habsburg Empire[59]

Between 1778 and 1779 Banat had been divided into three committees, incorporated into the administrative Kingdom of Hungary, excepted for the south of the region, still under Vienna control. Timisoara was no more the capital of a vilayet, it became the headquarter of the Committee of Timis but kept its

[59] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 16)

autonomy thanks to the status of royal town. During the second half of XVIII century, the suburbs developed:

- in 1780-1781 the districts of Rascian and German Fabric united and they expanded to east;
- in 1773, in occasion of Emperor Joseph II's visit, "Maiere Germane" was called Iodefin;
- the district of Mehala expanded to north-east;
- a new district raised around Maiere Vechi, inhabited by Romanians in the east and by Germans in the west.

At the end of the eighteenth century, Timisoara is considered one of the most beautiful and clean cities in Europe. The need to ensure good living conditions led to the reorganization of all the villages in the Banat, and the construction of new ones.

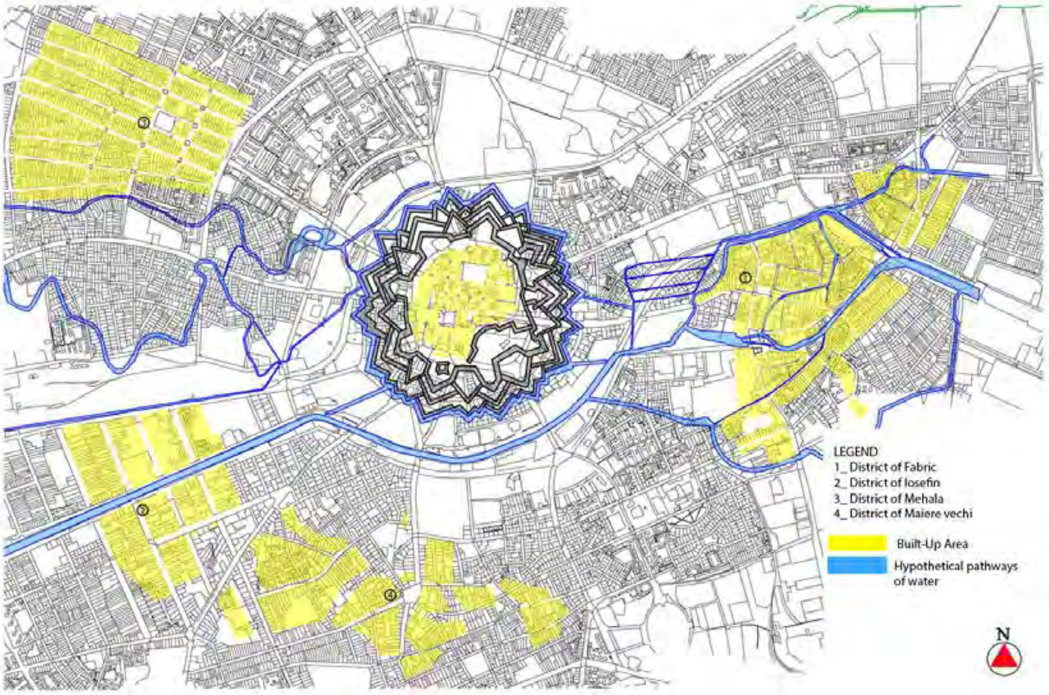

Fig. 1.6.6: Plan of Timisoara in 1784.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 16)

1.6.7 First half of XIX century – Voievodina Sarbeasca si Banatul Timisan[60]

In 1884 Timisoara, with the Committee of Timis, became part of the Austrian Empire. Between 1780 and 1848, the urban structure did not have such a large development, particularly the central district, which was limited by the fortifications around it. The district of Fabric extended to east and north of the two Bega canals, while the districts of Maierele and Iosefin united and the district of Mehala expanded to west and southwest. With the Revolution of 1848-1849, 139 buildings were destroyed by bombing. During the "Voievodina Sarbeasca si Banatul Timisan" in Timisoara many manufactures grew up, like brewery, soap manufacturing, carpet manufacturing and agriculture and the city became the place of some administrative centers, like a branch of the Austrian National Bank and the Chamber of Commerce and Industry. Many important technical innovations were introduced: on 1853 the telegraph line connected Timisoara to Budapest, in 1857 the first gas street lighting of Romania shone and Timisoara was connected to the railway network of the empire, finally in 1858 Timisoara is linked through railways to Danube port.

[60] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 18);

Evolutia istorica a aorasului (E-Patrimonium Timiensis, 2005).

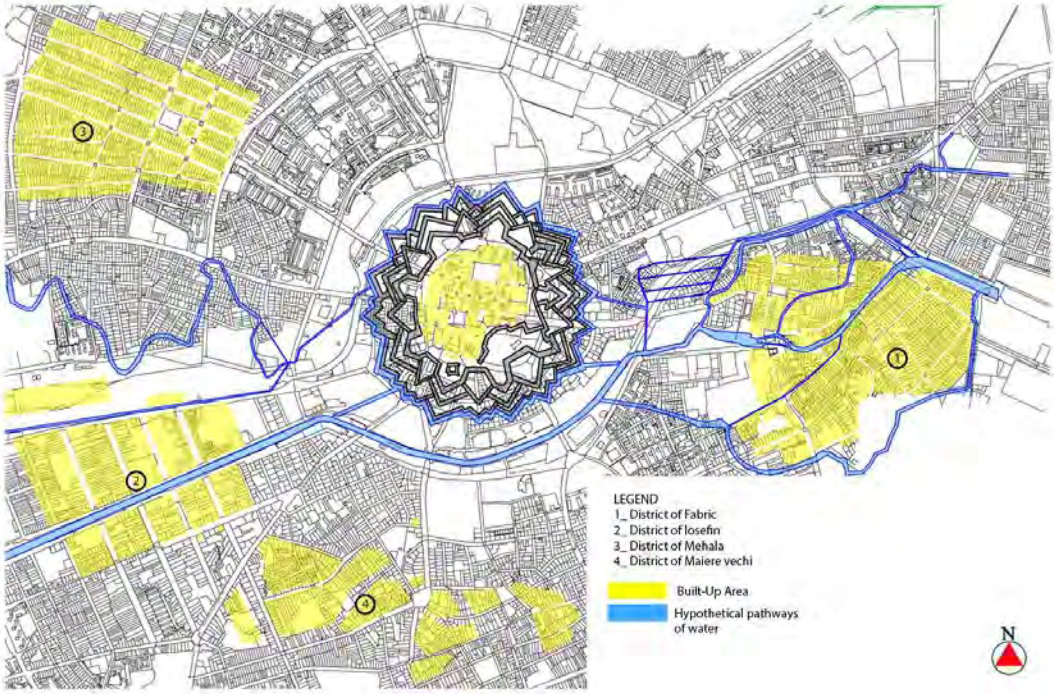

Fig. 1.6.7: Plan of Timisoara in 1853.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 18)

1.6.8 End of XIX century and beginning of XX century – Kingdom of Hungary[61]

In 1866, after the war against Prussia, the Austrian Empire split up in Austria and in Hungarian Kingdoms, and Timisoara was included in the second one. In 1868 the esplanade around the fortifications was reduced from 948 m to 569 m and the districts of Fabric and Iosefin approached this new limit. Timisoara lost its military importance and needed space for expansion, so from 1899 to 1910 the Council decided to demolish the old city gates and the fortification walls.From 1893 there were many projects and proposals aimed at the reunification of the urban setting and in 1899 the first electric tram of Romania crossed Timisoara. From 1904 to 1913 the districts of Iosefin and Fabric were connected to Cetate area through two boulevards, respectively Blvd 3 August 1919 and Blvd 16 December 1989, where monumental buildings were constructed. Although there

[61] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 20);

was a great growth in that period, large areas around the central quarter remained free and undeveloped.

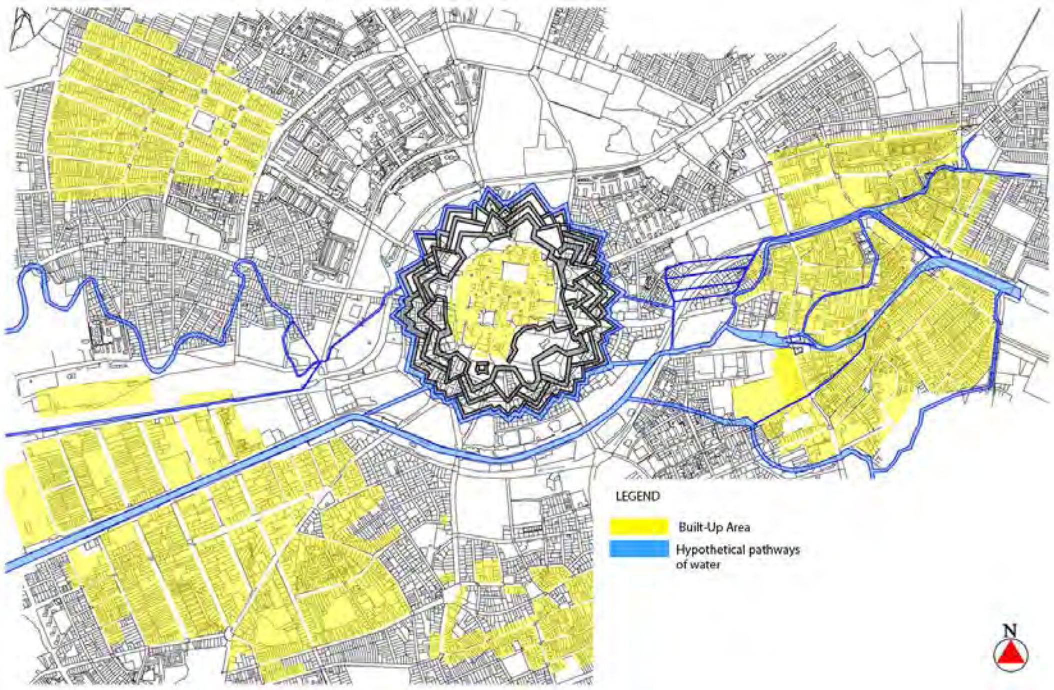

Fig. 1.6.8: Plan of Timisoara in 1876.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 20)

After the World War I the Austro-Hungarian Kingdom collapsed and Banat was incorporated in Romanian Kingdom. The architecture of that period was similar to the previous one, but the construction of a lot of villas in the free space between the districts conferred to Timisoara the aspect of a "Garden City". New boulevards were constructed, like Blvd of 1989 Revolution, Blvd Take Ionescu or Blvd Bogdanestilor[62].

[62] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 28)

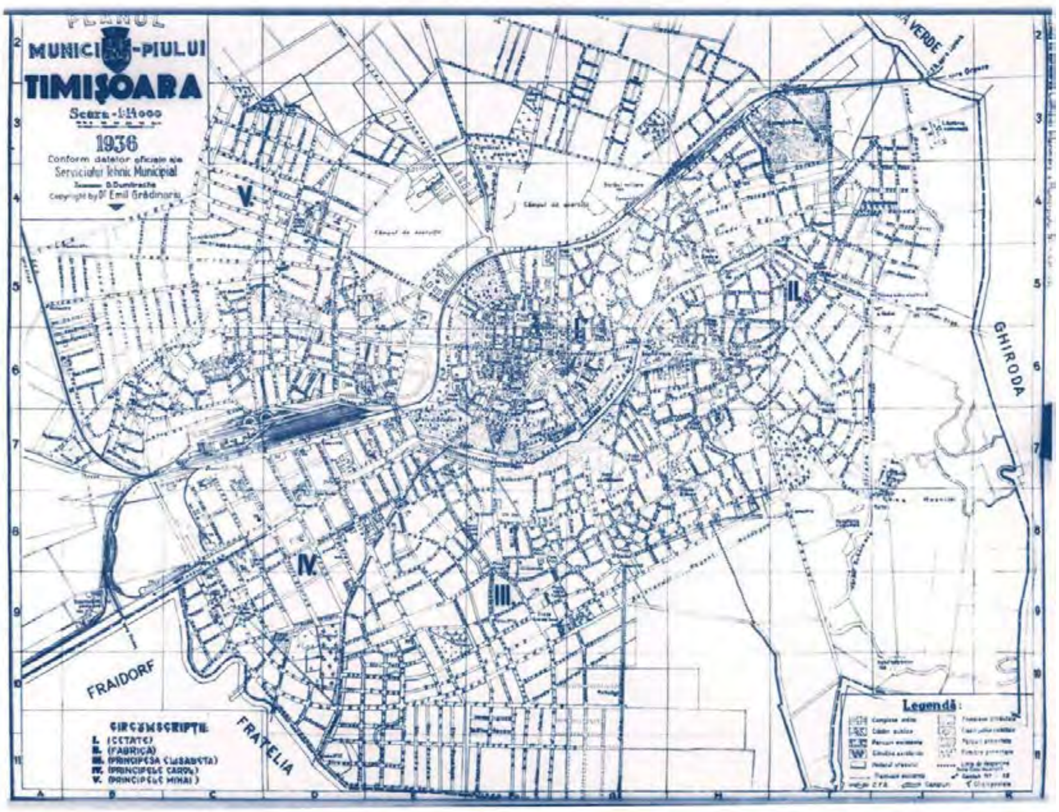

Fig. 1.6.9: Plan of Timisoara in 1936.

REFERENCE: Planuri prezentând evoluţia timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 28)

1.6.9 Second half of XX century – Soviet Period[63]

After the occupation by the Soviet troops in 1944 the Communist Regime was established in Romania. The construction activity was interrupted during the war; after that period it recovered slowly, mainly with the edification of residence filling the empty spaces between the ancient districts. In the first 23 years after the end of the war in the whole city were built 300 buildings. Since 1950, the central institution of Bucharest disposed plans of arrangement and in 1964 in Timisoara the "schema for the systematization" was promulgated, from the following year apartment blocks (1205 in 1965) began to be built occupying the interstitial spaces between the historic districts. in 1979 5927 flats of poor workmanship were built.

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[63] *Planuri prezentând evoluţia timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 30)

Marginally to the residential zone, a large industrial area stretched between Buzias way, the west area of the railway station and Sagul way. The Communist Party of Bucharest controlled urban development through a paper guide and in 1986-1989 it proposed the demolition of the historical buildings of the previous historical ages and replace them with residential "socialist" blocks.

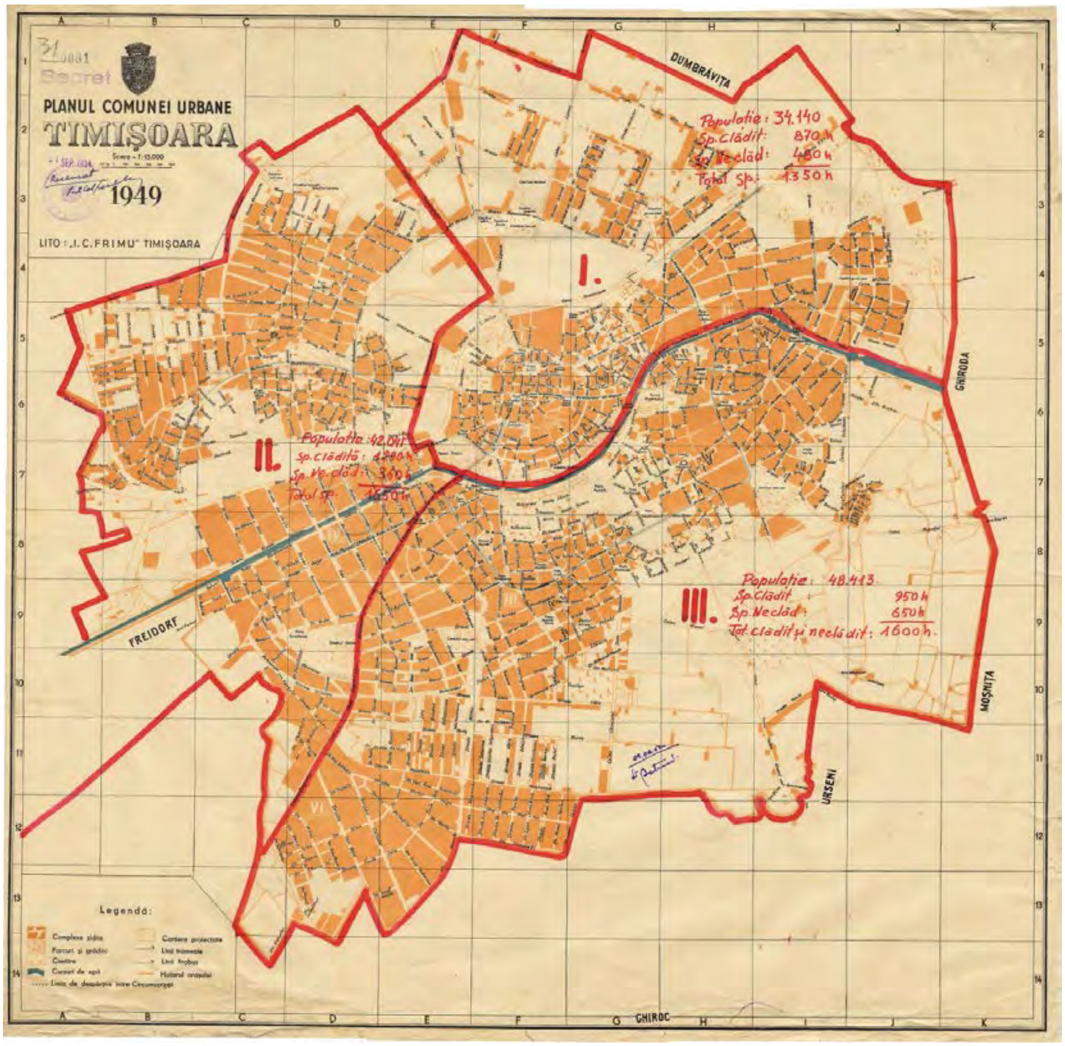

Fig. 1.6.9: Planul comunei urbane Timisoara in 1936.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.
(Primaria Municipiului Timişoara, Plansa 29)

1.6.10 End of XX century until today[64]

With the defeat of Ceausescu in 1989 Revolution grew up a new govern based on parliamentary democracy that reintroduced the concept of an urban development based on citizens requests. Economy influenced urban evolution. The positive aspect was the introduction of new commercial units in both historical and new areas. The negative aspect was the not authorized changes to historical buildings that leaded to the their degradation.

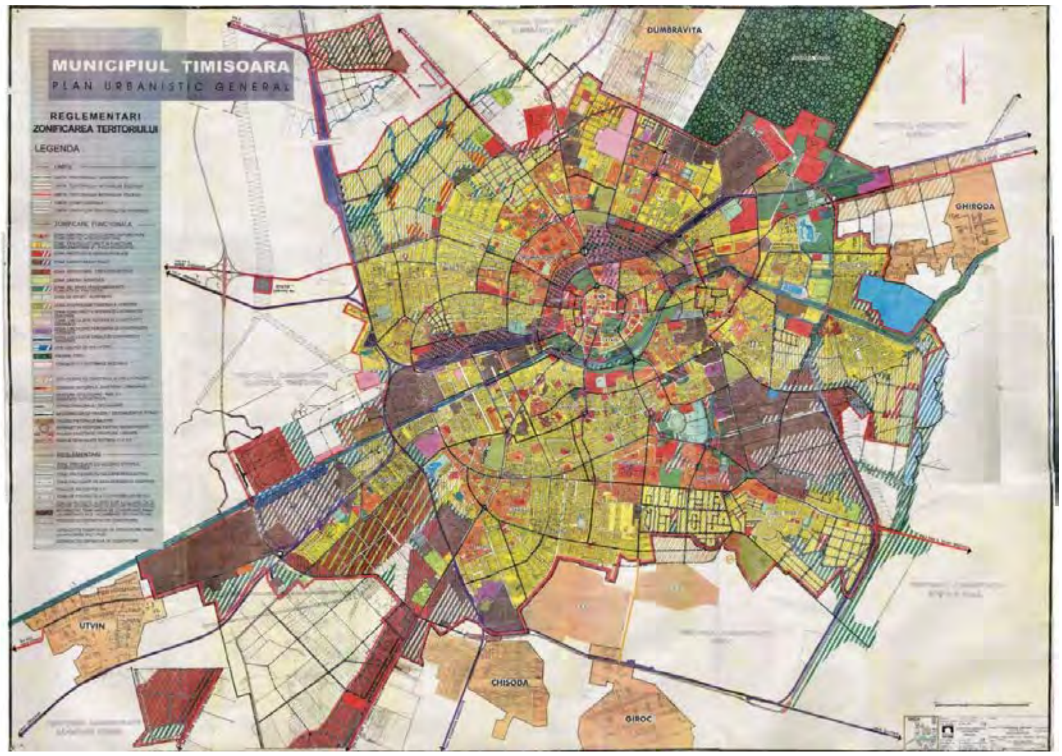

Fig. 1.6.10: Municipiul Timisoara plan urbanistic general in 1998.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 38)

The following image shows the evolution of the districts from 1750 until today, strongly influenced in their development by the fortified walls of Cetate and by the distance from the original suburbs. The regulation of Bega river and the drainage of the marshes led the satellites suburbs to expand toward the citadel. The demolition of the fortifications brought to the progressive union of Cetate

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[64] *Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent.* (Primaria Municipiului Timişoara, Plansa 38)

with the districts of Fabric, Iosefin and Mehala, with the filling of the spaces between them.

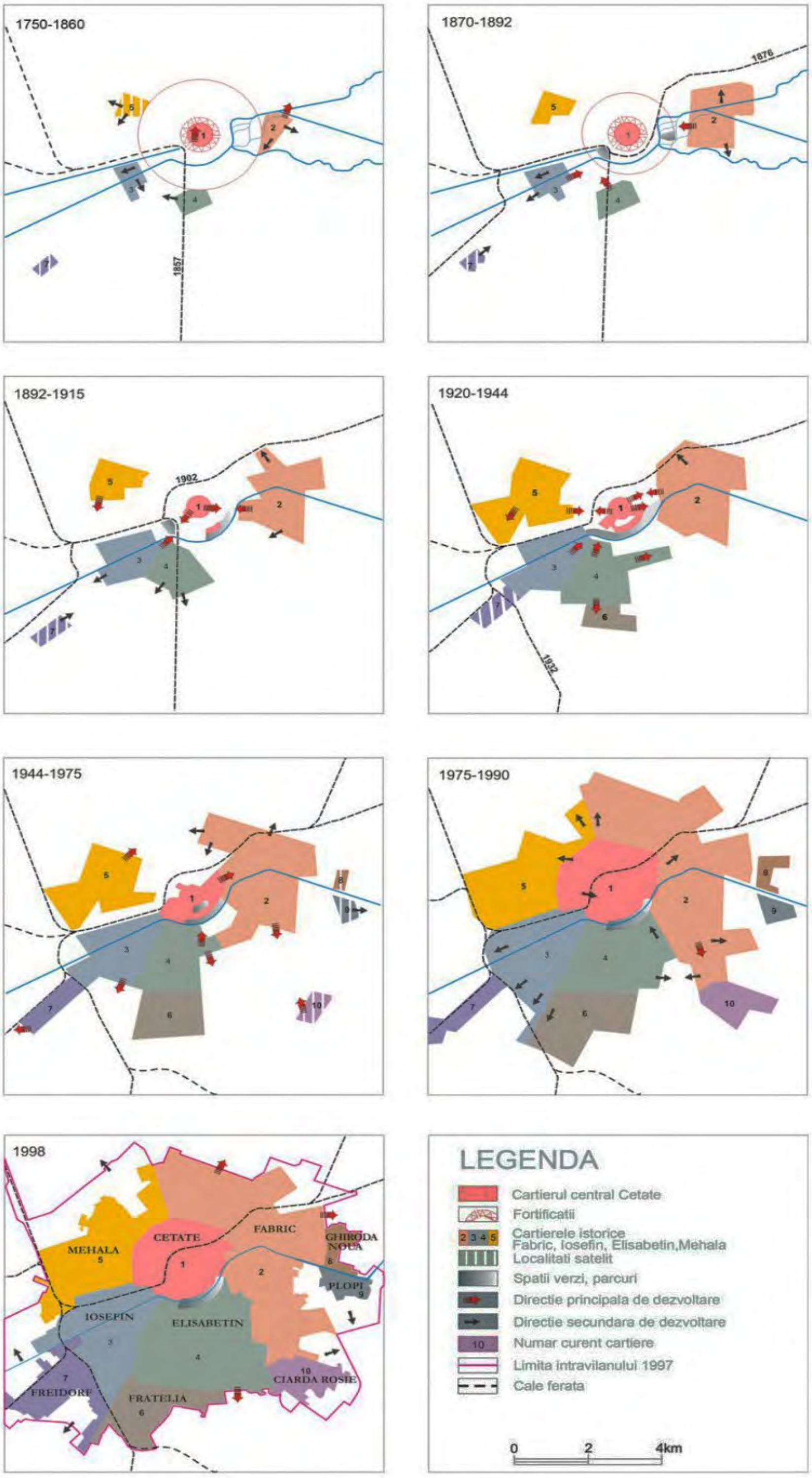


Fig. 1.6.10: Evolutionary scheme of Timisoara from 1750 to 1998.

REFERENCE: Planuri prezentând evoluția timişoarei din secolul al xii-lea până în prezent. (Primaria Municipiului Timişoara, Plansa 40)

1.7 BLOCKS IDENTIFICATION

The majority of analyzed blocks are collocated in the historical city center, in the district of Cetate, while few of them are located in the district of Iosefin, at south-west of the city center.

Fig. 1.7.1: Localization of analyzed blocks in Cetate and Iosefin districts

REFERENCE: (Google Earth 2014)

The center district occupies an area of 39 hectars. It was in past delimitated by the Habsburg fortification and the square implant of XVIII century still rules the area organization. Now it is delimitated from Gheorghe Dima Street at west, from Oituz Street at north and from Ion C. Bratiuanu Street at south-east. It is organized in 36 blocks, that usually coincide with the Habsburg square, for a total of about two-hundred buildings. A block can be composed by one building, like blocks C2, C13, C19, C29 or C32, by more isolated buildings, like blocks C20 or C28, or by an aggregate. Some blocks can contain churches or RC buildings that are not included in the present analysis, but they are numerated as structural unit as well (for example block C8). The district is a very active area of the city and almost all the buildings are occupied and used as residences, offices, shops or public services.

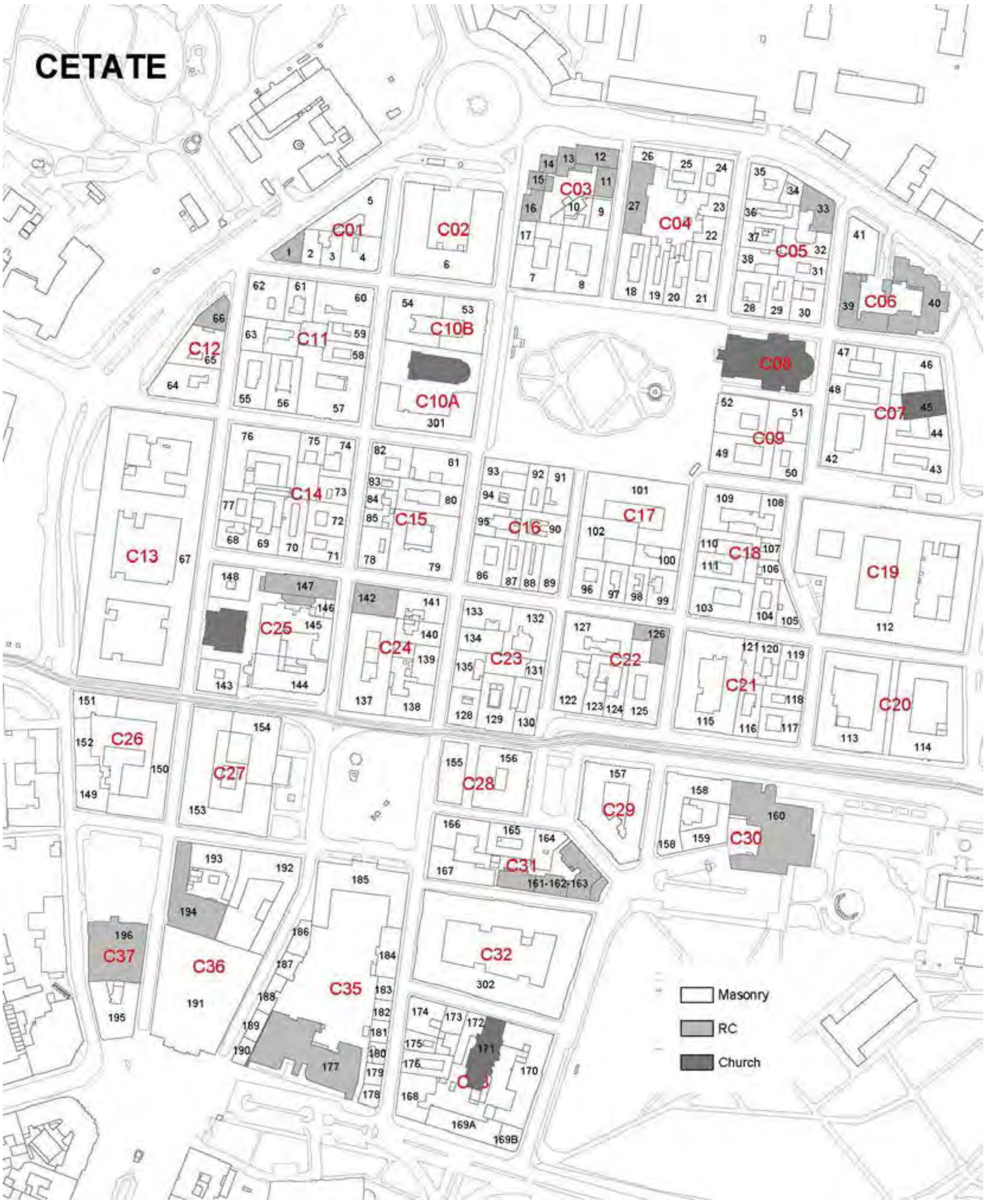


Fig. 1.7.2: Blocks and structural units identification in the district of Cetate

The objects of this thesis are masonry buildings organized in aggregates, so blocks composed by one building are not considered in the vulnerability study. This isolated buildings, usually characterized by big plan dimensions and a particular structural typology, are called “monumental blocks”. They do not constitute an aggregate and their typological characteristics are different from the common buildings of the city center. They are:

- block C2: composed by a singular building of C shape, is constituted by an abandoned military barrack;
- block C13: the block occupies an area of two squares; one building hosts the military hospital of the city, organized around three squared internal courts;
- block C19: composed by one building, organized around two rectangular and one squared internal courts, which houses the law court;
- block C20: composed by two rectangular buildings with an internal court, it was occupied in the past by the city prisons ;nowadays the block has many shops and offices;
- block C29: composed by one building with an internal court; it is prevalently a residential structure;
- block C32: one building with an internal court occupies the entire block, shops and restaurants are located on the ground floor and there are residential spaces on the upper floors.

The district of Iosefin is located in the south-western part of the city center, on the other side of Bega river. It is a residential district, but there are some shops and restaurants in the area. In this part of the city the analyzed buildings are 45, organized in 4 blocks: I41, I42, I43 and I44. All the analyzed units are masonry buildings, except for US 214, and they are all organized in aggregate.

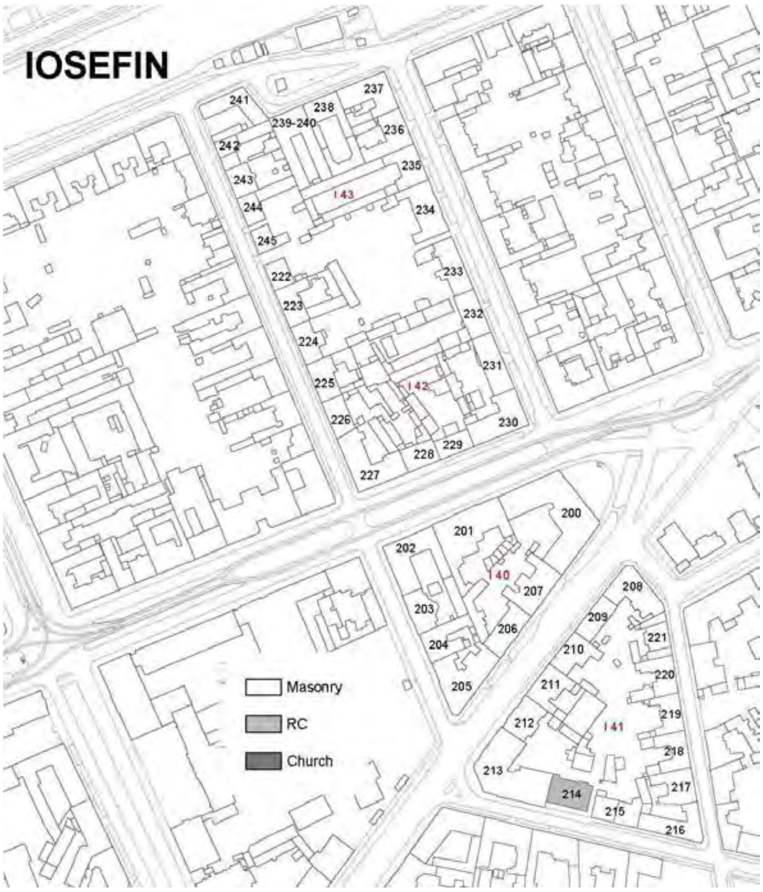


Fig. 1.7.3: Blocks and structural units identification in the district of Iosefin

1.8 TYPOLOGICAL, ARCHITECTURAL AND URBAN CHARACTERISTICS

1.8.1 District of Cetate

The blocks are regulated by a grid of streets perpendicular to each other. The streets are rotated of 7° to east-west horizontal. Usually blocks have a rectangular shape and their dimensions may range from 50 to 100 m, but if they are located near the center delimitation they can have a triangular or polygonal shape. The distance between buildings is usually about 10 m, but can reach 30 m along the street tramway, that cuts the center in two parts: the northern one, is composed by 25 blocks. This part is characterized by the major square Piata Unirii. The southern part is composed by 11 blocks and there is an other importan square called Piata Libertatii. The aggregates follow the rigid grid of streets, forming a continuous façade throughout all the block sides, and they are characterized by a strong presence of internal courtyard. These courtyards can be formed by a singular building, developed around the internal empty space, or by the assembling of more buildings around a common space. In the second case it is possible to find very high masonry walls between internal courtyards, sometimes as high as the building itself, dividing different properties. Five building shapes are recognizable in plan:

- the “O shape”, in which the building is organized around one internal courtyard in continuity for all its sides;
- the “C shape”, that is similar to the “O shape” but one side, or part of it, is missing and so there is no continuity for all the building;
- the “A shape”, that interests the buildings located in the vertex of a triangular block;
- the “L shape”, that contains building composed by two corps perpendicular to each other;
- the “H shape”, that contains rectangular buildings.

The biggest buildings, in particular “monumental buildings”, have a O or C shape, while the smaller ones have usually a rectangular shape.

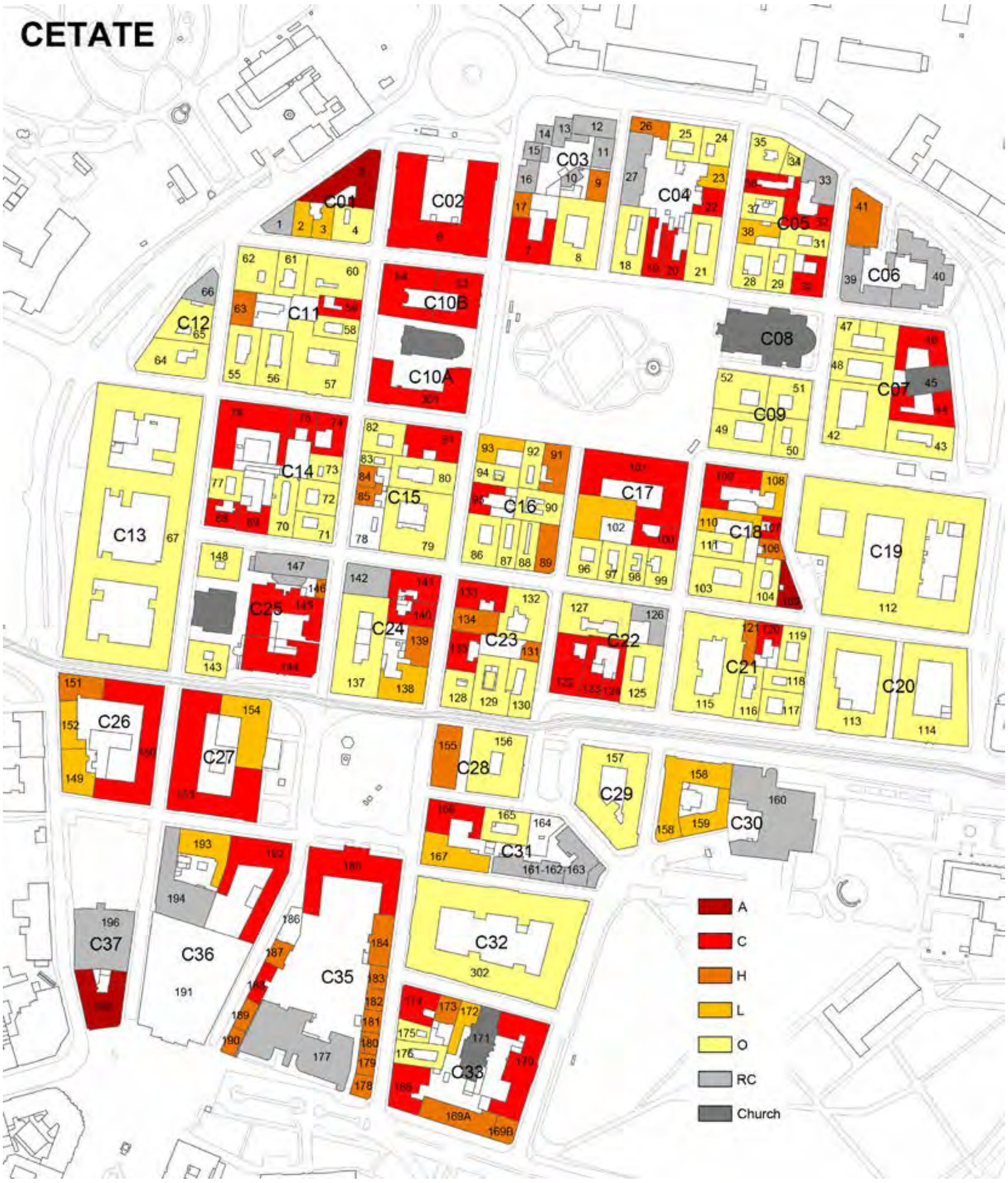


Fig. 1.8.1: Plan shape of Cetate buildings

The oldest buildings in town date back to the first half of XVIII century, when Count Mercy imposed the reconstruction in bricks of all the city center constructions, following the new grid of streets. Buildings dated in the first half of the century occupy the inner part of the center, while the constructions dated between the half of XVIII century and the half of XIX century occupy more external spaces, until the old fortifications. During the Revolution of 1848-1849 many buildings where partially destroyed by bombing, so between the end of the XIX century and the beginning of the XX century these constructions were rebuilt, often keeping the old structure until the ground floor and intervening on the upper floors. RC buildings are the only constructions in this area dated after 1945 and they are usually placed at the extremes of the center area. [65]

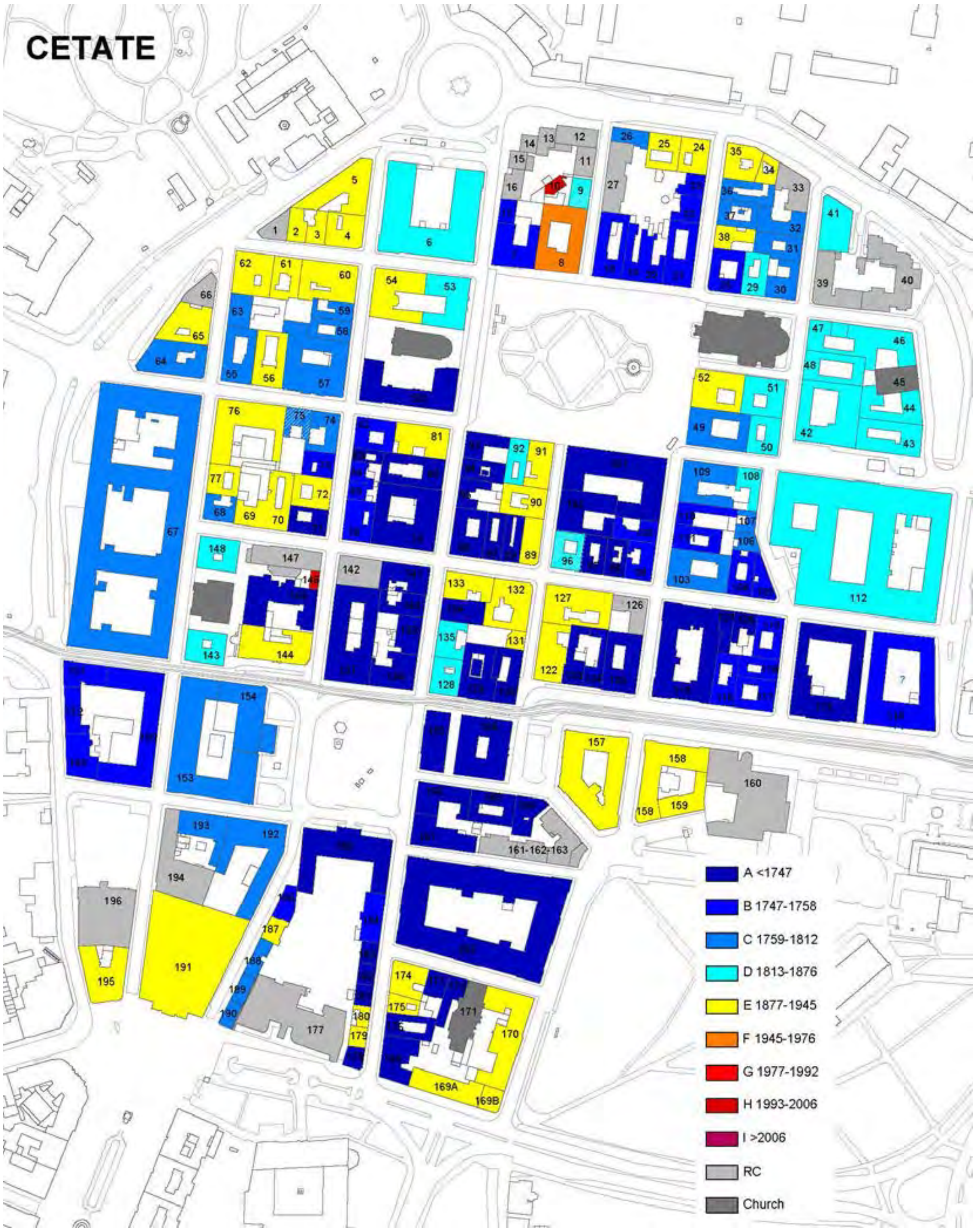


Fig. 1.8.2: Masonry buildings age in Cetate

Masonry buildings in the city center are usually from 2 to 4 stories high, but it is possible to find few cases of 1 or 5 stories. There are the basement or the

[65] *Cartierul "Cetatea Timişoarei", Index alfabetic al străzilor..* (Primaria Municipiului Timişoara)

underground floor in almost the totality of buildings, while a practicable attic is often the result of recent interventions on wooden roofs. The major part of the buildings are regular in elevation and a good part of them are regular also in plan. Elevation irregularity is often caused by later intervention on upper floors or by complex roof shapes.

The prevalent vertical structure is composed by solid brick masonry and walls thickness usually decrements from the underground to the upper floors. It can oscillate from 90 or 105 cm of basement to 45 or 60 cm of upper floors, with steps of 15 cm. Just few buildings, usually built around 1920-1930, have a bearing structure of metal reticular columns on the ground floor and masonry walls on the upper floors. The vertical connection between perpendicular walls is usually bad and two adjacent structural units can share the same common wall or can have two approached but separate walls. Other vertical structure in the area are metal column and local interventions in RC.

The horizontal structures are not the same for each floor, so in the same building it is possible to find even three different type of horizontal structure. The basements and the underground floors are characterized by brick vaults and just in few cases by iron beams and little brick vaults. On the ground floor it is possible to find brick vaults, if the building dates back to the XVIII century, iron beams and little brick vaults, if the building is dated in the second part of XIX century, or iron beams and very low brick vaults if the building dates back to the beginning of XX century. The last type of horizontal structure is often combined with masonry pillars or metal reticular column on ground floor, creating large windows and high ceilings on this floor. Almost the totality of upper floors and roofs have wooden structure, but a recurrent intervention on historical buildings in the latest years is the substitution of timber horizontal structure with concrete slab and roof reconstruction with a mixed wood and concrete structure. In this case the Romanian normative orders the inserting of a curb on the top of the bearing walls. In the same building it is possible to find a combination of both rigid and deformable horizontal structures, well or badly connected with the vertical structure.

Other elements diffused in the district are arcade and loggias, usually around internal courtyards, and external stairs and added corps, typically in the back of the buildings. In some constructions it is possible to find also overhanging elements, both on the main façade or on the back of the building, and they are built not with regular solid bricks, but with largely holed bricks. Towering and standing out elements are very common too, particularly around major squares, where frontons were added later on the top of buildings. A very characteristic element, due to the plan shape of building, is the passage to the internal courtyard. This can be about 3 or 4 m large and vaulted, or just 1 or 2 m large but two stories high, but in both cases it is usually in strict relation with the stairs system.
The façade has usually a big number of windows and openings, so the open surface usually exceeds the 30% of the total surface, and this is the reason why soft story due to many and/or of large dimension holes is present in almost the totality of buildings.


Fig. 1.8.3: Arcades and loggias(US 42 and US 77)

Fig. 1.8.4: Overhanging elements and holed bricks(US 5)

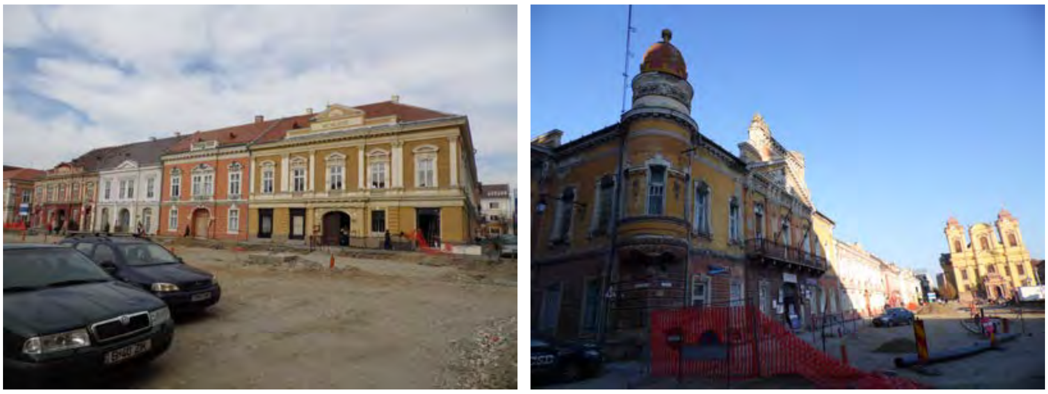


Fig. 1.8.5: Towering elements and frontons (Blocks C4 and C3)

Fig. 1.8.6: Vaulted passage and double high passage (US 125 and US 3)

Fig. 1.8.7: Widely holed facades(Blocks C9 and C16)

The majority of buildings is in good condition or with widespread damage on non-structural elements, but there are anyway many buildings in bad condition or with very serious damage. In these constructions the damage of plasters, coverings, tiles and chimneys is quite spread, but also in few cases very deep cracks and evident structural subsidence interest the construction. Even if the building from the outside seems in good conditions, it is not sure that has the same characteristics inside. It is very common indeed to restore just the outer part of the façade and leave the inner parts as they were.

Fig. 1.8.8: Non-structural and structural damage (US 133 and US 71)

1.8.2 District of Iosefin

The district blocks follow a sort of grid, but not as rigid as the Cetate one. Some of them have a rectangular shape, but more often they assume a polygonal or triangular shape. A block can reach almost 250 m of side dimension, with 20 or more buildings in it. The urban texture is not as thick as the city center and even in the narrower streets buildings stay at more or less 15 m from each other, while the principal boulevard reach a width of almost 40 m. Blocks form a continuous façade in all their sides, but inside them there is more empty space then in Cetate. In this district the most common building shapes are the “C”, “L” and “H” ones, and usually constructions are developed following the limits of the property. That is the cause of the fragmented organization of the internal spaces of the block.

Fig. 1.8.9: Plan shape of Iosefin buildings

The district of Iosefin begins to grow at the end of XVIII century as an answer to the limited space of the city center, but the analyzed blocks can be dated in the second half of the XIX century.

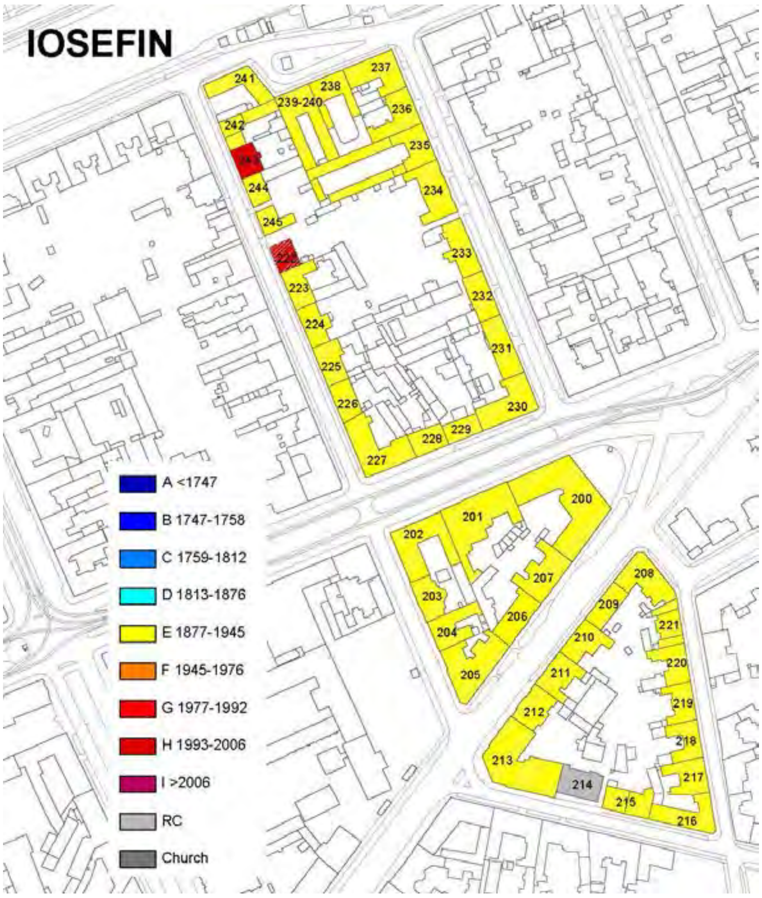


Fig. 1.8.10: Masonry buildings age in Iosefin

In the analyzed blocks, buildings are 1, 2 or 3 stories high and in the major part of them there is also a basement. They are all regular in elevation and many of them are also regular in plan.

just one of the observed buildings is in RC, the others have a solid brick masonry vertical structure and walls thickness is the same of Cetate. Due to the tricky condition of the survey in this area, the informations about the horizontal structures are less secure, but it is possible to identify the most common structures as timber or iron beams and very low brick vaults at the ground floor, brick vaults or iron beams and little brick vaults for the basement and timber for the upper floors. The roof is usually a timber structure, but as in the historical center many interventions of substitution of wooden structure with concrete structure have been made in the latest years.

Due to the internal organization of these blocks, the majority of buildings has added bodies on their back side and, as in the city center, almost all the constructions have a passage to the internal courtyard or garden. Just few buildings have towering elements and there are frontons just in the most noble constructions, like ones near the central square or along the boulevard.

Most of the buildings are in good or medium conditions, but there are some that are in bad conditions or with a medium structural damage. These units often shows deep cracks and evident deformations in the façade.

Fig. 1.8.11: Passage to the internal courtyard (US 210)

Fig. 1.8.12: Frontons(US 206)

Fig. 1.8.13: Non-structural and structural damage (US 208)

1.9 RECURRENT CONSTRUCTIVE TECHNIQUES

The constructive techniques in Timisoara were very influenced by the Habsburg presence in the territory. All the buildings built in the same period show recurring elements and a certain homogeneity, not only for the Habsburg period, but also for Hungarian period and the first part of XX century. The difference between each period is expressed principally through horizontal structures.

1.9.1 Vertical structure

The historical buildings vertical structure is solid brick masonry. Bricks dimensions are 7x15x30 cm and their length characterize the walls thickness. Two bricks arranged in length one in front to the other form a 60 cm thick wall, three bricks form a 90 cm thick wall and so on. The thickness step is 15 cm, so half brick length, and the most common wall thickness are 45 cm, 60 cm, 75 cm, 90 cm. In some undergrounds or in monumental buildings is possible to reach a thickness of 105 cm. The wall dimension can stay the same for the entire building high, but more often it decrease with stories high.

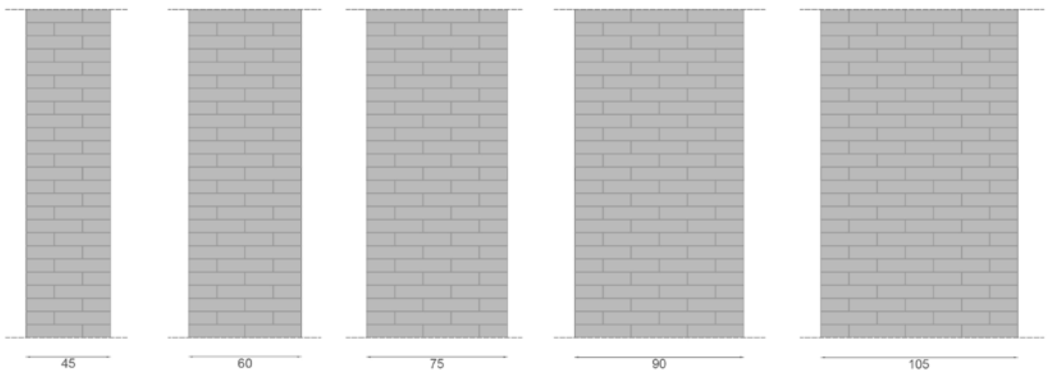


Fig. 1.9.1: Masonry sections for each wall thickness

In few cases of recent upraising it was possible to observe that the new wall has a very bad mortar distribution that does not cover the entire space between each brick. Besides, bricks are not regularly arranged and with aligned joints. Some buildings built between 1877 and 1915 have overhanging elements on the façade. These elements are made by holed bricks, having the same dimension of solid ones but on the front face they present three holes. The holes occupied more or less 1/3 of the short side and 1/7 of the long one.

In few cases it is possible to see under the plaster layer that many bricks of different dimensions are used in the same wall. It is not known the nature or the motivation of this technique, but it is evident that it creates a wall of bad mechanical characteristics.

Fig. 1.9.2: Upraising masonry (US 100 and US 85)

Fig. 1.9.3: Holed bricks and irregular masonry (US 5 and US167)

In case of raising, the new stories can be built in concrete blocks. Just in one case it is possible to observe this vertical structure under the plaster layer, and it was in a heavily reworked building, so it is not clear if this intervention is diffused. Another vertical element seen just in one building is an RC septum that interests all the building height. It is collocated parallel to the main façade and it is colligated to the concrete slab under the attic, forming an unique concrete object with a "T" shape.

Fig. 1.9.4: Concrete bricks(US 48)

A vertical element that is common to find in the city center is the metal column. There are two types of it: the first one is cylindrical and solid and it is usually collocated in the internal courtyard to support loggias or balconies, and the second one is reticular and it is the bearing structure of the entire ground floor. The reticular metal column is typical for buildings dated back between the end of the XIX century and the beginning of XX century and the principal examples are concentrated at the corner of Episcop Augustin Pacha Street and Eugeniu de Savoya Street. This vertical structure is usually combined with iron beams and very low brick vaults, creating a ground floor characterized by many and wide windows. Reticular columns are usually 60x60 cm and they distance 3 or 4 m from each other.

Fig. 1.9.5: Solid metal columns(US 8 and US 28)

1.9.2 Horizontal structures[66]

In Timisoara there are six types of horizontal structures. While ground floor and basement structures are often visible even from the outside of the building, the recognition of upper floors horizontal diaphragms is more complicated. The six typologies are:

- Timber horizontal structure: this typology usually interests upper floors, but in case of one storey building can be found also at the ground floor. There are two principal types of timber structures, one with adjoining wooden beams posed side by side, and one with distance between each beams, filled with soil and shards. In some cases the second typology was modified with new wooden beams insertion in the spaces between the original beams, to stiffen the horizontal structure. This intervention makes this typology very similar to the adjoining one. Due to this similarity we will consider the heaviest and probably the most common one, that is the adjoining beams typology.

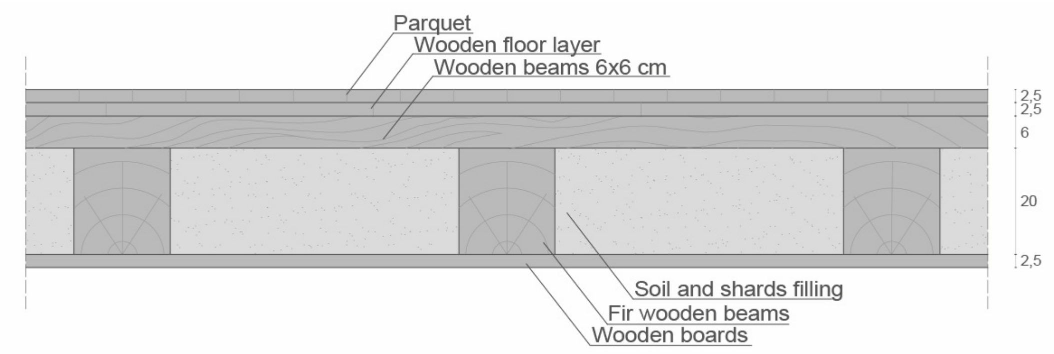


Fig. 1.9.6: Timber horizontal structure detail, distanced beams typology

[66] *Illustrated dictionary of historic load-bearing structures* (Szabo', 2005)
Verein "Der Bauconstructeur" an der k. k. technischen Hochschule in Wien (Prokop, 1899)

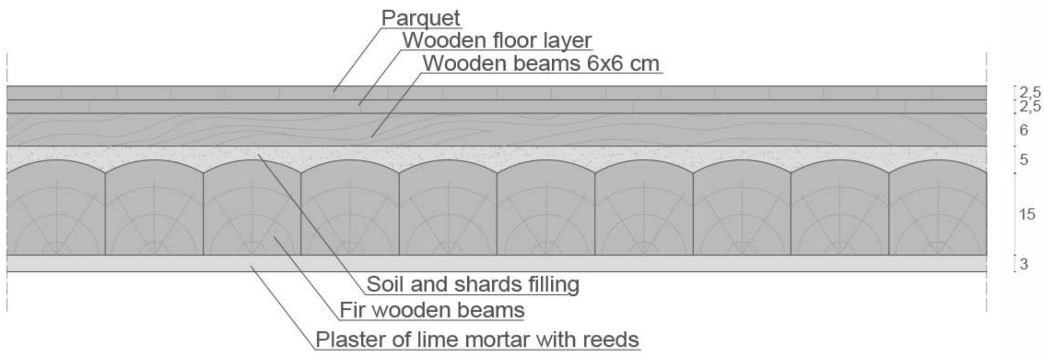


Fig. 1.9.7: Timber horizontal structure detail, adjoining typology

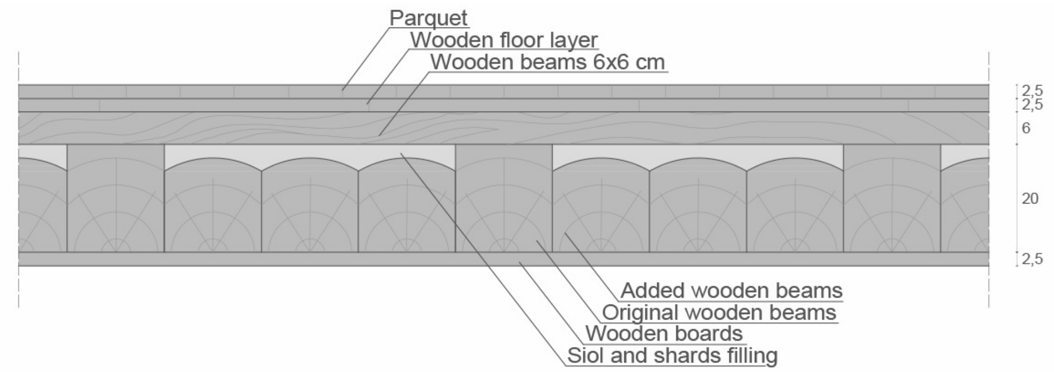


Fig. 1.9.8: Timber horizontal structure detail, distanced beams typology in which new beams were added in the spaces between old beams

Fig. 1.9.9: Timber horizontal structure examples(US 130 and US 124)

- <u>Concrete slab</u>: historical buildings originally did not have this structure, but it comes from the substitution of wooden structure in latest restoration or renovation interventions. There are not many information about layers and dimension of the structure, but it is possible to speculate about a 15 cm thickness of the concrete slab and between 5 and 10 cm of mortar bed and finishing.

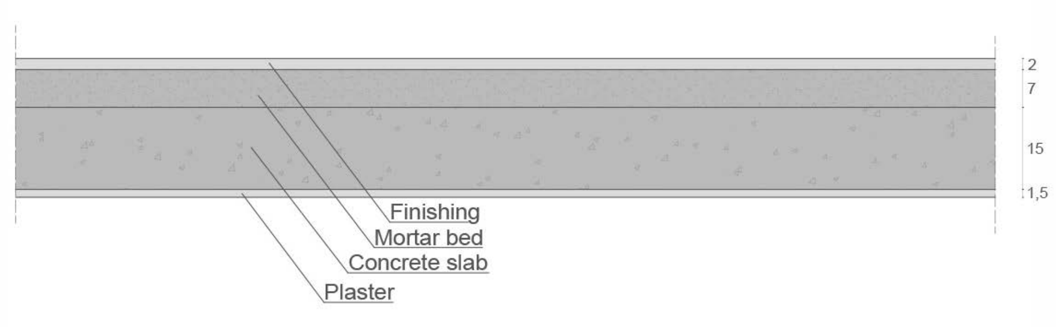


Fig. 1.9.10: Concrete slab detail

- Brick vault: this is the most common typology for basements and underground floors, but it is also very diffuse for ground floor of historical buildings from half of XVIII century. It is possible to find barrel vaults, usually for corridors or very long rooms, or groin vaults. It is possible that the same building has both vaults typology. Due to plan irregularity, vaults are usually asymmetrical. Over the brick vault there are 50-60 cm of soil and shards filling and on the top of it there are wooden boards of 6x6 cm and a wooden floor layer.

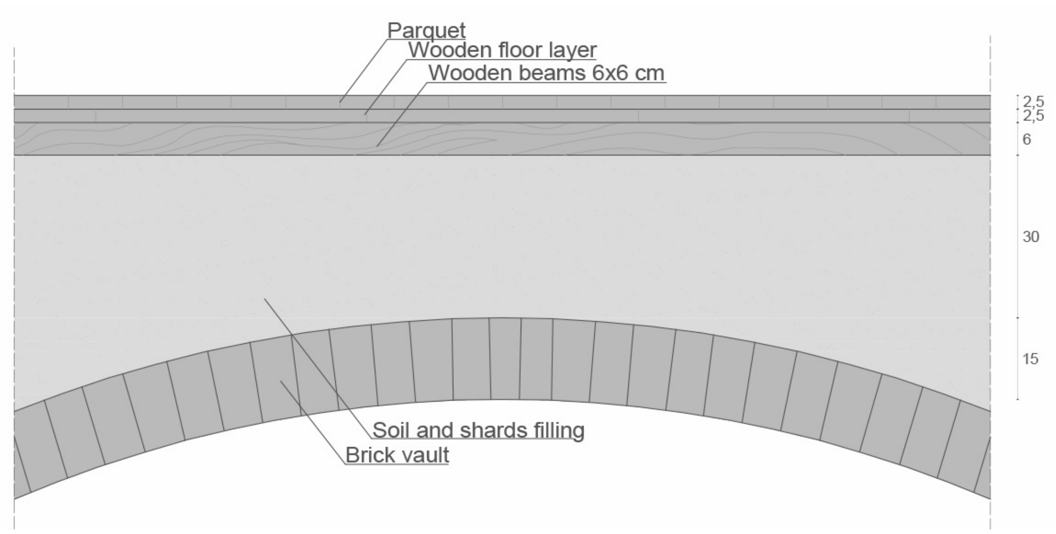


Fig. 1.9.11: Brick vault detail

Fig. 1.9.12: Brick vault example and soil and shard filling (US 192 and US 130)

- <u>Brick vaults and horizontal concrete diaphragms</u>: like concrete slab, this horizontal structure is not original, but it is a result of latest interventions. The substitution of the wooden layers with a concrete slab causes the discharge of the brick vault and the consequent stiffening of the horizontal structure. The filling is not known, but it is reasonable to suppose soil filling, similar to the previous one tipology. As in the case of the concrete slab, dimensions and layers are not certain for this kind of horizontal structure. The brick vault from below looks exactly like an original one, so the presence of this structure is supposed in relation with other horizontal structure typology and visible interventions.

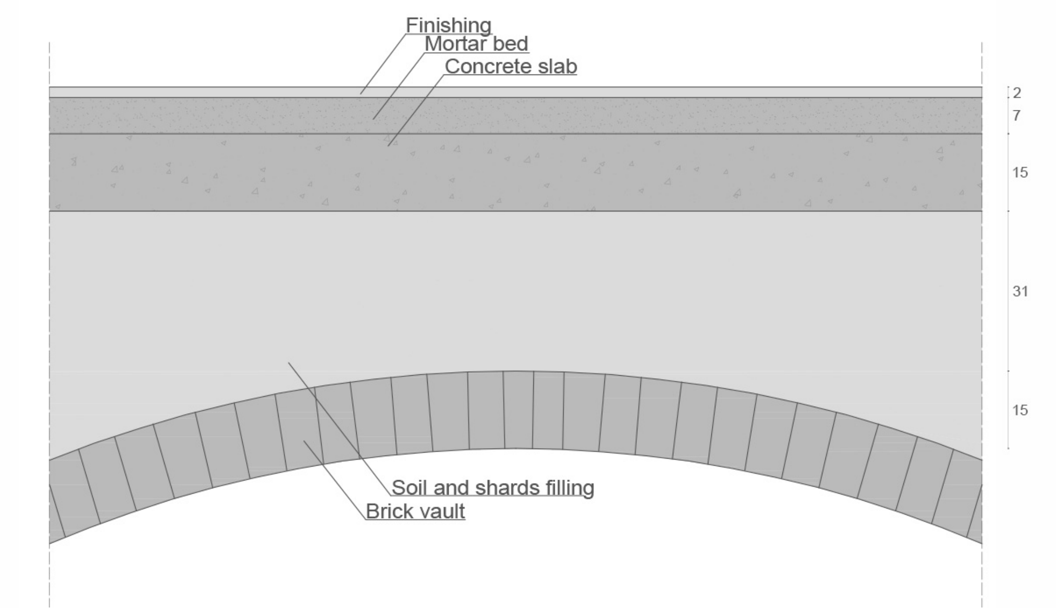


Fig. 1.9.13: Brick vault and horizontal concrete diaphragms detail

- Iron beams and little brick vaults: this horizontal structure is typical for basements and ground floors of buildings dated back to the second half of XIX century. Sometimes it is possible to find it also on the upper floors, but in very few cases. Usually the warping of this structure is perpendicular to the main façade, but it can be found also parallel to it. The exact dimension of the iron beam is not known, but it is similar to a IPE 200.

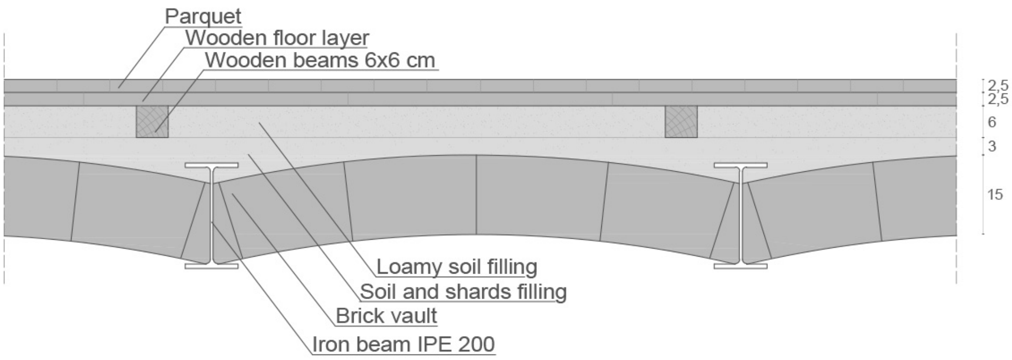


Fig. 1.9.14: Iron beams and little brick vaults detail

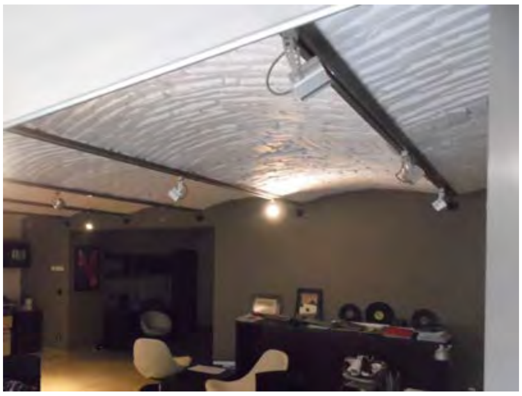



Fig. 1.9.15: Iron beams and little brick vaults examples(US 100 and US 3)

- Iron beams and very low brick vaults: the only difference between this structure and the previous one is the vaults arch. This arch is so low that from below can look almost plan. This structure is typical of buildings of the beginning of XX century, particularly combined with masonry pillars or metal reticular columns.

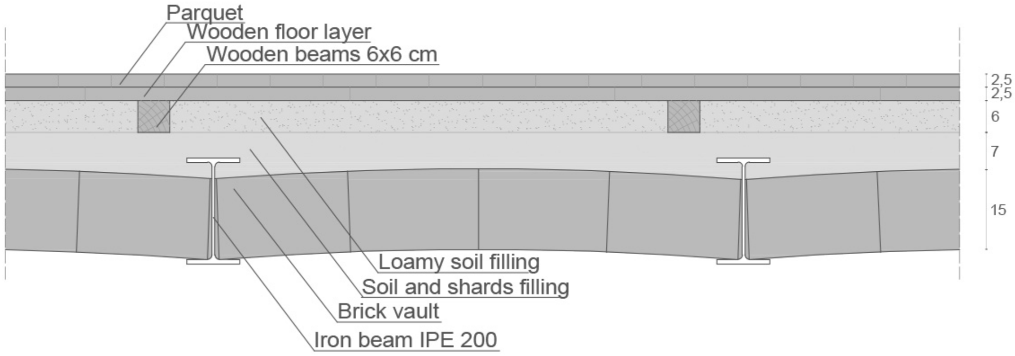


Fig. 1.9.16: Iron beams and very low brick vaults detail

for these structures, concrete slabs and brick vaults with horizontal concrete diaphragms are considered as rigid structures and well connected to vertical walls. brick vaults are considered as deformable but well connected and timber structures and iron beams with little brick vaults as deformable and badly connected.

1.9.3 Roofs

In Timisoara there are historically non–thrusting timber roofs with a slope of about 34°. There are different dispositions of wooden beams, but the attic is always impracticable. The top horizontal structure is usually independent from the roof structure and it separates the top storey from the attic. The roof covering is usually constituted by ceramic tiles posed over secondary wooden beams, disposed perpendicularly to the principal beams. The timber trusses roof has been drawn referring to the material provided by Arch. Bogdan Demetrescu.

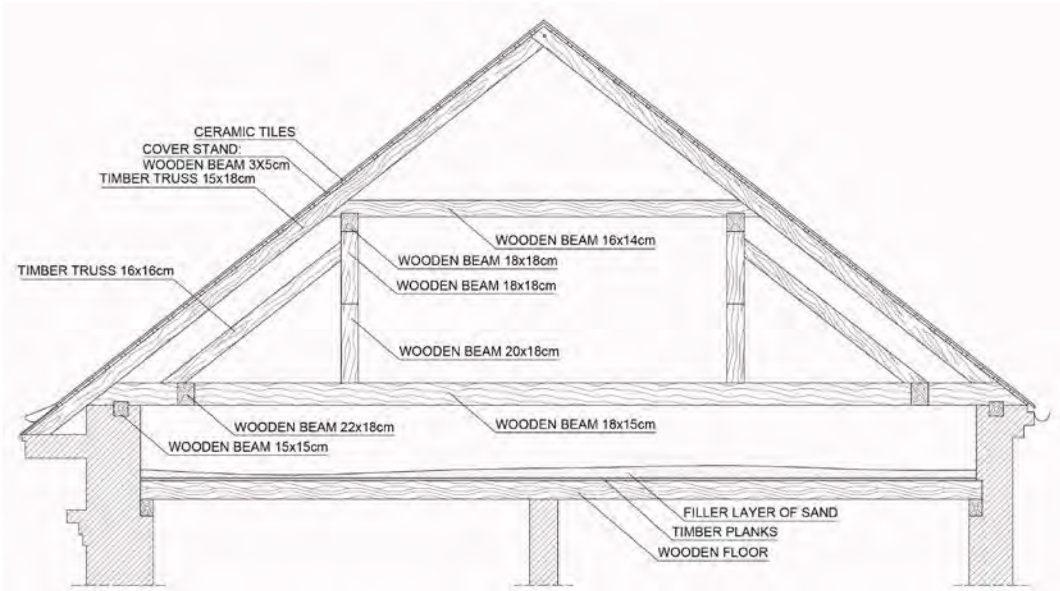


Fig. 1.9.17: Non –thrusting timber roof detail

REFERENCE: *Verein "Der Bauconstructeur" an der k. k. technischen Hochschule in Wien* (Prokop, 1899)

Fig. 1.9.18: Non –thrusting timber roof examples (US 100 and US 124)

In the latest years roof reconstruction became a common intervention on historical buildings. The substitution of the horizontal timber structure of the attic with a concrete slab makes this space fit for use, adding a floor on the building total count. The new roof has a mixed non-thrusting structure, in which the bearing function is accomplished by concrete beams while new wooden beams and ceramic tiles are used for the covering. The insertion of a concrete ring beam all over the perimeter walls is obliged by Romanian normative. This intervention increases loads over the historical masonry walls, that often are left in the original condition (no intervention). It was not possible to collect more information about this intervention except for on-site observations and experts indications, so the details of this new type of roof are hypothesized, as well as elements measures.

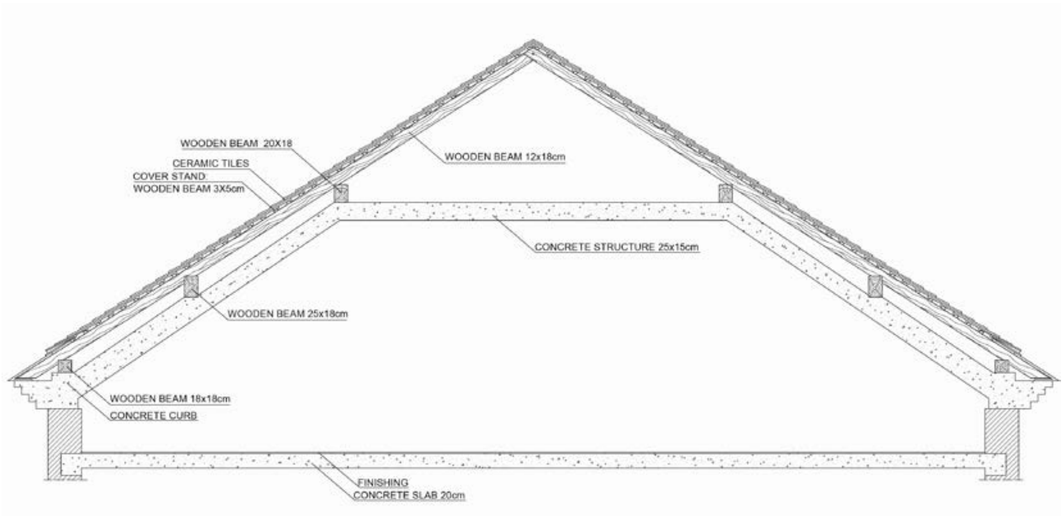


Fig. 1.9.19: Non –thrusting mixed roof detail (hypothesizes)

Fig. 1.9.20: Non –thrusting mixed roof examples (US 82)

1.10 REINFORCING ELEMENTS

Reinforcing elements are those structural elements or devices that improve the seismic response of the structure. Some of them are easily recognizable, such as buttresses and contrast elements, but some of them are hidden inside the building, like tie rods and their anchor plates or reinforced plasters. Due to the rapid and external surveys, it was difficult to recognize the second above mentioned group. Anyway, reinforcing elements in Timisoara are:

- Buttresses and/or spurs: just 3 buildings present this kind of reinforcement. They are collocated where there are not adjacent buildings to contrast the out of plan force, and they usually interest just the first storey.

Fig. 1.10.1: Spurs examples(US 194 and US 95)

- Contrast elements: the only example of this typology is a series of contrast arches between two walls of the same building. Masonry arches are collocated in correspondence of each storey, except the last one.

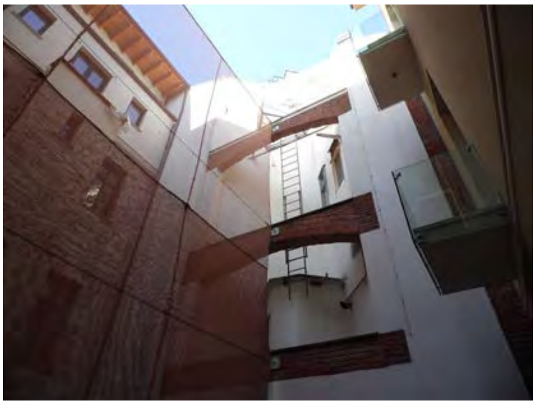


Fig. 1.10.2: Contrast elements examples(US 5)

- Tie rods: they are metallic elements that improve connections between masonry walls and horizontal structures, preventing the out of plan overturning. The presence of these elements do not assure automatically a reinforcing function, cause they can be ineffective in many ways. The wrong position or the wrong action direction are examples of their inefficiency. Tie rods in Timisoara are usually parallel to the main façade and are visible in the latest floors wall. The eventual presence of tie rods on the main façade is hidden by decorative elements and considering this condition it is not possible to tell for sure the efficiency of observed tie rods and for this reason they are considered ineffective.

Fig. 1.10.3: Tie rods examples(US 79 and US 237)

- Reinforced plaster: this intervention it is very difficult to recognize at finished work, cause the metal elements will be covered by the plaster layer. There is only one case in which the metal grid is visible, in a work in progress building.

Fig. 1.10.43: Reinforced plaster examples(US 17

2 TYPOLOGICAL ANALYSIS

The analysis carried out in the previous chapter about history, geology and the recurrent elements that characterized the center of Timisoara represent a comprehensive initial study of the blocks. These preliminary studies are a good starting point to develop the typological analysis that will be deepened in this chapter.

2.1 ON-SITE ACTIVITY

The activities in the center of Timisoara and in the district of Iosefin took place from 4th to 18th of November. This phase was essential for a global knowledge of all the blocks and for the development of vulnerability analysis. The general reconnaissance of the block, helped by historical informations and a satellite map, allow the definition of different structural units. The structural unit is usually delimitated by open spaces, structural joints or adjacent buildings with a different structural typology. In the same structural unit the flow of vertical loads must have continuity from sky to earth.[1]

Measurements has been carried out for each building, and a complete photographic survey has been made; all the data have been collected in appropriate forms.

The presence of Romanian students helped the data collection, allowing the entrance in internal courtyard and collecting more information about buildings. Despite this, just few buildings were completely visited.

This fact caused the inability to set the typology of the horizontal structures for all the structural units and sometimes the walls thickness ; due to this, all the uncertain data have been marked by two different colors: green if the information was probable but unsure and red if it was only a hypothesis.

[1] §8.7.1 *Costruzioni in muratura* (NTC 2008, p. 332)

2.2 SURVEY FORMS

The data collected during the on-site activity were included in specific forms, drawn up by the Construction Technologies Institute, an organism part of the Italian National Research Council. These forms have been adapted to the specific case study of the city of Timisoara, omitting or adding some parts. All forms has been translated from Italian to English and the parts about Italian normative and cartography has been modified referring to the available material. Average interstorey height column has been subdivided in four columns, respectively for ground floor, upper floors, attic and underground. Information about wall thickness has been introduced and it has been divided for ground floor, upper floors and attic. Holes in façade column has been divided for the principal façade and the general façade. Prevalent vertical and horizontal structures columns have been subdivided for ground floor, upper floors, attic and basement or underground. Modifications have been made also in forms legenda, adding characteristics typical of Timisoara and omitting options that were not representative of the city, in particular for vertical and horizontal structures typologies, interventions and other elements.

The form consists in a first part about geometrical-typological and vulnerability information (this part is different between masonry buildings and reinforced concrete buildings) and a second part concerning exposure and damage. For each structural unit both forms had been filled and the completed ones are in annex A. The contents of the forms and their legends will be explained in the following paragraphs.

2.2.1 Masonry buildings: geometrical-typological data and vulnerability information

This form (Fig 2.2.1, Tab 2.2.1) organizes all the general information about the single structural unit:

- General information about buildings location (street, number, other information), age and previous interventions;

- Geometrical data such as the number of stories, plan area, heights, percentage of holes in façade;
- Typological data about vertical structures, horizontal structures, roof and joints;
- Buildings regularity and vulnerability factors;
- Inspection accuracy.

Below the form[2] and the relative legend[3] are shown.

[2] *Scheda per il rilevamento speditivo degli edifici in muratura* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione, 2003)

[3] *Scheda per il rilevamento speditivo degli edifici in muratura. Istruzioni* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione, 2003)

MASONRY BUILDINGS - geometric-typological and vulnerability data			
		street	
		number	
		other informations	
	I	building number	
	M	building in the aggregate	
	C	age	
	C	interventions	
geometric data	I	total stories number	
	I	out of ground stories number	
	I	stories number on the main facade	
	I	floor area [m2]	
	I	average floor area [m2]	
	N1	average intersotey height [m]	PT
			P+
	N1	maximum interstorey height [m]	
	N1	maximum building height [m]	
	N1	minimum building height [m]	
	N1	main facade height [m]	
	N1	walls thickness [m]	
	C	holes in facade	PF
			GF
typological data	C	structural typology	
	C	prevalent vertical structures	
	C	other vertical structures	
	M	prevalent horizontal structures	
	M	other horizontal structures	
	M	roof	
	C	connection of vertical structures	
	C	connection of horizontal structures	
	C	regularity	
other regularity and vulnerability factors	M	other elements	
	M	soft-storey	
	M	reinforced elements	
	C	non-structural elements	
	C	present state	
	C	accuracy of inspection	

Type of information: C = one-alphanumeric character code; I = integer; N1 = number with one decimal digit; M = alphanumeric string with one or more characters (multiple choice)

Fig. 2.2.1 Masonry buildings: geometric typological data and vulnerability information form

REFERENCE: Scheda per il rilevamento speditivo degli edifici in muratura (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

BUILDING IN THE AGGREGATE (multiple choice)	1	isolated
	2	head
	3	corner
	4	inside
	5	backward
	6	protruding
	A	same height
	B	higher
	C	lower
	D	higher and smaller
	E	towering
	F	staggered slabs (at different levels)
AGE	A	< 1747
	B	1747-1758
	C	1759-1812
	D	1813-1876
	E	1877-1945
	F	1946-1977
	G	1978-1992
	H	1993-2006
	I	>2006
INTERVENTIONS	A	enlargement
	B	raising
	C	renewal
	D	restoration
	E	maintenance
	F	facade restoration
	G	partially demolished

	H bombed and partially reconstructed I possible demolition and reconstruction R no intervention S substitution (if the unit is replaced with an r.c. one, complete r.c. form) T work in progress
TOTAL STORIES NUMBER	Fill out the total number of stories of the building, including underground floors. Attic should be excluded unless its average height is less than 20% of the avarege interstorey height of the building.
OUT OF GROUND STORIES NUMBER	Fill out the stories number of the building, considering the lower part around it and including the attic if its average height is less than 20% of the average interstorey height of the building. A attic U underground B basement
STORIES NUMBER OF THE MAIN FACADE	Fill out the storey number on the main facade. A attic U underground B basement
FLOOR AREA	The plan area of the building can be estimated through the use of the cartography, with maximum approximation of 10%.
AVERAGE FLOOR AREA	Fill out the average gross floor area of each storey of the building.
AVERAGE INTERSTOREY HEIGHT	Fill out the average height of each storey of the building, approximated to 0,5 m. GF ground floor F+ upper stories

	FA attic FU underground
MAXIMUM INTERSTOREY HEIGHT	Fill out the maximum interstorey height measured for each storey, approximated to 0,5 m.
MAXIMUM/ MINIMUM BUILDING HEIGHT	Fill out the maximum/minimum height of the building, estimated from the eaves.
MAIN FACADE HEIGHT	Fill out the average height of the main facade, estimated from the eaves.
HOLES IN FACADE	The evaluation takes into consideration both the main façade and the average percentage for the other visible facades: PF principal façade GF other visible façades A < 10% B 10%-20% C 20%-30% D > 30%
STRUCTURAL TYPOLOGY	1 only perimeter walls 2 interior walls 3 reinforced concrete above masonry 4 masonry above r.c 5 masonry and r.c. on the same floor 6 reinforced masonry 7 confined masonry 8 masonry and Rc septum 9 masonry above the ground floor with metal reticular column 10 masonry pillars at the ground floor

VERTICAL STRUCTURES TYPOLOGY	A	Single leaf walls
	B	double leaf with inner core
	C	brick masonry(solid or multi-hole)
	D	brick masonry(holed)
	E	mixed structures
	F	hewn stone
	G	rounded stone
	H	tufa blocks, square stone
	I	concrete blocks (heavy)
	L	concrete blocks (light)
	M	RC septum
	N	metal column
	O	concrete
	P	metal reticular column
HORIZONTAL STRUCTURES TYPOLOGY	A	timber
	B	timber with tie rods
	C	metal beams with vaults or tiles
	D	metal beams with vaults or tiles and tie rods
	E	concrete and masonry or concrete slab
	F	vaults without tie rods
	G	vaults with tie rods
	H	vaults and horizontal diaphragm
	I	vaults and horizontal diaphragm with tie rods
	L	metal beams with concrete slab
	M	iron beams and brick tiles
	N	iron beams and little brick vaults
	O	iron beams and very low brick vaults
	P	metal and glass
ROOF	A	thrusting timber roof
	B	limited thrust timber roof
	C	contrasted thrust or horizontal timber beams
	D	concrete and masonry or concrete slab
	E	thrusting steel
	F	non-thrusting steel
	G	thrusting mixed roof

	H	non-thrusting mixed roof
	1	flat
	2	one flap
	3	more flaps
VERTICAL STRUCTURES CONNECTION	1	tie rods and/or tie beams for each level
	2	good connections between walls
	3	no ring beams and bad connections
	4	________________________________
HORIZONTAL STRUCTURES CONNECTION	1	rigid and well connected diaphragm
	2	deformable and well connected diaphragm
	3	rigid and poorly connected diaphragm
	4	deformable and poorly connected diaphragm
REGULARITY	A	regularity in plan and in elevation
	B	regularity in elevation
	C	regular plan
	D	none
OTHER ELEMETS	A	arcade
	B	loggia
	C	external stairs
	D	added bodies
	E	isolated pillars
	F	false walls
	G	overhanging
	H	heavy roof
	I	demolition of structural elements
	L	non-aligned holes
	L*	non-aligned horizontal diaphragms
	M	irregular strengthening
	N	overhanging and towering/ standing out elements

<table>
<tr><td></td><td colspan="3">O vaulted passage to the court
P passage to the court
Q fronton
R loft at the ground floor</td></tr>
<tr><td rowspan="5">SOFT STOREY</td><td>STOREY</td><td>GROUNDFLOOR</td><td>UPPERFLOOR</td></tr>
<tr><td>many holes and/or of large dimension</td><td>A</td><td>E</td></tr>
<tr><td>considerable reduction of floor dimensions</td><td>B</td><td>F</td></tr>
<tr><td>reduced or absence of interior walls</td><td>C</td><td>G</td></tr>
<tr><td>Store with worst mechanical characteristics of the masonry</td><td>D</td><td>H</td></tr>
<tr><td>REINFORCED ELEMENTS</td><td colspan="3">A buttresses and/or spurs
B contrast elements
C* tie rods
D confined openings
E reinforced masonry with injections or non-reinforced plaster
F reinforced masonry or with reinforced plaster
G masonry with other or no identified reinforcement
H diaphragm in RC
I substitution of wood horizontal structures with RC ones
L RC curb
M introduction of metal beams in the horizontal diaphragm</td></tr>
<tr><td>NON-STRUCTURAL ELEMENTS</td><td colspan="3">A absence of non-structural elements
B well connected non-structural elements
C poorly connected small elements</td></tr>
</table>

	D	poorly connected big elements
PRESENT STATE	A	good conditions
	B	medium conditions or widespread damage
	C	bad conditions or medium damage
	D	worst conditions or serious damage
ACCURACY OF THE INSPECTION	A	from the outside inspection
	B	partially inside inspection
	C	complete inspection

Tab. 2.2.1: Legend of the masonry building form: geometrical-typological data and vulnerability information

REFERENCE: Scheda per il rilevamento speditivo degli edifici in muratura. Istruzioni (Consiglio Nazionale delle ricerche, istituto per le Tecnologie della Costruzione)

2.2.2 Masonry buildings: exposition and damage

the second part of this form (Fig 2.2.2, Tab 2.2.2) defines all the general information about exposition and level of damage for each structural unit, such as:

- Actual use and number of occupants;
- Level and extension of damage according to European Macroseismic Scale, for every single element category such as vertical and horizontal structures, roof, stairs and infills and partitions.
- global evaluation and damage of non-structural elements

Below the form[4] and the relative legend[5] are shown

[4] *Scheda per il rilevamento speditivo degli edifici in muratura* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

[5] *Scheda per il rilevamento speditivo degli edifici in muratura. Istruzioni* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

EXPOSITION AND DAMAGE																																	
	exposition (type and n° of unit)											damage to structural, non-structural and existing elements																					
												vertical structures				horizontal diaphragm				stairs				roof				infills and partitions					
building number	utilization	use	residential	productive	commercial	office	public services and religious buildings	warehouse	strategic	tourism - accommodation facilities	n° of occupants	null	D4-D5	D2-D3	D1	null	D4-D5	D2-D3	D1	null	D4-D5	D2-D3	D1	null	D4-D5	D2-D3	D1	null	D4-D5	D2-D3	D1	global level	non-structural elements
I	C	M	I	I	I	I	I	I	I	I	I	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	M

Type of informations contained in columns: C = one-alphanumeric character code; I = integer; N1 = number with one decimal digit; M = alphanumeric string with one or more characters (multiple choice)

Fig. 2.2.2 Masonry buildings: exposition and damage

REFERENCE: Scheda per il rilevamento speditivo degli edifici in muratura (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

<table>
<tr><td>EXPOSITION
-
UTILIZATION</td><td colspan="7">A > 65%
B 30% - 65%
C < 30%
D not used
E under construction
F not finished
G neglected</td></tr>
<tr><td>EXPOSITION
- USE</td><td colspan="7">A residential
B productive
C crafts
D commercial
E office
F public services and religious buildings
G warehouse
H strategic
I tourism - accommodation facilities
L ______________________</td></tr>
<tr><td rowspan="2">DAMAGE -
LEVEL/EXTENSION</td><td colspan="2"></td><td colspan="2">>2/3</td><td colspan="2">1/3-2/3</td><td><1/3</td></tr>
<tr><td colspan="2">D4-D5
D2-D3
D1</td><td colspan="2">A
D
G</td><td colspan="2">B
E
H</td><td>C
F
I</td></tr>
<tr><td rowspan="2">DAMAGE -
EMS/GLOBAL
LEVEL</td><td>CODE</td><td>0</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><td>Structural

Non-structural</td><td>Null

null</td><td>Null

low</td><td>Low

severe</td><td>Mean-severe
/</td><td>Partial collapse
/</td><td>Total collapse
/</td></tr>
<tr><td>DAMAGE -
NON-STRUCTURAL ELEMENTS</td><td colspan="7">1 plasters, coverings and false ceilings
2 tiles, chimneys …
3 ledge, parapets …
4 other internal or external objects
5 other damage</td></tr>
</table>

Tab. 2.2.1: Legend of the masonry building form: exposition and damage

REFERENCE: Scheda per il rilevamento speditivo degli edifici in muratura. Istruzioni (Consiglio Nazionale delle ricerche, istituto per le Tecnologie della Costruzione)

2.2.3 Reinforced concrete buildings

This form (Fig 2.2.3, Tab 2.2.3) organizes the general information about reinforced concrete buildings, such as:

- Geometrical data like number of stories, plan area, height,;
- Typological data about structural system, joints and building regularity;

Below the form[6] and its legend[7] are shown.

[6] *Informazioni preliminari al censimento di vulnerabilità(cemento armato)* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

[7] *Istruzioni per la compilazione delle schede per il censimento speditivo di vulnerabilità* (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

REINFORCED CONCRETE BUILDING																	
1	n° building																
2	n° storey																
3	H first floor [m]																
4	H max [m]																
5	H min [m]																
6	use																
7	age																
8	floor area [mq]																
9	n° storey on the main facade																
10	H main facade [m]																
11	joint																
12	structural system																
13	structural mesh																
14	first level pillars dimensions																
15	plan regularity																
16	soft storey																
17	first level infills																
18	squat elements																
19	structurl bow-windows																
20	use																
21	months of use																
22	occupants																
23	accuracy of inspections																

Fig. 2.2.3 Reinforced concrete building form

REFERENCE: Scheda per il rilevamento speditivo degli edifici in cemento armato (Consiglio Nazionale delle Ricerche, Istituto per le Tecnologie della Costruzione)

TOTAL NUMBER OF STORIES	Fill out the total number of the building stories, including underground floors. Attic should be excluded unless its average height is less than 20% of the average interstorey height of the building
FIRST FLOOR H	Fill out the height of the ground floor
H MAX/MIN	Fill out the maximum/minimum height of the building, estimated from the ground to the eaves
USE	residential 1=yes 2=no productive 1=yes 2=no public services 1=yes 2=no
AGE	A < 1919 B 1919-1945 C 1946-1960 D 1961-1971 E 1972-1981 F > 1981 G _________
FLOOR AREA	The plan area of the building can be estimated through the use of the cartography, with maximum approximation of 10%
NUMBER OF STORIES OF THE MAIN FACADE	Fill out the number of the storey on the main facade. Attic should be excluded if its average height is less than 20% of the average interstorey height of the building.
MAIN FACADE HEIGHT	Fill out the average height of the main facade, estimated from the ground to the eaves.
JOINT	1 isolated

	2 according to law 3 not according to law
STRUCTURAL SYSTEM	A prevalence of walls or frames with stiff masonry infills (without large holes and with resistant materials) B prevalence of frames with beams that are higher than the thickness of the horizontal diaphragms and bad quality infills C prevalence of frames with beams that have the same thickness of the horizontal diaphragms and bad quality or absent infills D frames with high beams on the perimeter with bad quality infills and beams that has the same thickness of the horizontal diaphragms E presence of frames with beams that are higher than the thickness of the horizontal diaphragms and r.c. walls F prevalence of r.c. walls
FIRST LEVEL PILLARS DIMENSION	A average dimension < 25 cm B average dimension > 25 cm and < 40 cm C average dimension > 40 cm
PLAN REGULARITY	1 compact and regular 2 compact and regular on the average 3 not compact and irregular
SOFT STOREY	1 absent 2 pilotis 3 absent or inadequate infills 4 overhanging infills
FIRST LEVEL INFILLS	A on 4 perimeter walls B on 3 perimeter walls C on 2 perimeter walls D on 1 perimeter wall

SQUAT ELEMENTS	A absent B for beams in stair system or different level floors C ribbon windows D other
STRUCTURAL BOW-WINDOWS	1 absent 2 < 1,5 m 3 > 1,5 m
USE	1 neglected 2 not used (<10%) 3 partially used (10%-70%) 4 used (>70%)
MONTHS OF USE	Write the number of months of use
OCCUPANTS	A 1 family B 2 families C 3-4 families D 5-8 families E 9-15 families F 16-30 families G > 30 families
ACCURACY OF INSPECTION	A from the outside inspection B partially internal inspection C complete inspection

Tab. 2.2.3: Legend of the reinforced concrete buildings form

REFERENCE: Istruzioni per la compilazione della scheda per il censimento speditivo di vulnerabilità, edifici in cemento armato (Consiglio Nazionale delle ricerche, istituto per le Tecnologie della Costruzione)

2.3 DATA ANALYSIS

The data collected in situ are reorganized and statistically analyzed in order to identify the most prevalent characteristics of the center and the suburbs; they are graphically represented by the tables in ANNEX X and in the following paragraphs by histograms and pie charts.

The analyzed data represents all the collected data (both the center of Timisoara and the district of Iosefin) . On the other hand, the typologies recognized only in Iosefin have been divided from the others.

Histograms represent the number of buildings that show the analyzed characteristic, emphasizing the more frequent feature, while in the pie-carts the percentage of building on the total number of buildings is specified.

The identification of the most frequent characteristics is the starting point to create typologies models that represent, in a schematic way, the majority of the buildings of Timisoara. The second step of typologies identification is explained in paragraph 2.4.

2.3.1 Building typology

The analyzed structural units are 243 and the biggest part of them (87%) are masonry buildings of which 10 are monumental buildings (4%). Reinforced concrete buildings represent 9% of the total (Fig 2.3.1, Fig 2.3.2).

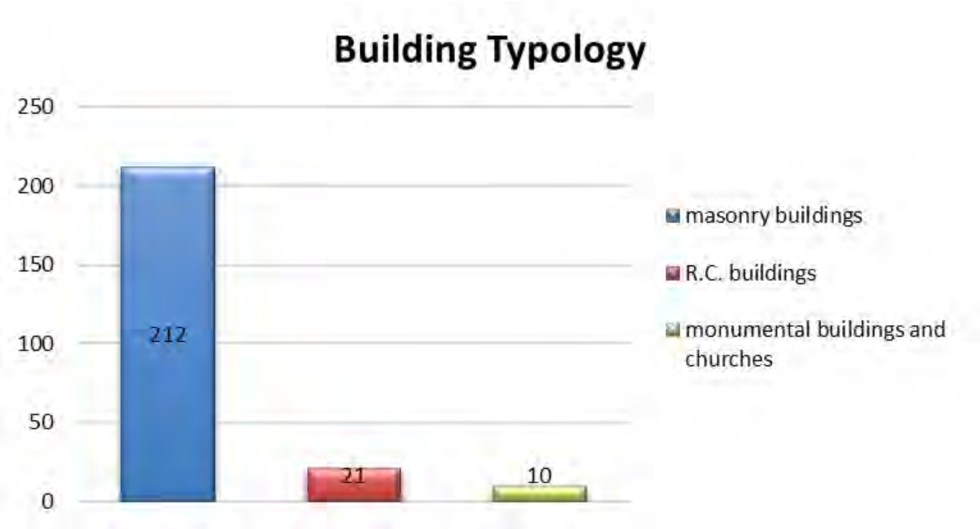


Fig. 2.3.1 Number of buildings for each building typology

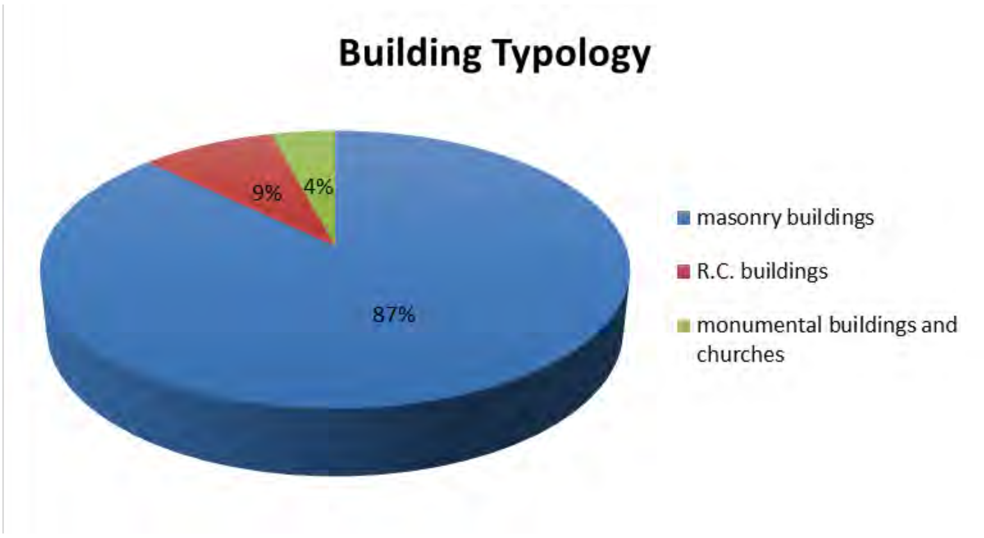


Fig. 2.3.2 Percentage of buildings for each building typology

2.3.2 Inspection accuracy

The biggest part of the buildings in the historical center is actually used for several functions and there are few abandoned units. Many buildings present shops or public offices at the ground floor and apartments at the upper floors.

A partially inside inspection was possible usually for offices or shops (36%). Only for 28 structural units (13%), like libraries and museums, a complete inspection was possible but for the substantial part of cases (51%) it was possible to carry out just a limited inspection from the outside or from the courtyard (Fig 2.3.3, Fig 2.3.4). This fact greatly limited the level of knowledge of the analyzed structures.

Fig. 2.3.3 Number of buildings for each level of inspection accuracy

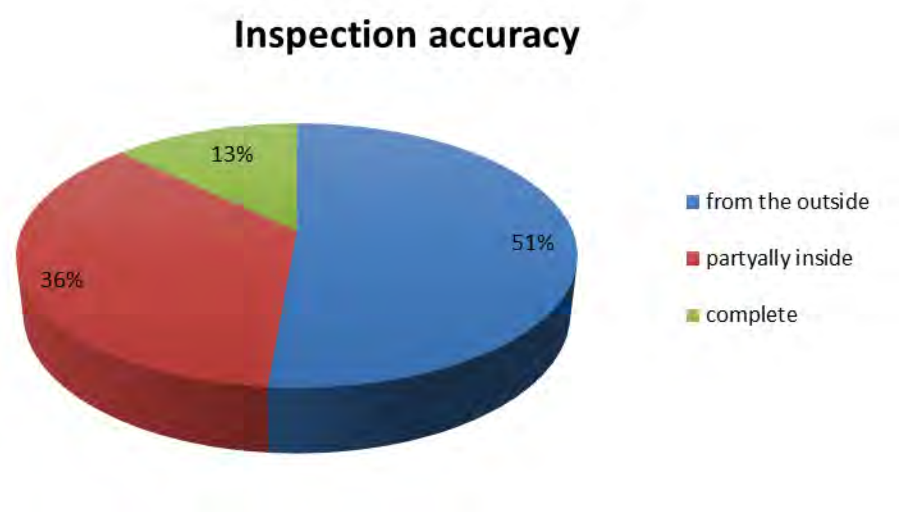


Fig. 2.3.4 Percentage of buildings for each level of inspection accuracy

2.3.3 Ages

Most of the buildings of Timisoara had been destroyed and re-constructed during the Hasburg period, so only 48 structural units (22%) are prior to 1747. The 28% of the buildings were built between 1747 and 1812, while only 20 buildings were constructed from 1812 to 1876. The biggest part of the units was built between 1877- 1945, after the 1848 Revolution that destroyed a big part of the city. Only the 2% are recent construction, realized after 1946 (Fig 2.3.5, Fig 2.3.6).

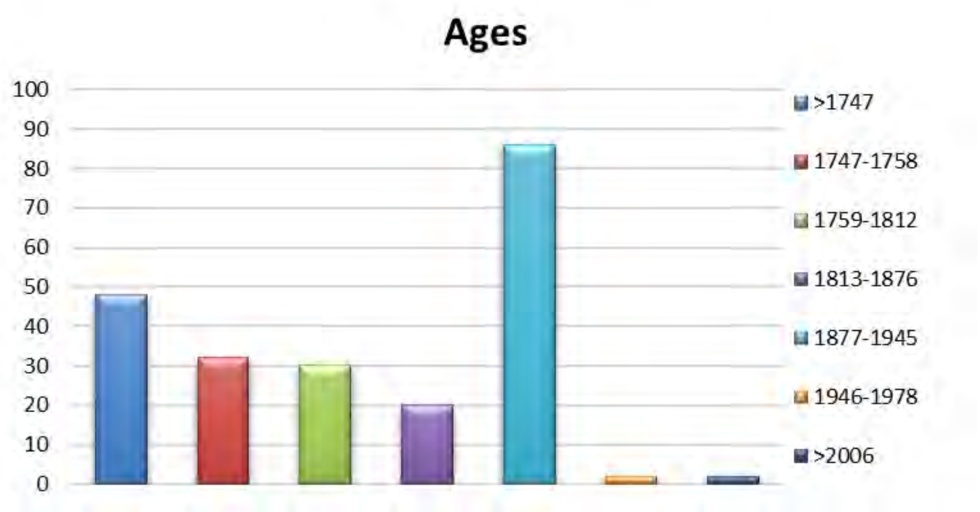


Fig. 2.3.5 Number of buildings for each age range

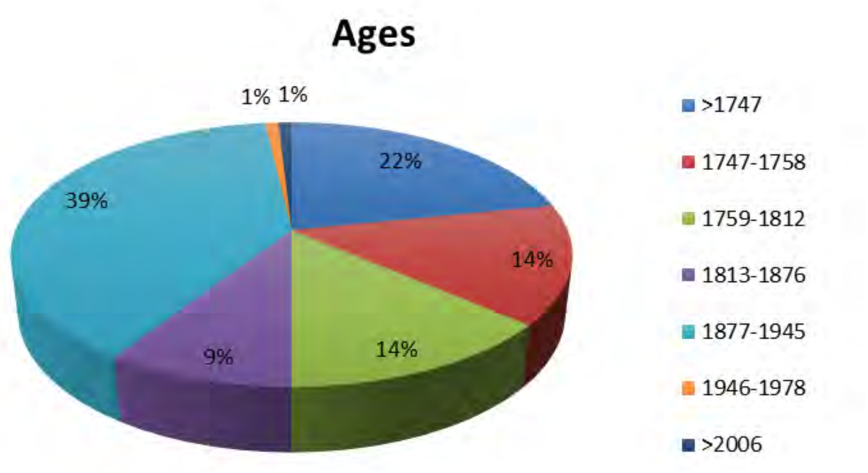


Fig. 2.3.6 Percentage of buildings for each age range

2.3.4 Interventions

The most common interventions are renewal (22%), restoration (22%), façade restoration (8%), and maintenance (21%), that includes more than the 50% of the buildings. For 33 structural units interventions were not recognized and there were ongoing works on 17 buildings. A little percentage considers enlargement and possible demolition/bombing and later reconstruction (Fig 2.3.7, Fig 2.3.8).

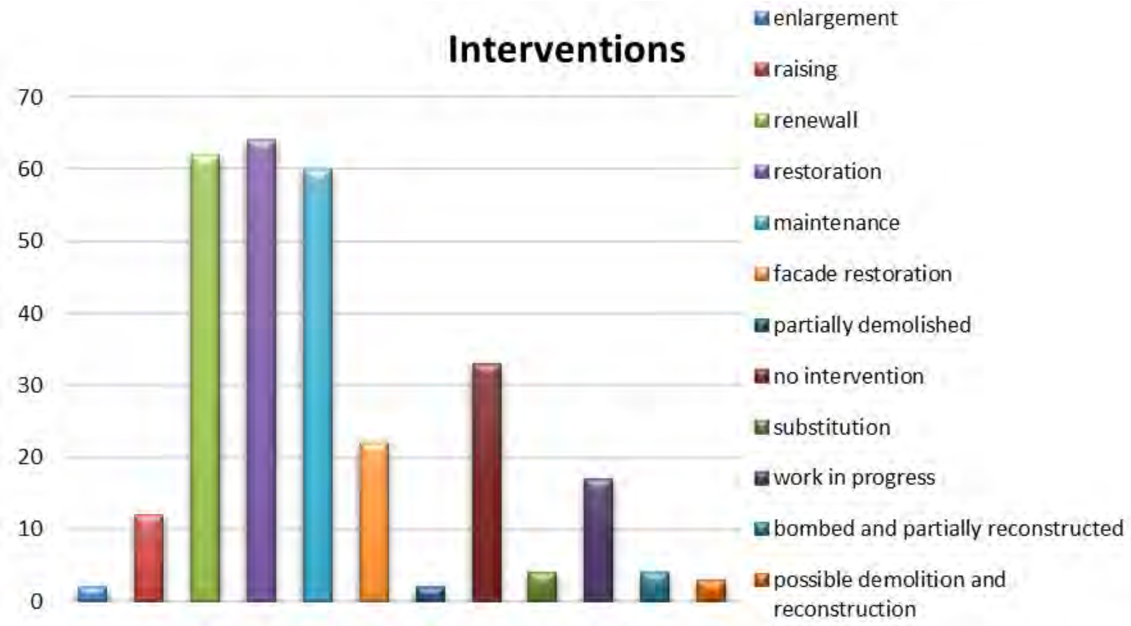


Fig. 2.3.7 Number of buildings for each intervention

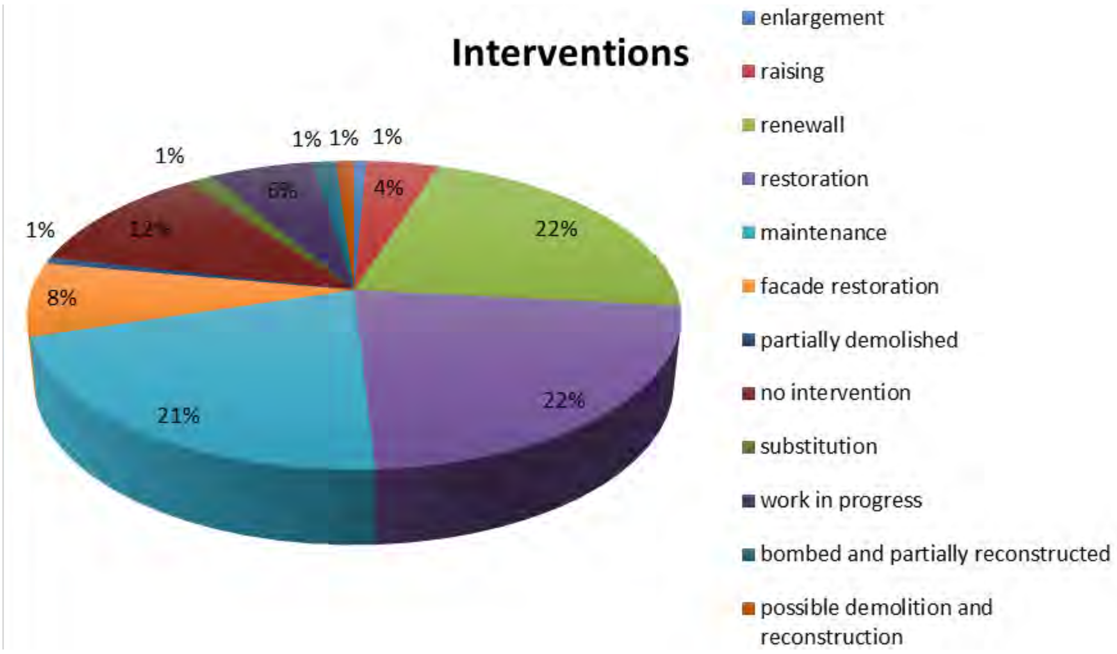


Fig. 2.3.8 Percentage of buildings for each intervention

2.3.5 Stories number

The biggest part of buildings is two or three stories high, in fact 166 buildings represent respectively the 35% and the 40% of the total. Afterward there is a 14% of building with only one story and a 10% with four stories. Finally, there are only 2 structural units with five stories and one with six (Fig 2.3.9, Fig 2.3.10).

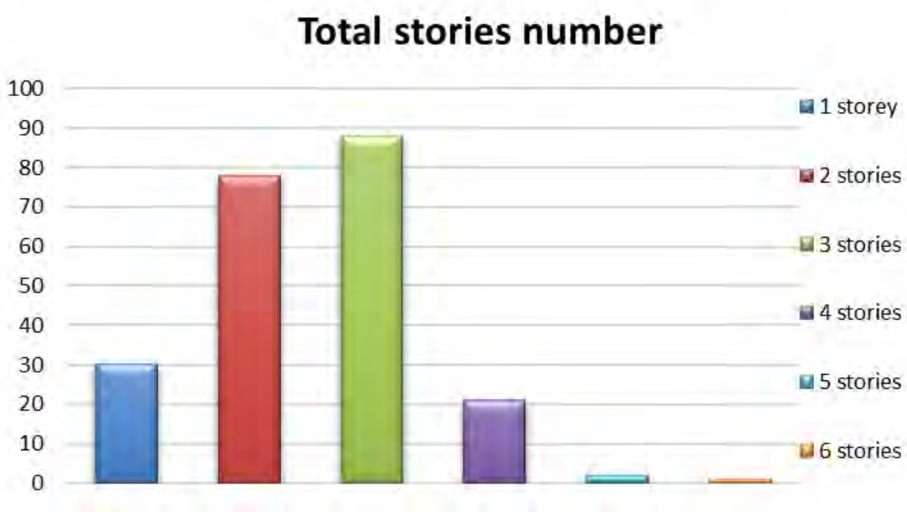


Fig. 2.3.9 Number of buildings for each number of stories

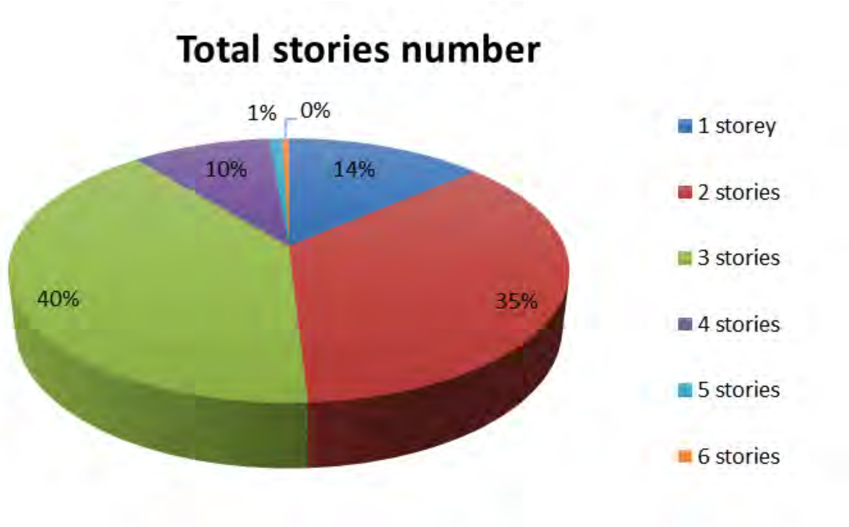


Fig. 2.3.10 Percentage of buildings for each number of stories

As can be seen in Fig 2.3.11 and Fig 2.3.12 there are 55 buildings (29%) with a basement and 40 (20%) with an underground. There are then 27% of building with basement and attic and 24% with underground and attic.

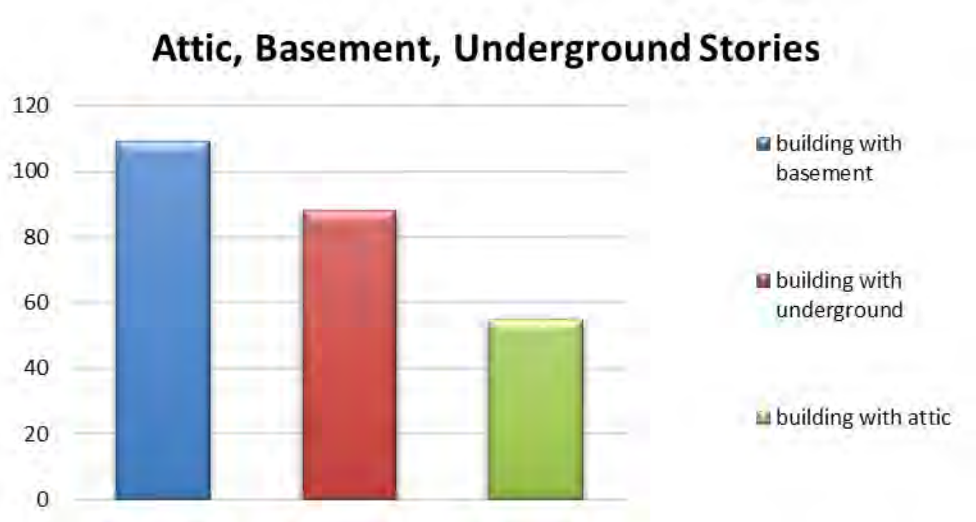


Fig. 2.3.11 Number of buildings with attics, basements and underground stories

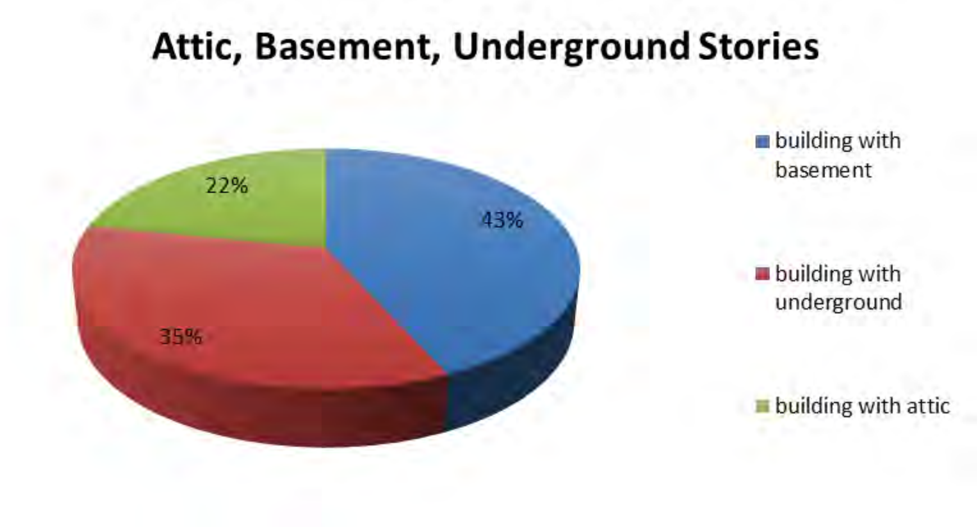


Fig. 2.3.12 Percentage of buildings with attics, basements and underground stories

The following histogram (Fig 2.3.13, Fig 2.3.14) shows that there is an equal division between buildings with basement (29%), with basement and attic (27%), with underground (20%) or with underground and attic (24%).

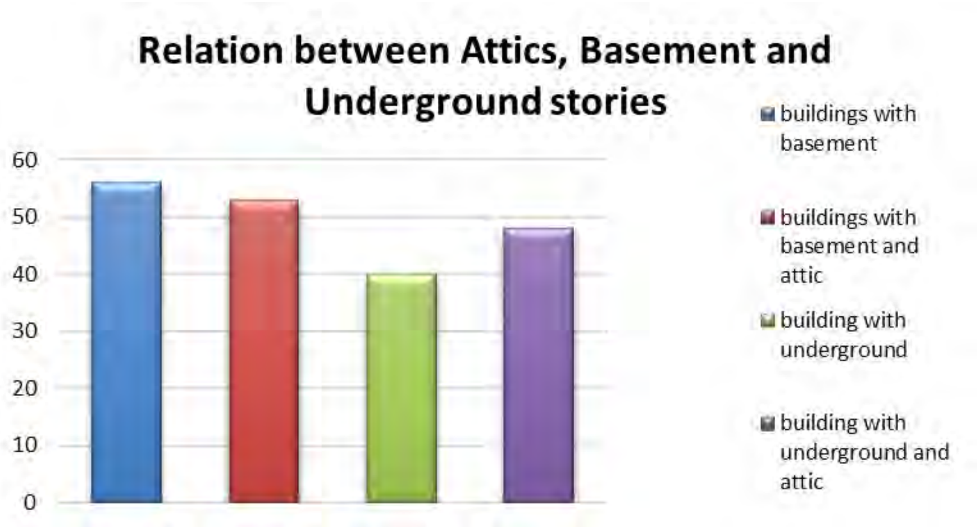


Fig. 2.3.13 Number of buildings with basement, basement and attic, underground and underground and attic

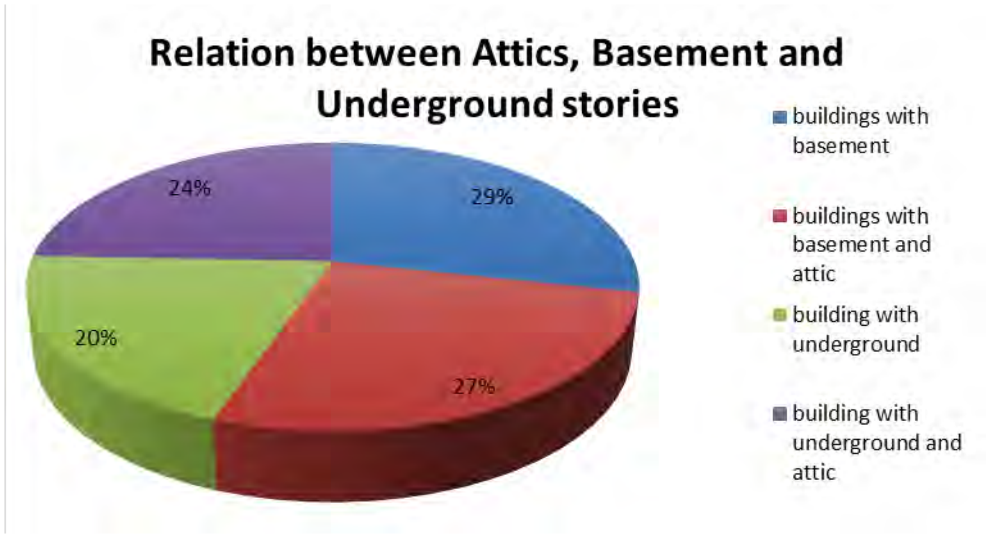


Fig. 2.3.14 Percentage of buildings with basement, basement and attic, underground and underground and attic

The following histogram represented the number of out of ground stories related with basement and underground. As can be seen the biggest part of buildings is two or three stories high with basement or underground (Fig 2.3.15, Fig 2.3.16).

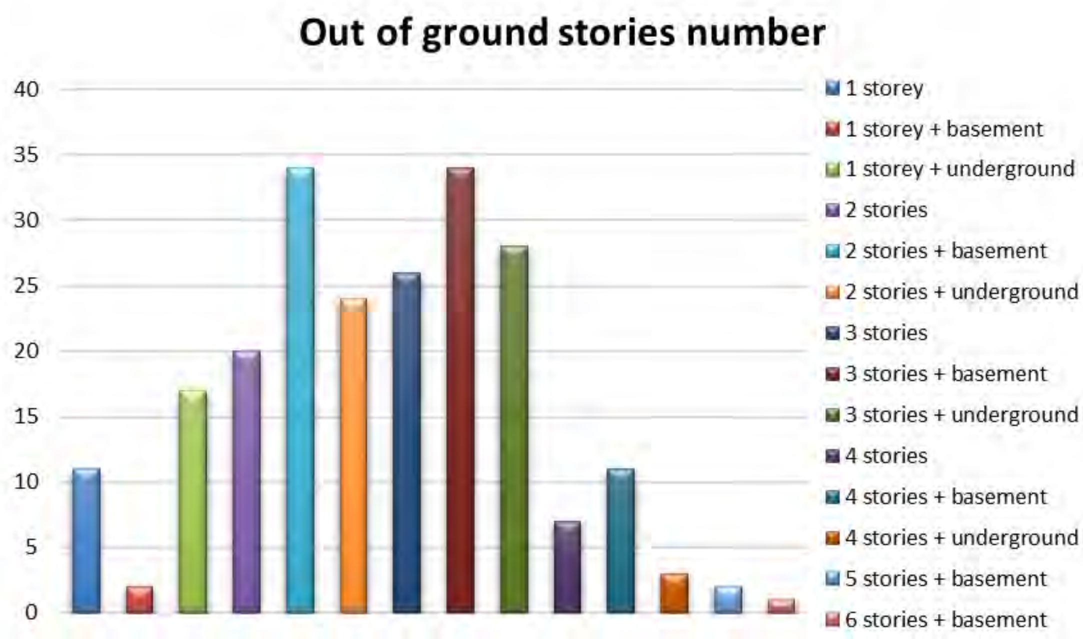


Fig. 2.3.15 Number of buildings for each number of out of ground stories

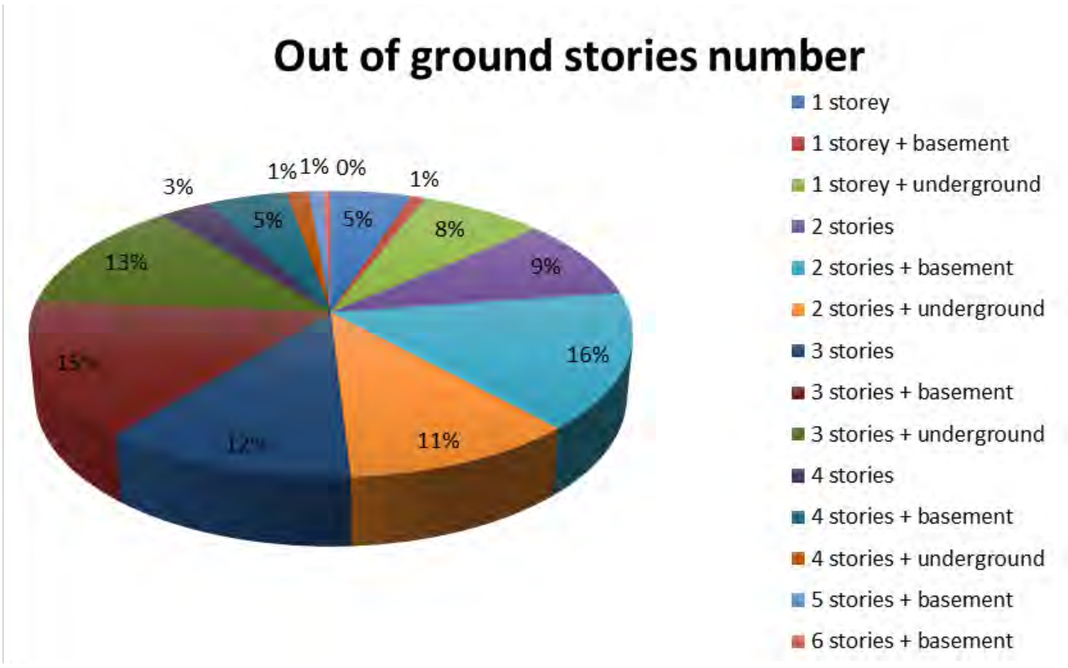


Fig. 2.3.16 Percentage of buildings for each number of out of ground stories

2.3.6 Floor area

Due to the large number of buildings, it was decided to consider twelve area ranges, from less than 100 mq to more than 2000 mq with a step of 100 mq until 1000 mq, that considers all the biggest units (Fig 2.3.17, Fig 2.3.18). The majority of buildings has an area included between 300-400 mq (24%) and also the steps before and after this one have a large number of units, with 33 (15%) and 31 (15%) structural units. Other two relevant groups are 100-200 mq (9%) and 1000-2000 mq (9%); the others have a percentage smaller than 9% (Fig 2.3.17, Fig 2.3.18).

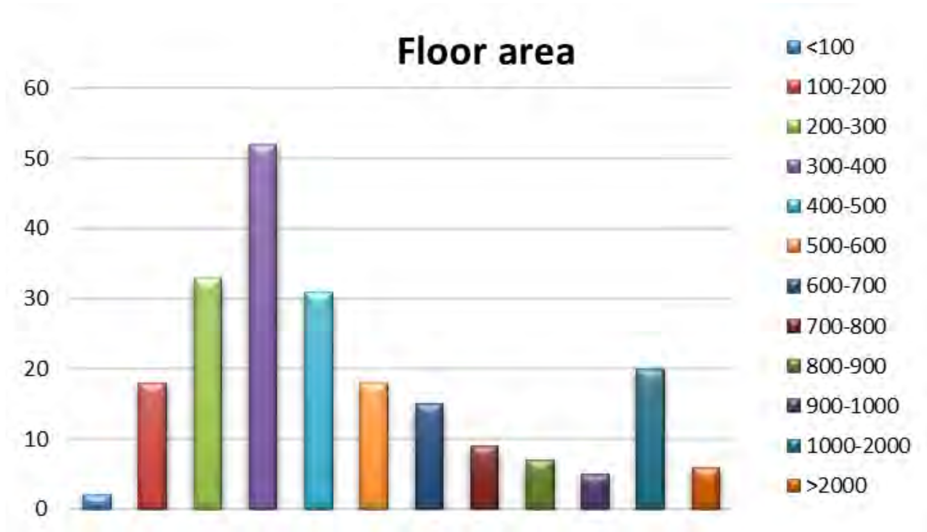


Fig. 2.3.17 Number of buildings for each range of area

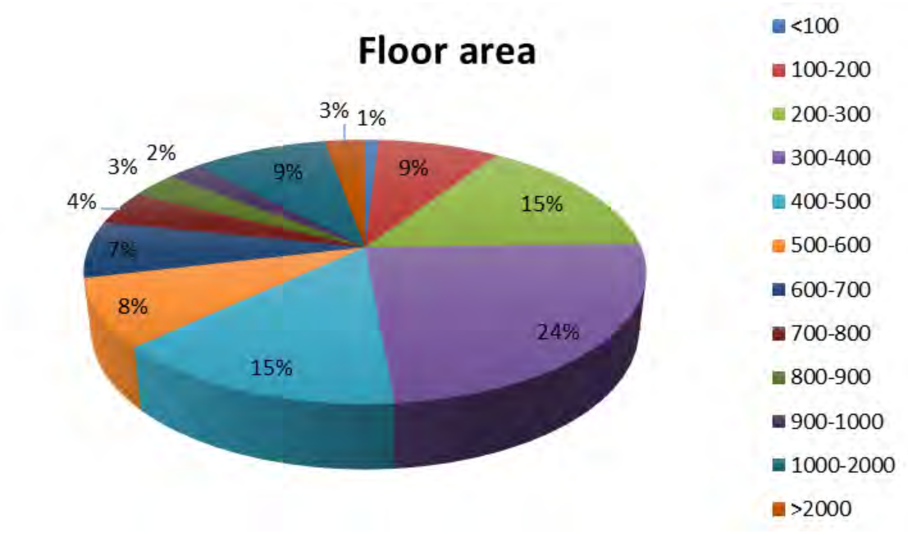


Fig. 2.3.18 Percentage of buildings for each range of area

2.3.7 Interstorey height

Referring to buildings interstorey height, data are quite diversified: there are several buildings between 3,5 and 4,5 meters and some between 4,5 and 5,4 meters. Only 4 buildings are included between 3-3,4 meters, while the 44% of the units have an interstorey height between 3,5-3,9 meters. Another important range is from 4 to 4,4 meters with 65 building (32%) and the other two ranges describe the higher interstorey, between 5- 5,4, 5,5-6 and more than 6, with 22 total buildings (Fig 2.3.19, Fig 2.3.20).

Fig. 2.3.19 Number of buildings for each range of interstorey height

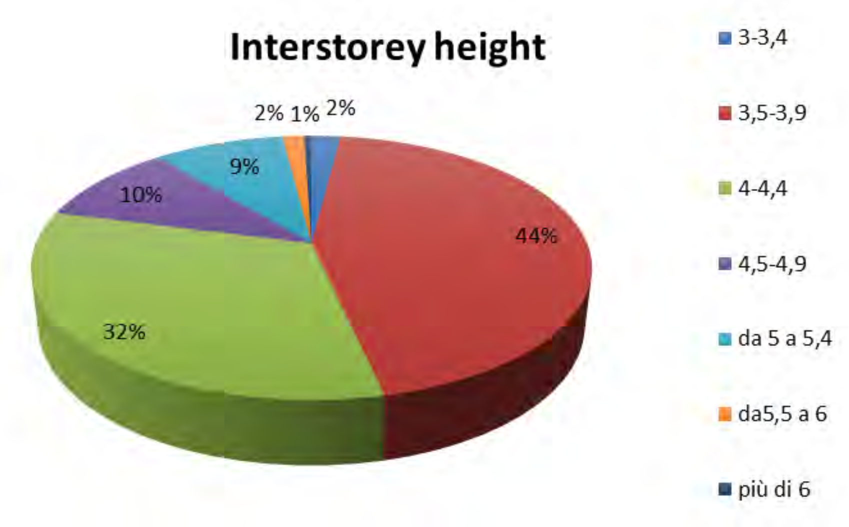


Fig. 2.3.20 Percentage of buildings for each range of interstorey height

2.3.8 Building height

In the city of Timisoara there are not really high building and the biggest part has an highness between 10-15 meters (48%), 61 units are between 5 and 10 meters (28%), and 5% are units with only one floor, lower than 5 meters. 32 units have a height ranging between 15-20 meters (15%) and the 4% is higher than 20 meters (Fig 2.3.21, Fig 2.3.22).

Fig. 2.3.21 Number of buildings for each range of building height

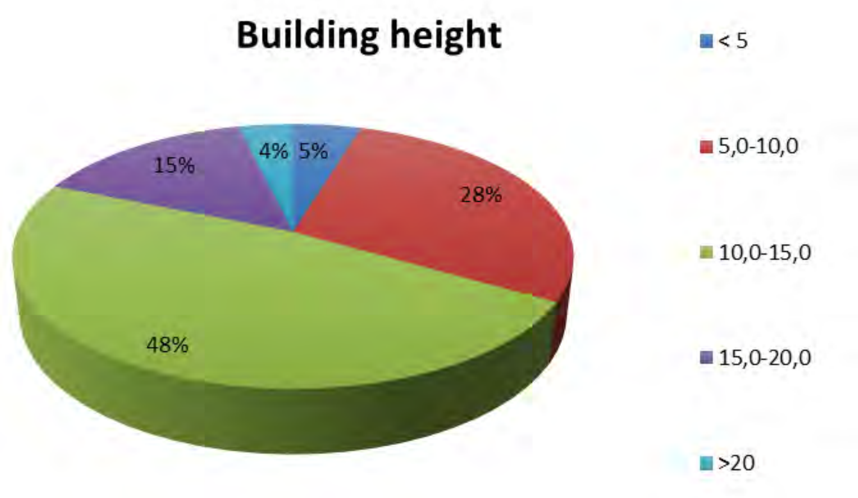


Fig. 2.3.22 Percentage of buildings for each range of building height

2.3.9 Holes in façade

The buildings in Timisoara are characterized by a regular façade pattern of windows and openings. Most of the buildings in the city of Timisoara have a high percentage of openings, referred to the principal and the other visible facades, in fact one 199 buildings have more than 30% of holes in the total surface and only 25 between 20% and 30% (Fig 2.3.23, Fig 2.3.24, Fig 2.3.25, Fig 2.3.26).

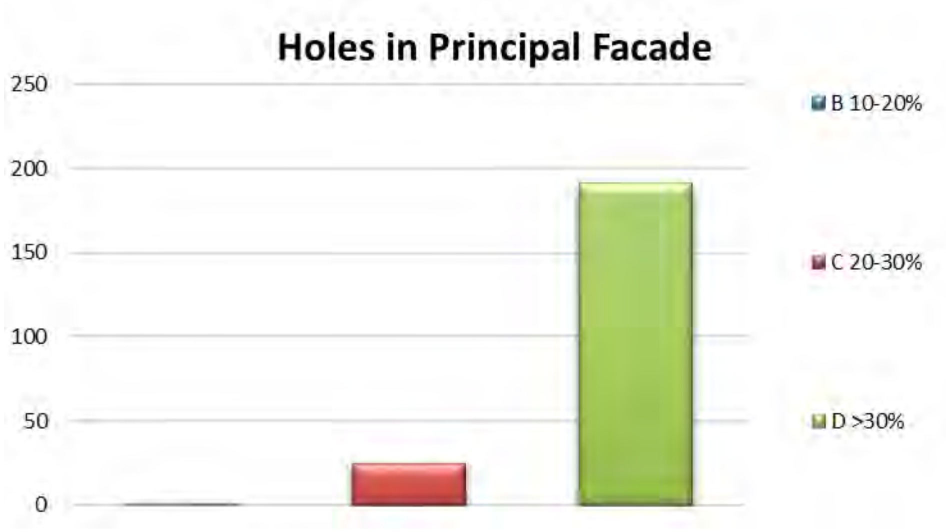


Fig. 2.3.23 Number of buildings for each percentage of holes in principal facade

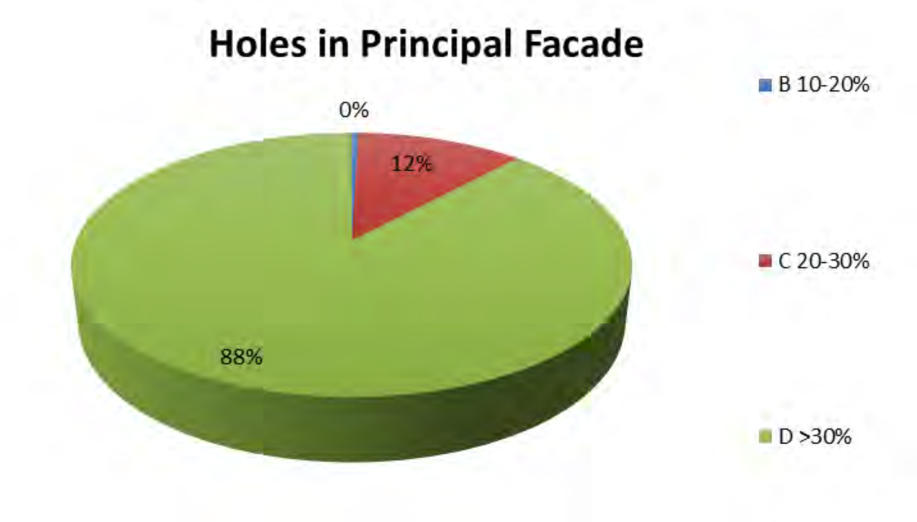


Fig. 2.3.24 Percentage of buildings for each percentage of holes in principal facade

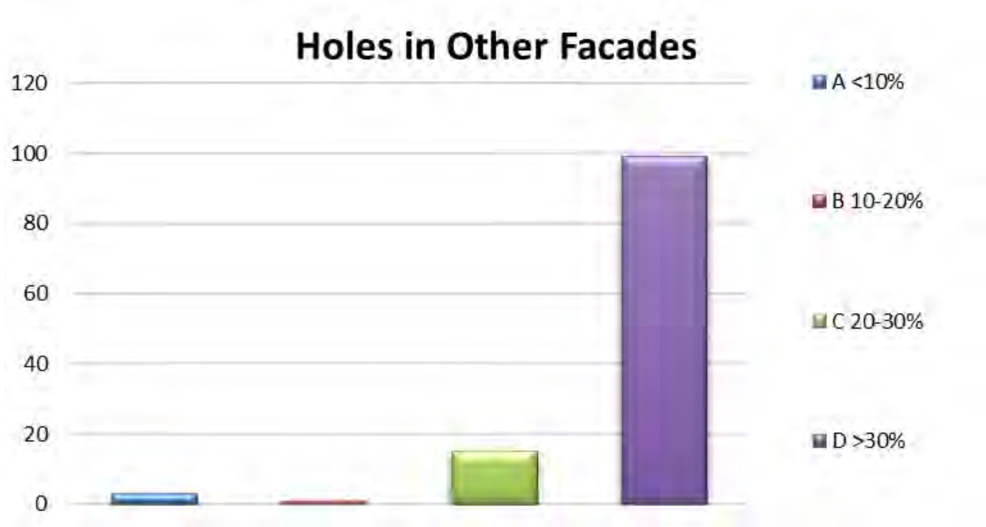


Fig. 2.3.25 Number of buildings for each percentage of holes in the other facades

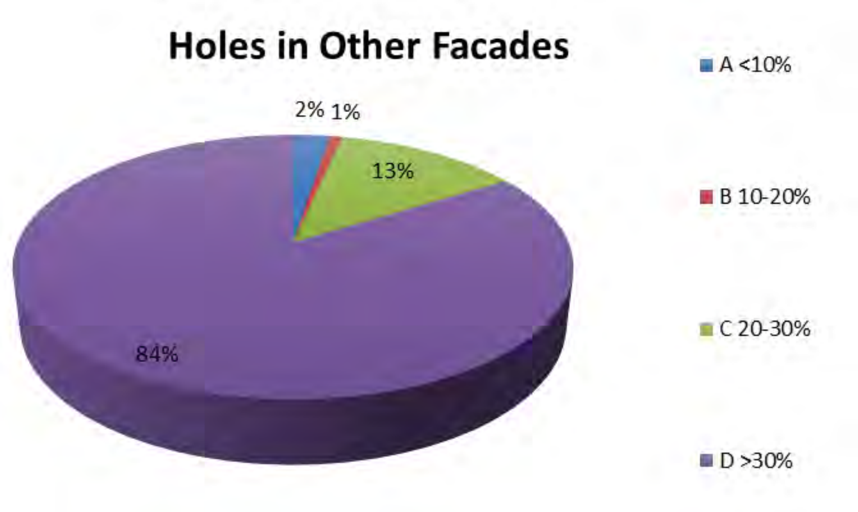


Fig. 2.3.26 Perceentage of buildings for each percentage of holes in the other facades

2.3.10 Structural typology

The city is composed in prevalence by masonry buildings among which it is possible to note the predominance of structural units characterized by interior bearing walls (90%), detected in 195 buildings (Fig 2.3.27, Fig 2.3.28). 10 cases present intervention with reinforced concrete while 5 buildings have reticular metal column at the ground floor and masonry walls at upper floors and other 5 have a masonry pillars structure.

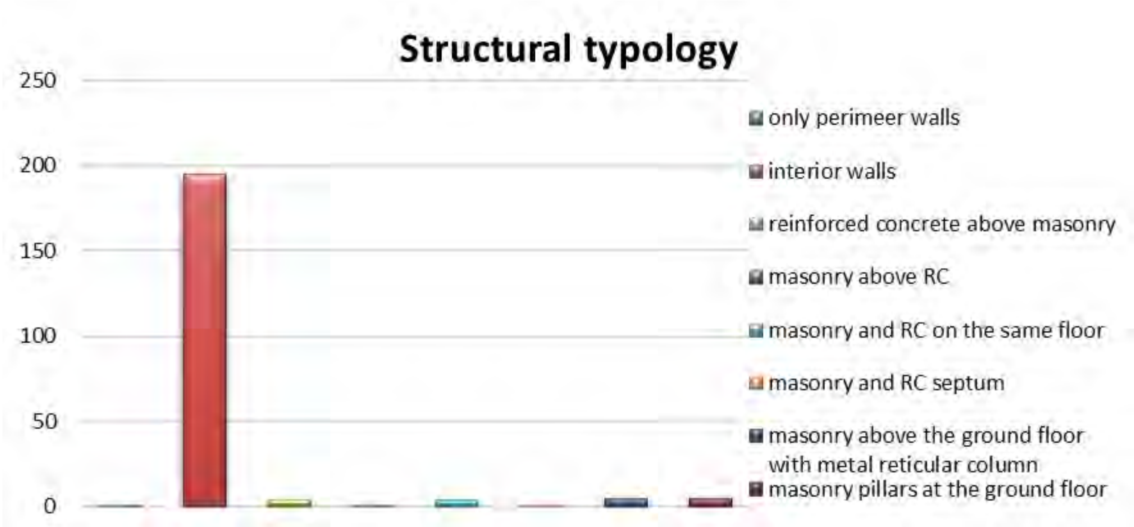


Fig. 2.3.27 Number of buildings for each structural typology

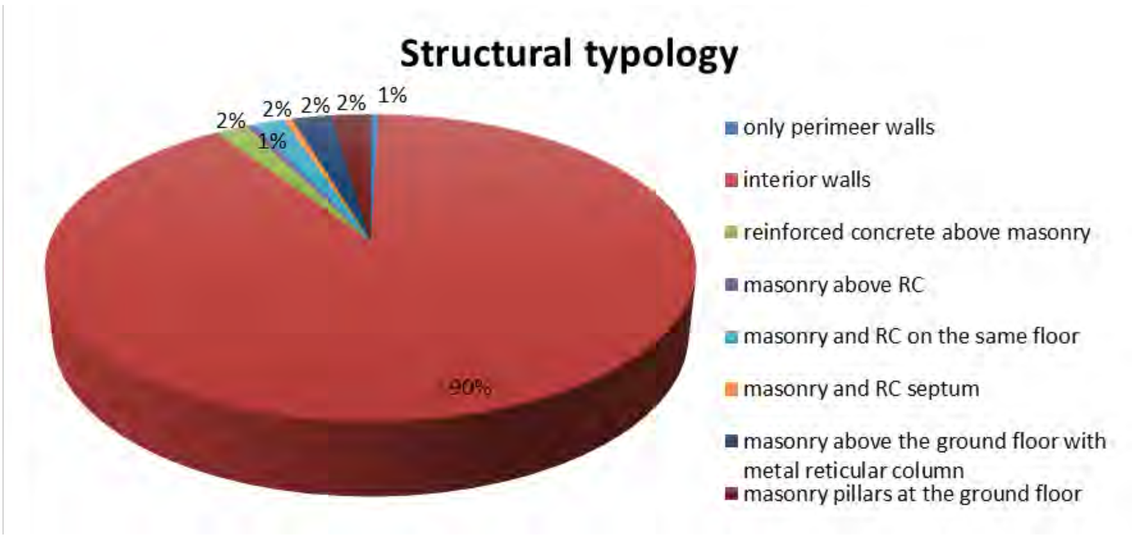


Fig. 2.3.28 Pecentage of buildings for each structural typology

2.3.11 Vertical structures

During the survey just few buildings had been inspected inside to list the presence of different vertical structures.

Regarding prevalent typologies, they have been detected mainly from the outside and the most frequent technique is brick masonry (89%), which is identified in 213 buildings. Only one structural unit presents a mixed structure with a RC wall inside and masonry perimeter bearing walls, while 5 buildings are characterized by the presence at the ground floor of metal reticular columns that describe big openings. The presence of other vertical structures is not significant, but in 9 cases, in the courtyard, metal columns were detected (Fig 2.3.29, Fig 2.3.30).

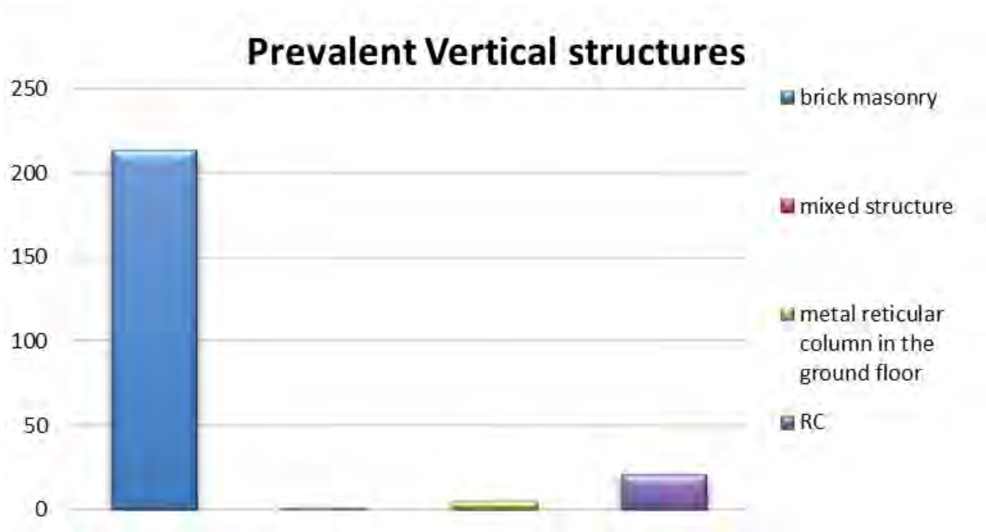


Fig. 2.3.29 Number of buildings for each prevalent vertical structures

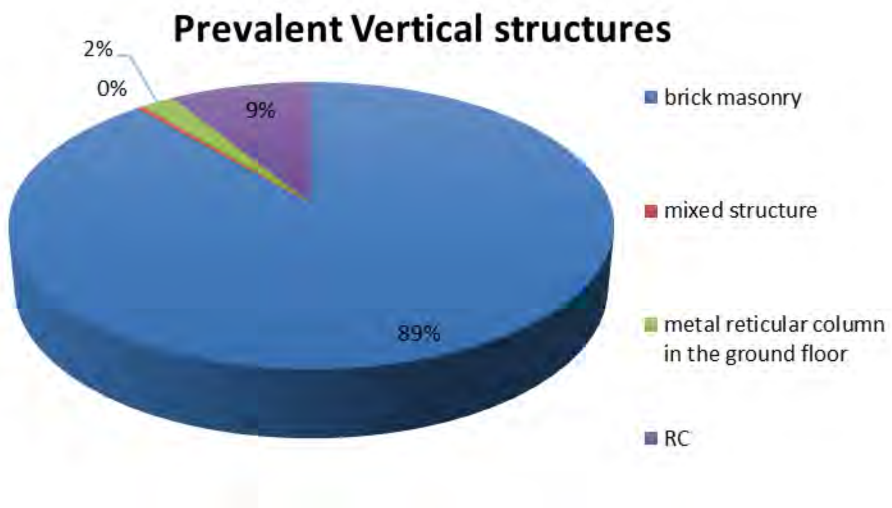


Fig. 2.3.30 Percentage of buildings for each prevalent vertical structures

2.3.12 Horizontal structures

During the survey of horizontal structures, the typology of underground, ground floor, and upper floors horizontal structures were identified.

Regarding the underground floor, the prevalent one is masonry vaults, detected in 137 structural units (72%). Only 25 buildings have a concrete slab, while another important percentage (globally 25%) is represented by vaults or very low vaults and iron beams (Fig 2.3.31, Fig 2.3.32).

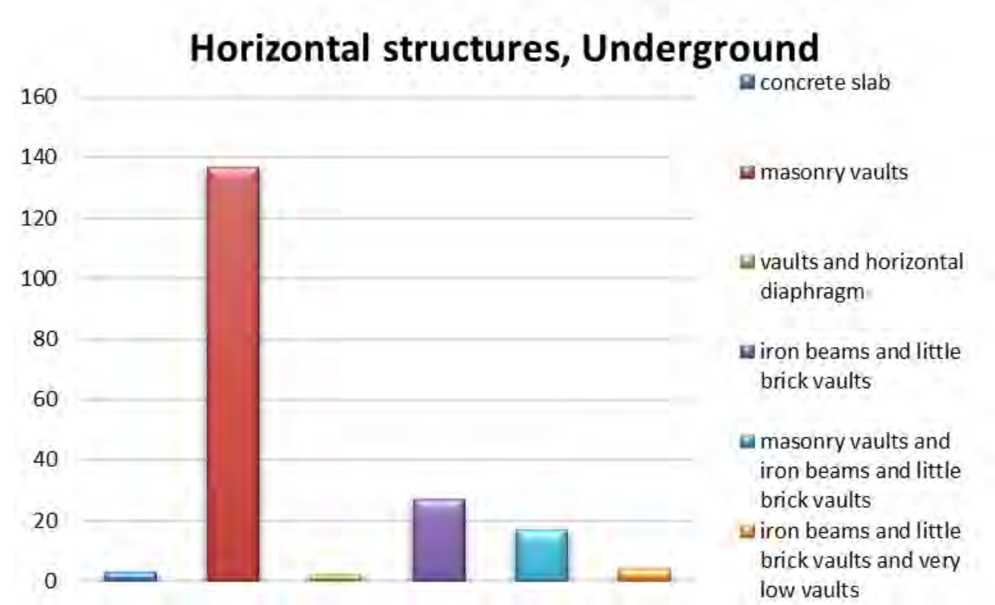


Fig. 2.3.31 Number of buildings for each prevalent horizontal structures, Underground

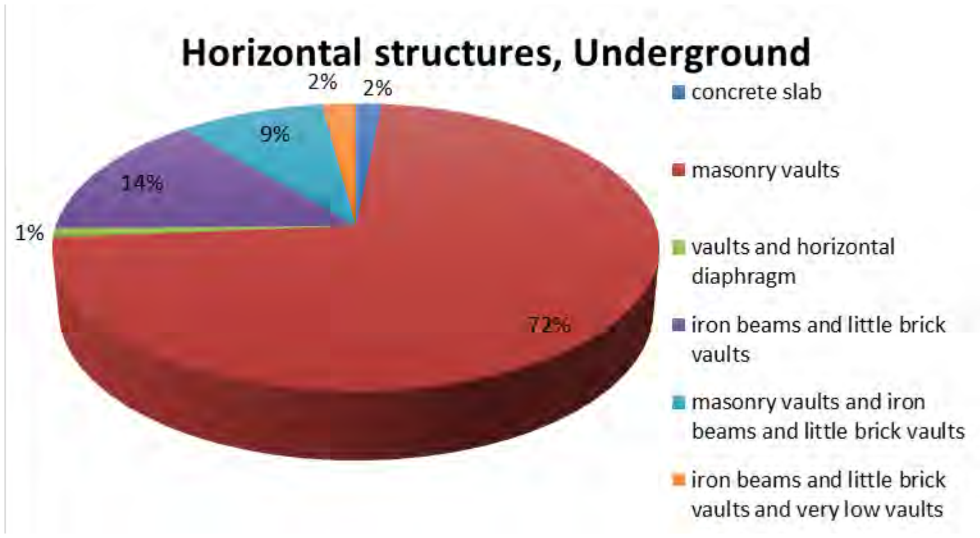


Fig. 2.3.31 Percentage of buildings for each prevalent horizontal structures, Underground

Likewise, the ground floor prevalent horizontal structure is masonry vaults, detected in 103 units (47%), but an important part is composed by timber (17%) and concrete slab (12%). In 45 buildings little brick vaults or very low vaults (21%)were detected (Fig 2.3.33, Fig 2.3.34).

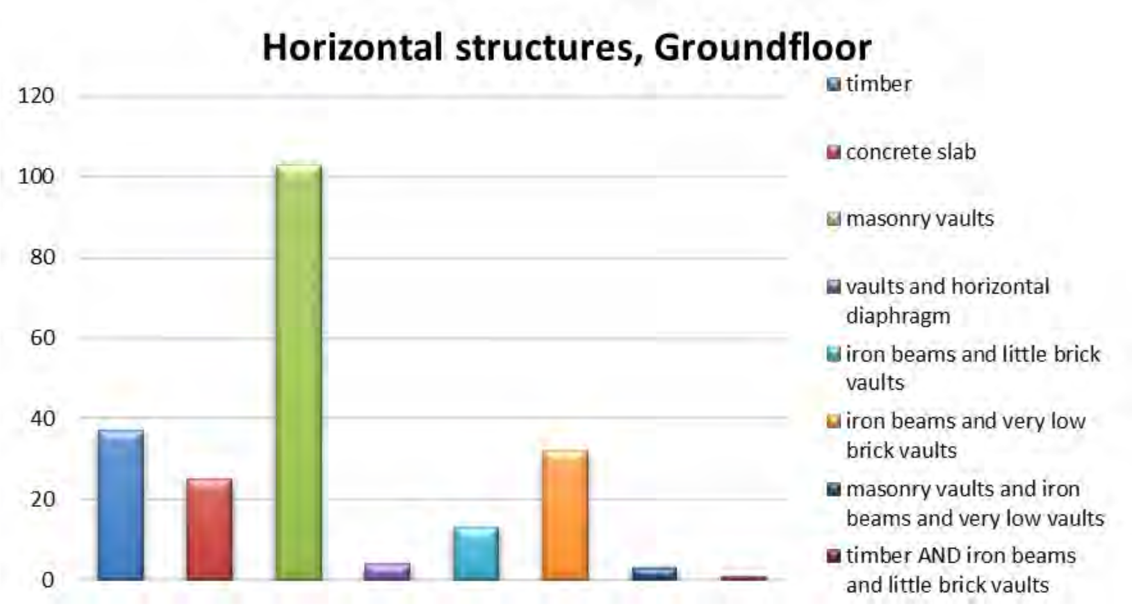


Fig. 2.3.33 Number of buildings for each prevalent horizontal structures, Ground floor

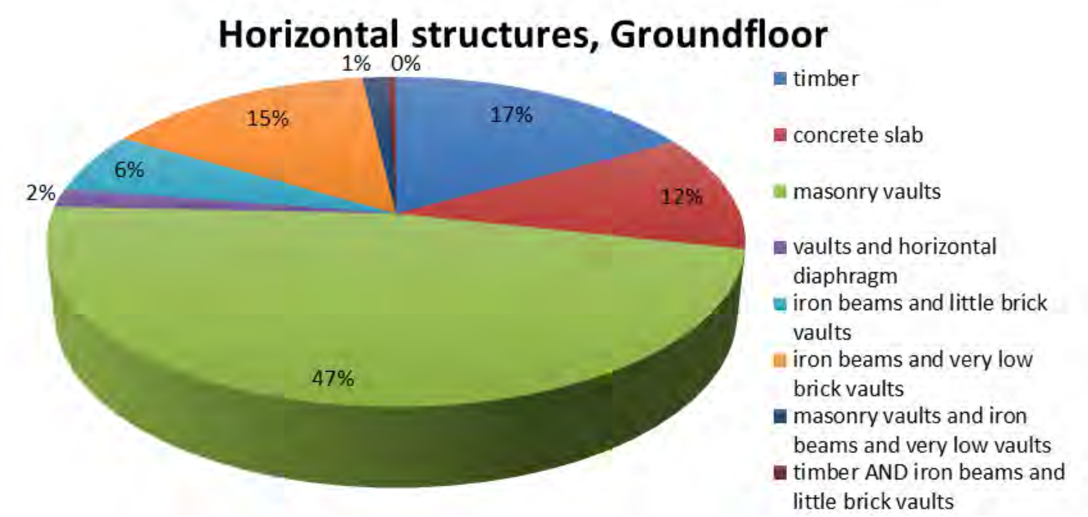


Fig. 2.3.34 Percentage of buildings for each prevalent horizontal structures, Ground floor

Differently, the predominant horizontal structures of upper floors is timber (83%). 21 units present concrete slabs (11%) and a little part is composed by the combination of timber, concrete slab and little vaults. The determination of the horizontal structures of the upper floors was difficult and a lot of times it is only a hypothesis, caused by the impossibility to enter the buildings.

As a result of the analysis, vaults without tie rods and timber structures are the most characteristic horizontal structure typologies in Timisoara, but also little brick vault with iron beams and the concrete slab are widely used for horizontal structures (Fig 2.3.35, Fig 2.3.36).

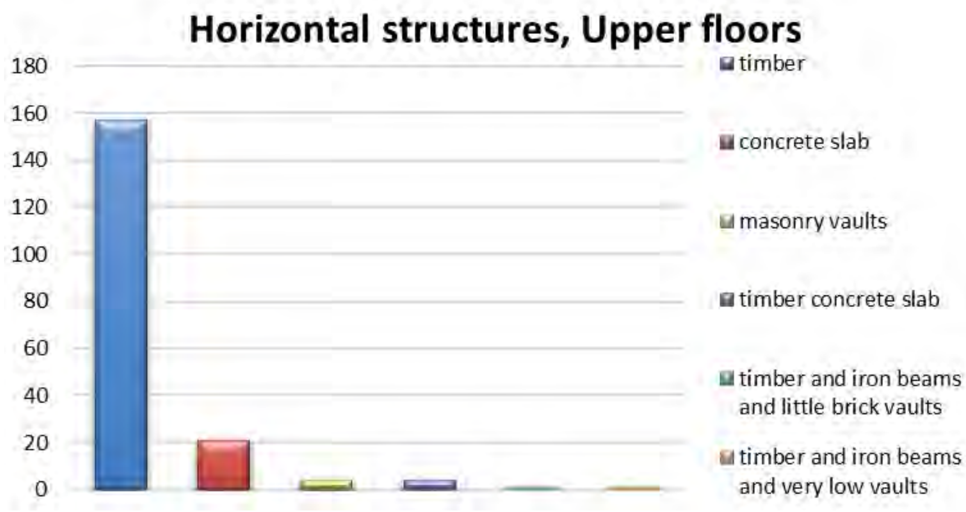


Fig. 2.3.35 Number of buildings for each prevalent horizontal structures, Upper floors

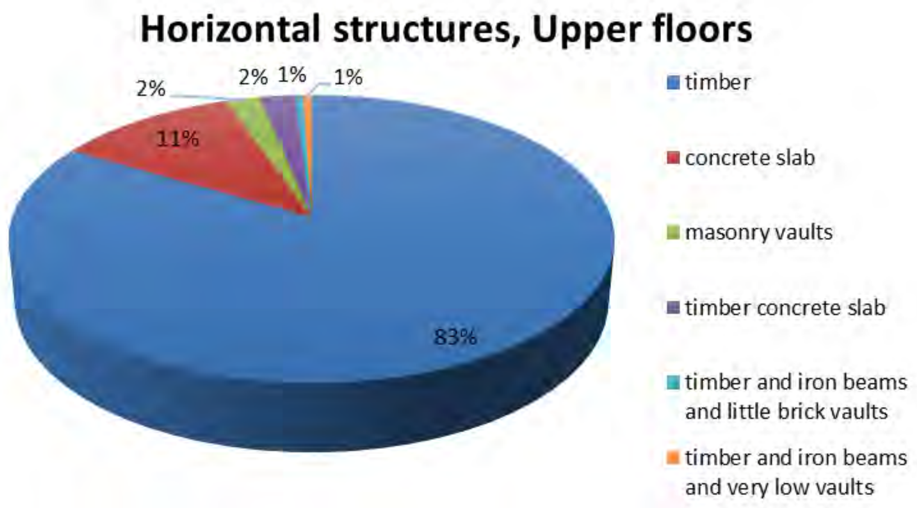


Fig. 2.3.6 Percentage of buildings for each prevalent horizontal structures, Upper floors

2.3.13 Roofs

The determination of roof typologies was based only on external evaluation and on the literature analysis. In Timisoara roofs appear very high and in many cases they have been transformed into attics. 175 buildings have a contrasted thrust timber roof (81%), characterized by the structure described in chapter 1.9.3. This technology was used until the XX's century, but the recent renewal or restoration interventions, with the introduction of walkable attics, caused sometimes the re-construction of the roof with the same shape but using reinforced concrete; this is the reason why in 40 units (40%) a non-thrusting mixed roof was found (Fig 2.3.37, Fig 2.3.38). Due to the big dimension of the units, the number of flaps is usually more than three.

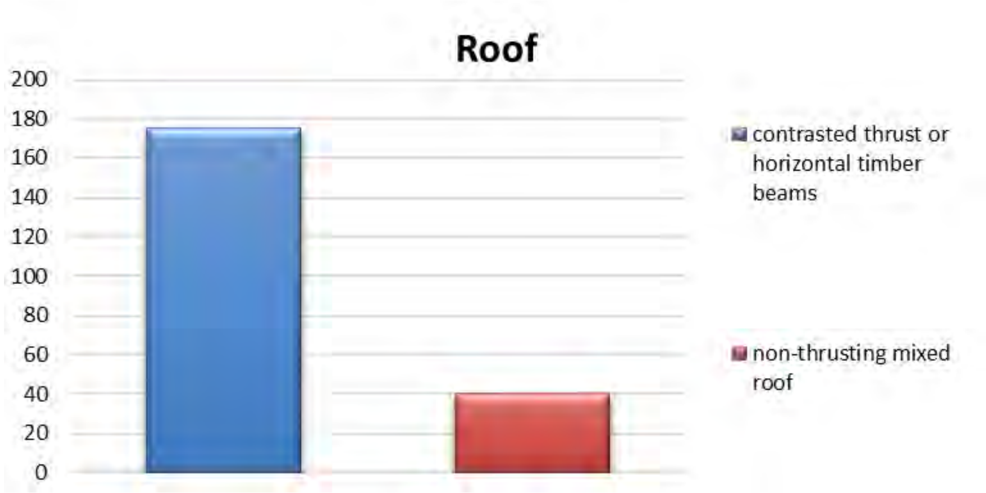


Fig. 2.3.37 Number of buildings for each type of roof

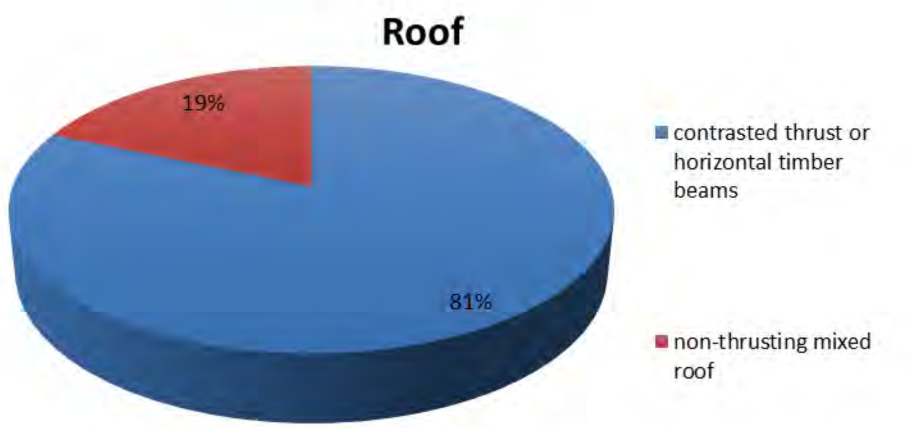


Fig. 2.3.38 Percentage of buildings for each type of roof

2.3.14 Joints

For most of the buildings, it was not possible to verify the quality of vertical structure joints. However, the experience of local architects showed a bad connection between perpendicular walls and the presence of juxtaposed walls to delimit two different units. In few cases the detachment of the façade was noted and it was difficult to establish the presence of active tie rods so in uncertain cases bad connections and absence of tie rods have been chosen, in order to represent the worst option.

The connection of the horizontal structures is based on literature and on experience of local architects because just in few cases it was possible to verify them. The definition of joint typology depends on detected horizontal structures. Vaults are considered deformable and well connected; timber and little vaults with iron beams are considered deformable and bad connected structures because of the connection detail with the vertical structure, as explained in the chapter 1.9.2. Concrete slab is considered a rigid and good connected structure, thanks to the presence of the reinforced concrete curb; there are no cases of rigid and bad connected horizontal structures. More than half of the buildings (57%) presents both bricks vaults and timber structure, so the identified connection are "deformable and good connected" for the fist and "deformable and bad connected" for the second. 3 units present only vaults, so are characterized by deformable and good connected horizontal structures (1%), seventeen structural units presents only concrete slabs (rigid and well connected structures - 8%) and other nine has both rigid and well connected and deformable and bad connected structures. Fifty-two units have only deformable and bad connected horizontal diaphragms (24%) and finally twelve buildings have both vaults and concrete slabs, so rigid and deformable horizontal elements,both good connected (6%) (Fig 2.3.39, Fig 2.3.40).

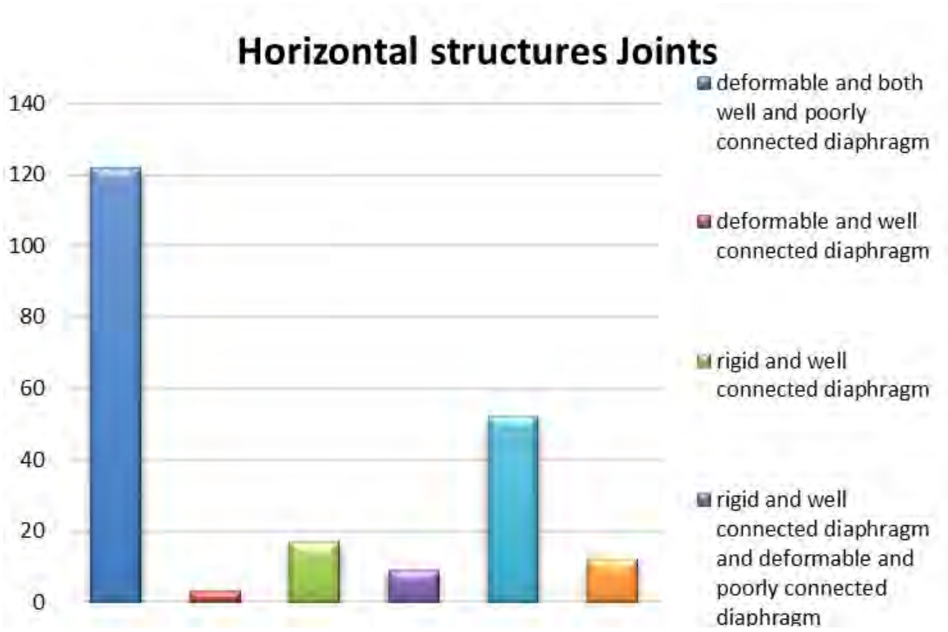


Fig. 2.3.39 Number of buildings for each type of horizontal structures joints

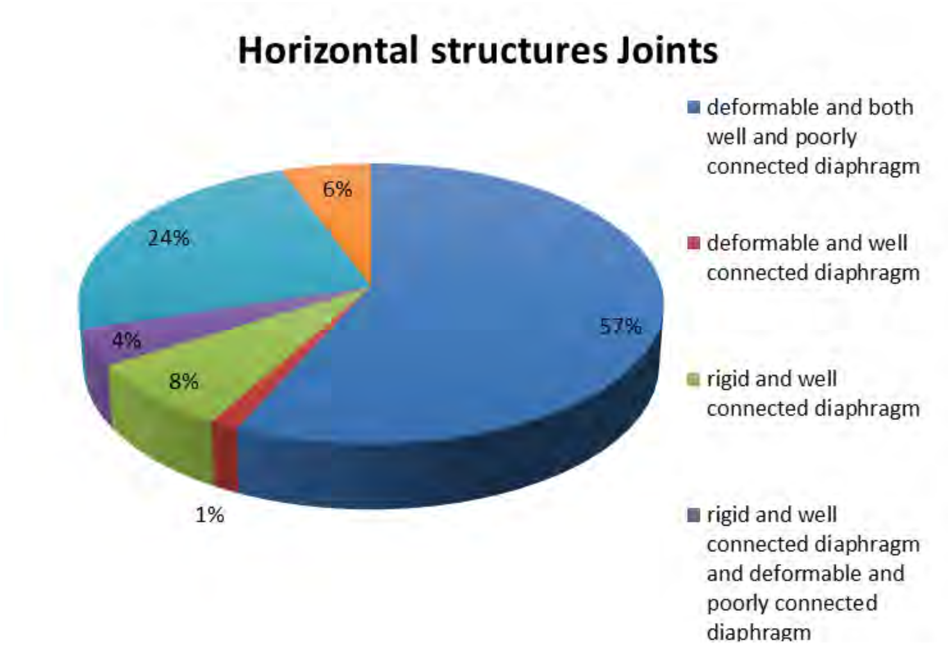


Fig. 2.3.40 Percentage of buildings for each type of horizontal structures joints

2.3.15 Regularity

Buildings were defined regular in plan or in elevation according to the definition given by Eurocode 8.[8] A building plan is classified as regular, if it fulfill the following conditions: the structure of the building plan must be approximately symmetrical respect to two orthogonal axes and it must be compact, so the protruding bodies does not exceed more than 5% the plan area[9]. Due to the impossibility of detecting all the parameters of the definition, it was considered regular if all vertical resistant systems extend for the entire height of the building and there was no towering elements or super elevations.

A lot of buildings of the historical center were destroyed and reconstructed during the Hasburg period and they present a regular composition. 67 of them are regular both in plan and in elevation (31%), while 105 are regular only in elevation (48%) and only 10 are regular just in elevation (5%). Finally 36 are shown no regularity at all (16%) (Fig 2.3.41, Fig 2.3.42).

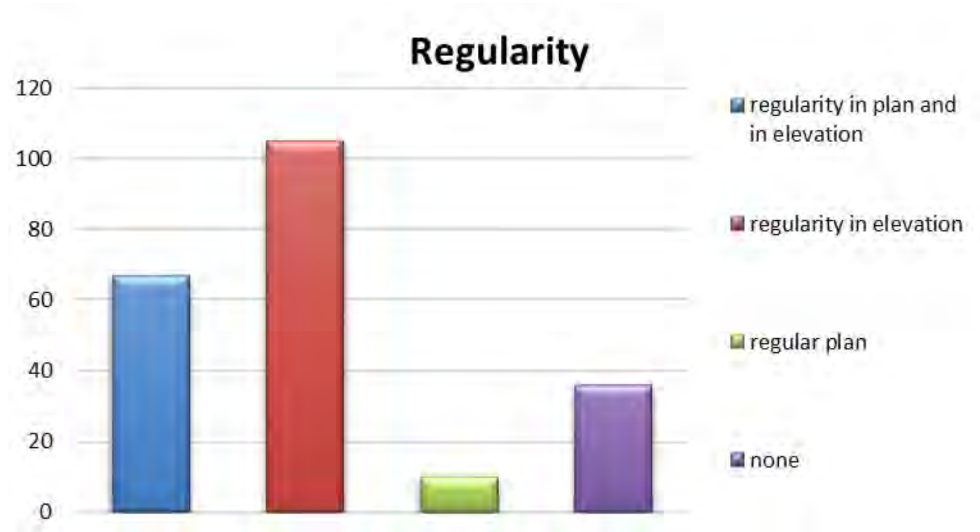


Fig. 2.3.41 Number of buildings for each type of regularity

[8] §4.2.3 *Criteri di regolarità strutturale* (Eurocode 8 – ENV 1998-1)

[9] §4.2.3.2 *Design of structures for earthquake resistance* (Eurocode 8 – ENV 1998-1)

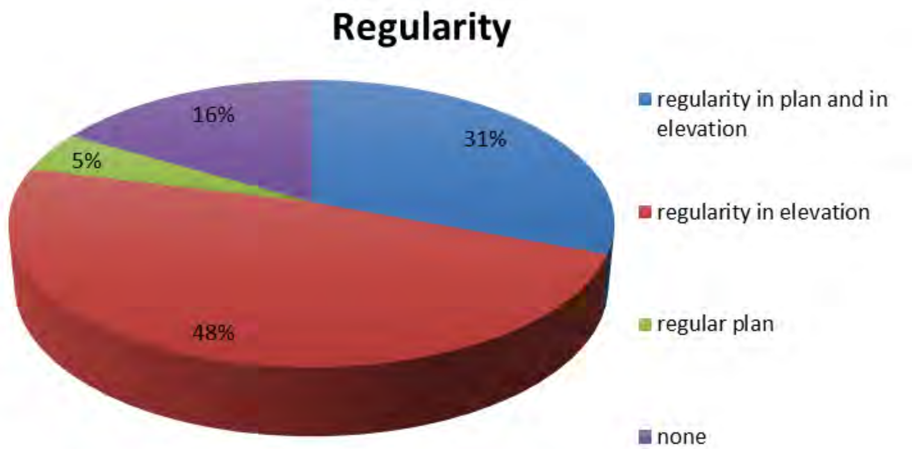


Fig. 2.3.42 Percentage of buildings for each type of regularity

2.3.16 Other regularity and vulnerability information

In the city of Timisoara almost all the buildings present one or more vulnerability elements, only ten of them have no relevant elements. The most common element is the vaulted passage, that together with the simple passage to the internal court, (both described in chapter 1.8.1) is present in 187 units (21%; 12%) and in 97 buildings added bodies were recognized (17%). All the mixed RC and timber roofs were catalogued as heavy roofs (8%) and in 51 structural units there were standing out elements. Another element recognized also by literature as one of the most dangerous during an earthquake is the fronton, an element that was added later during the building life. It usually has a bad connection to the original structure (9%). less frequent elements are loggias in the courtyard (4%) or external stairs (4%), and in 17 structural units there are not aligned holes, from the ground floor to the upper floors (Fig 2.3.43, Fig 2.3.44).

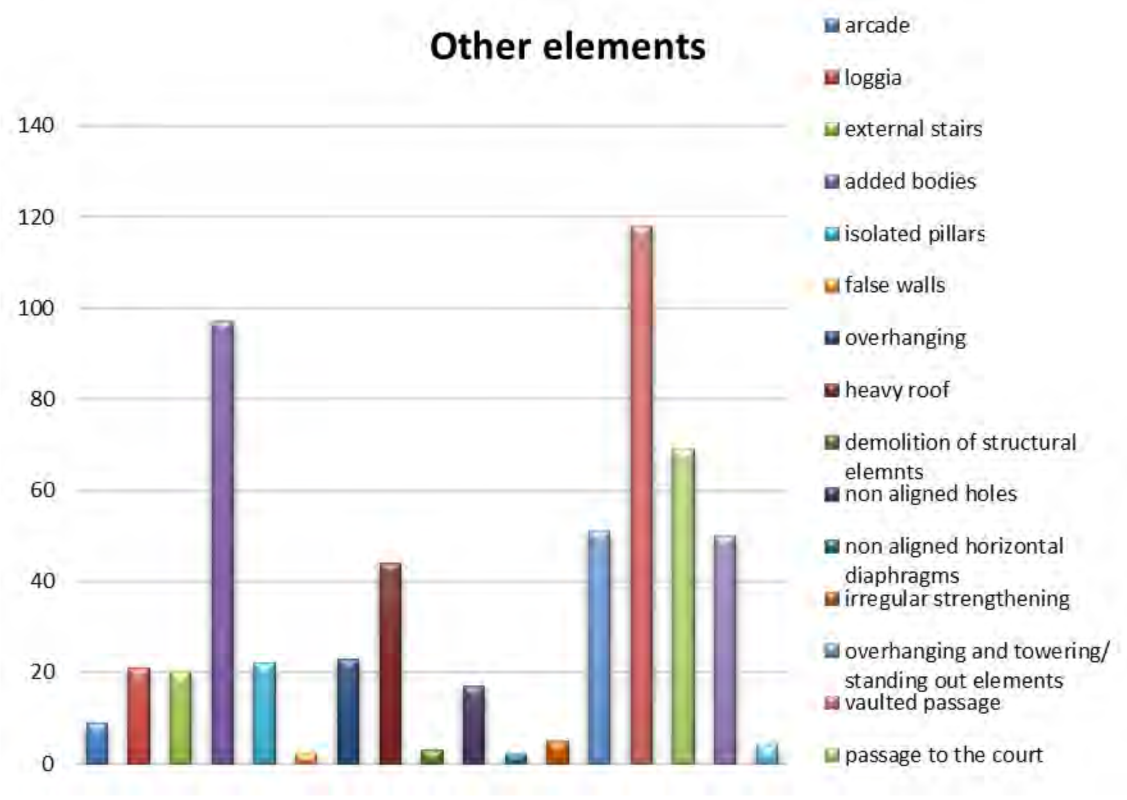


Fig. 2.3.43 Number of buildings in which each elements is present

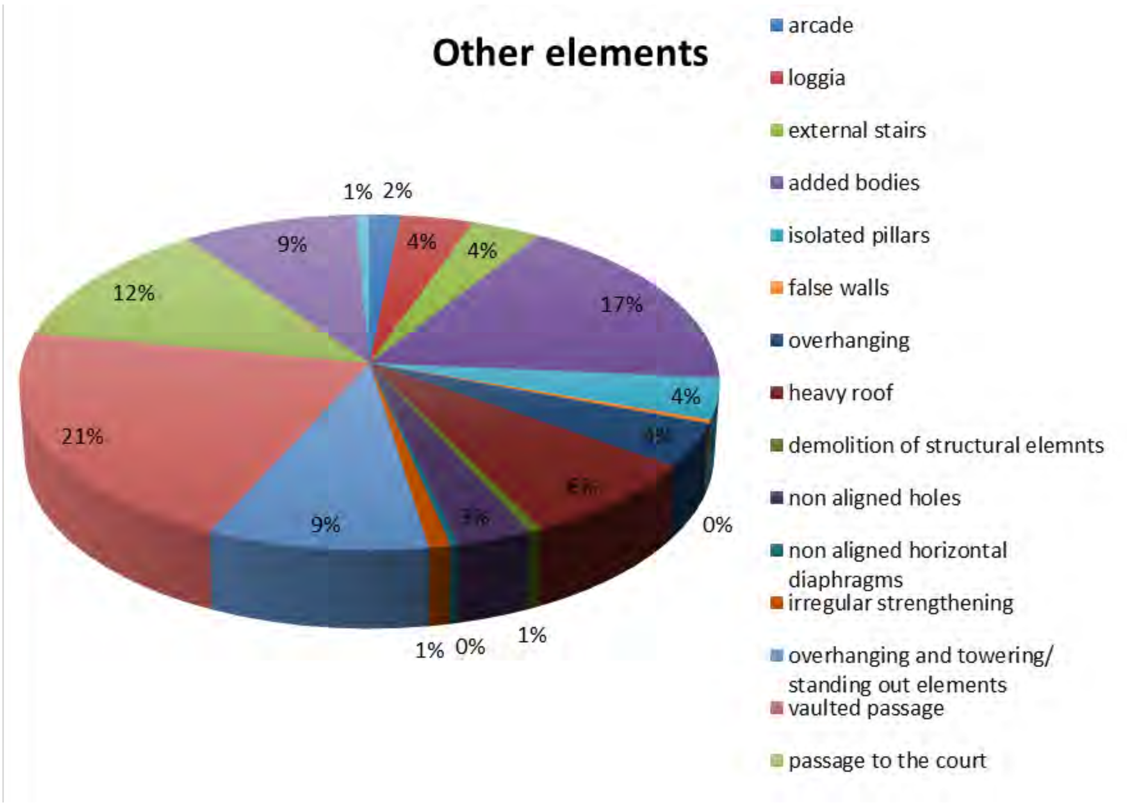


Fig. 2.3.44 Percentage of buildings in which each elements is present

Only 46 units are not characterized by soft storey, while the prevalence (88%) presents many holes or holes of large dimension at the ground floor and at the upper floors. 2% of buildings presents also the reduction of floor dimension in the upper floors. 14 units present large holes only at the ground floor (8%) and only few units present the combination of the vulnerability elements previously mentioned (Fig 2.3.45, Fig 2.3.46).

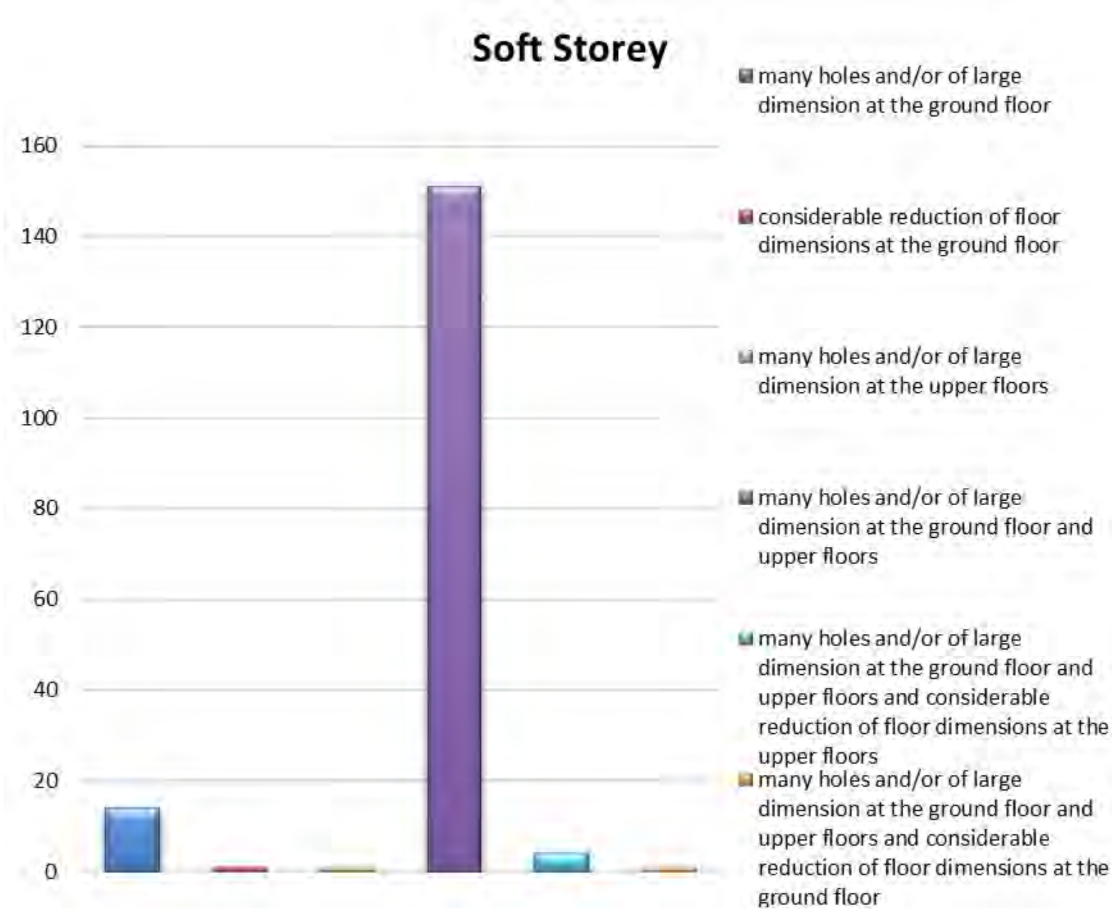


Fig. 2.3.45 Number of buildings for each type of soft storey

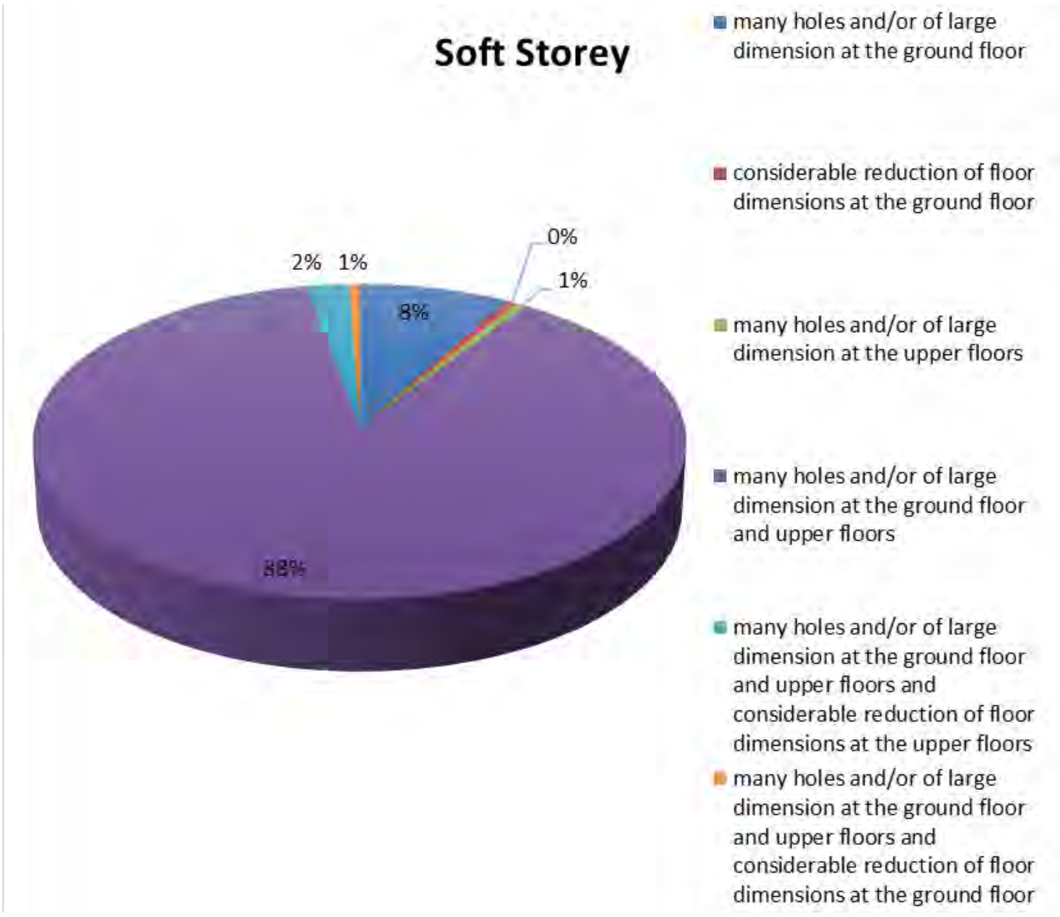


Fig. 2.3.46 Percentage of buildings for each type of soft storey

2.3.17 Reinforcing elements

Regarding reinforcing elements, 99 units have none (Fig 2.3.47, Fig 2.3.48), while the other may present one or more. 44 units present tie rods (activation uncertain) (30%), and in 38 cases there were signs of renovations or other ongoing interventions that has been catalogued as not identified reinforced masonry technique (26%). In 31 units the introduction of RC curbs was recognizable (21%). Other interventions, like the presence of buttresses, contrast elements, or confined openings, are noticed just in few cases.

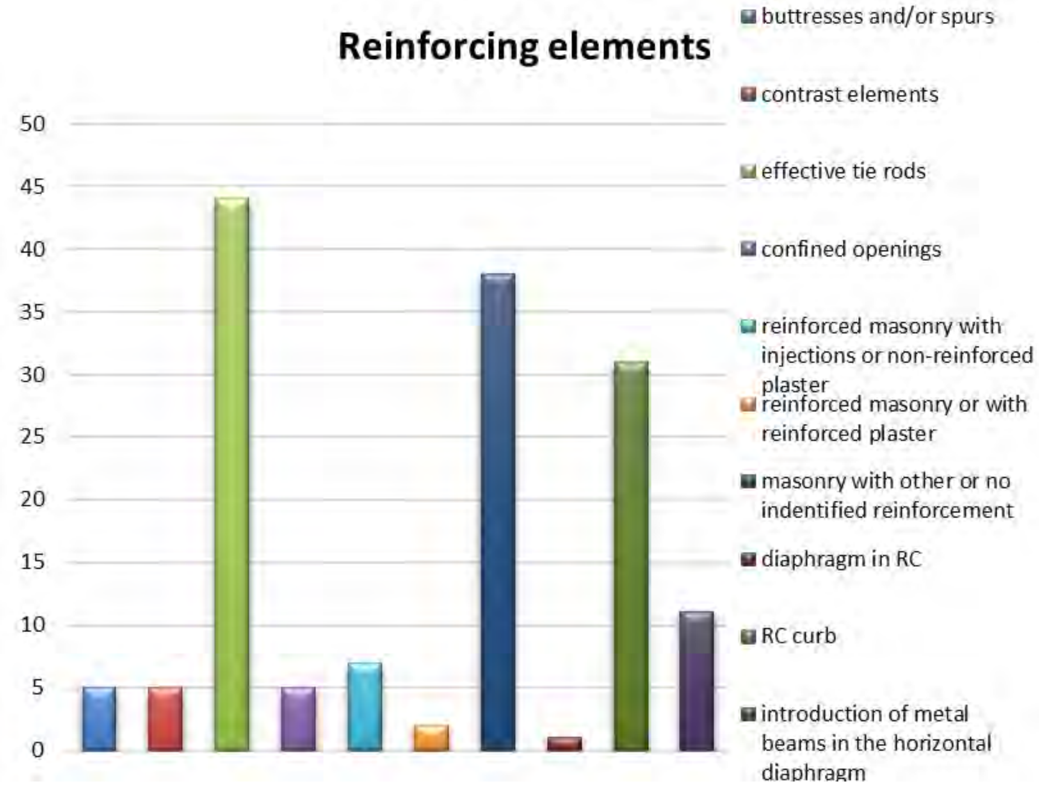


Fig. 2.3.47 Number of buildings for each reinforcing elements

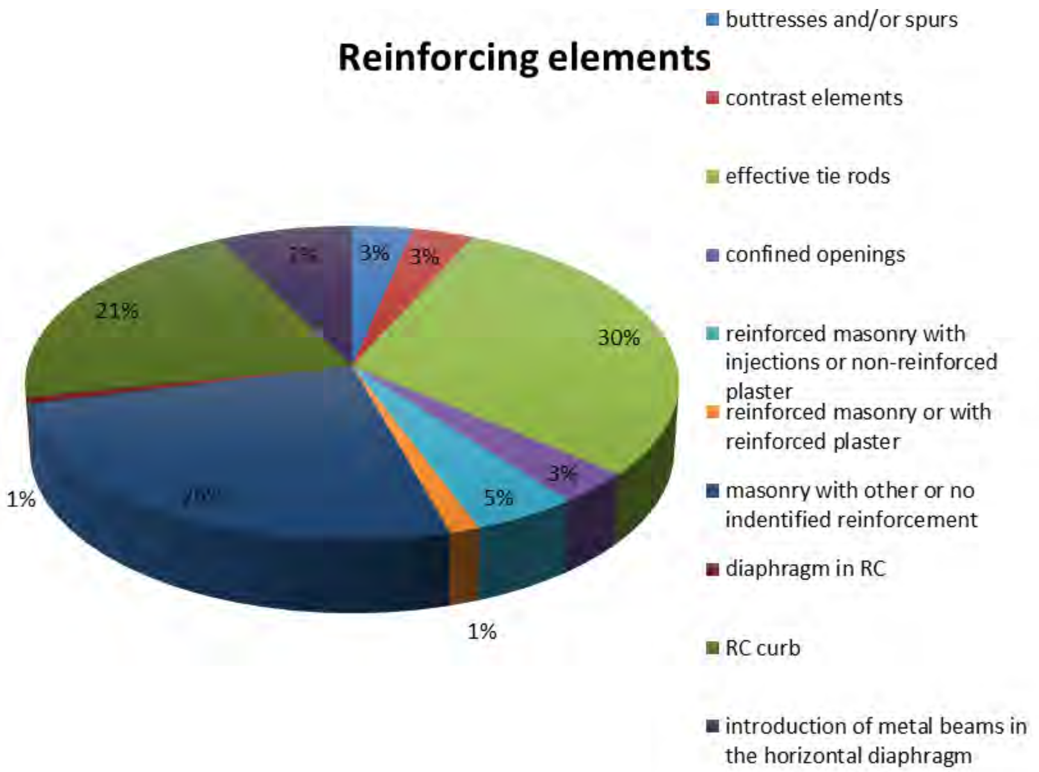


Fig. 2.3.48 Percentage of buildings for each reinforcing elements

2.3.18 Non-structural elements

In the city of Timisoara almost every building presents chimneys in bad conditions and frontons with bad connection so the 34% of buildings presents poorly connected big elements and the 41 % has crumbling decoration or signboard, considered as small and bad connected elements. Big or small well connected elements represents respectively the 15% and 10% (Fig 2.3.49, Fig 2.3.50).

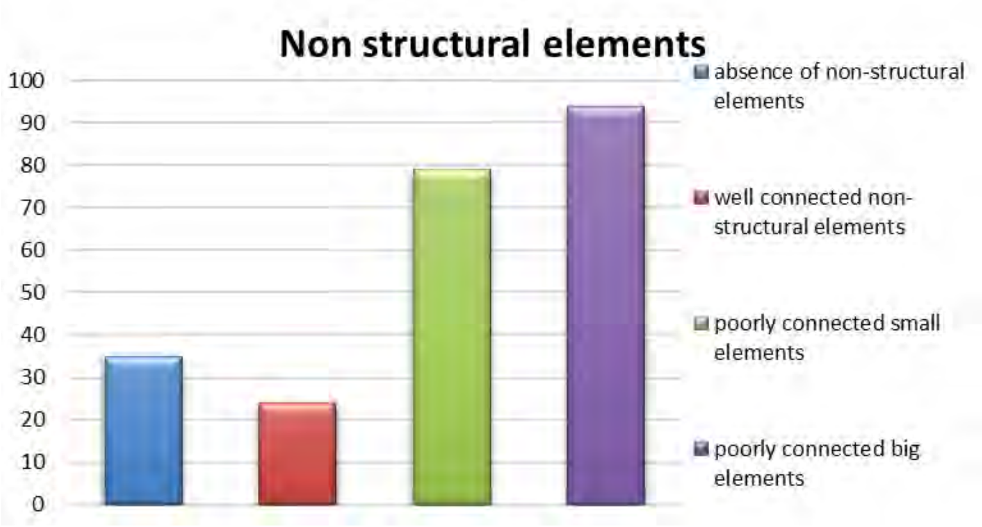


Fig. 2.3.49 Number of buildings for each case of non-structural elements

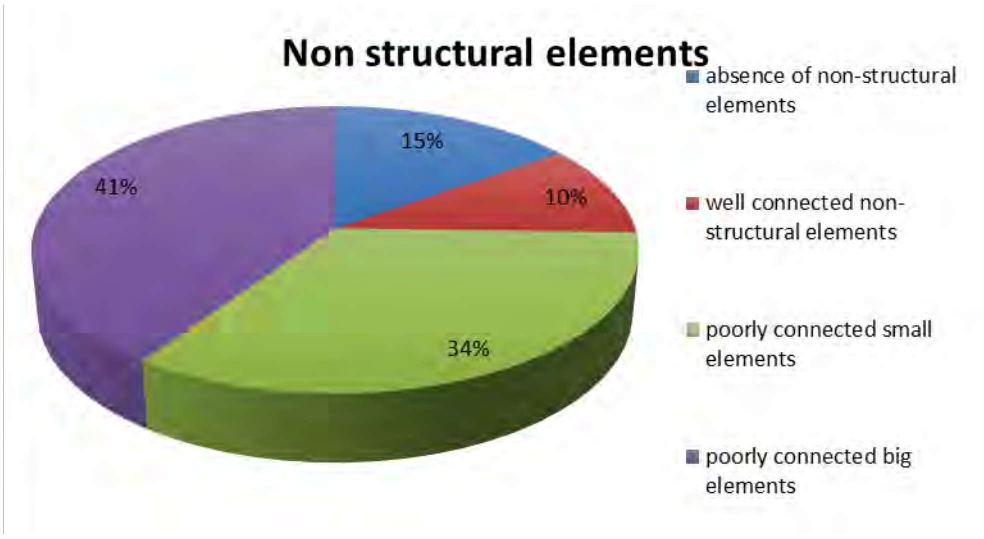


Fig. 2.3.50 Percentage of buildings for each case of non-structural elements

2.3.19 Status quo

Buildings with important serious damage are only 8 and they represent the 3% of the total. The 21% presents capillary cracks and initials structural problems, while for the 35% the detachment of plaster and little lesions were observed. Finally ninety-five units do not present any damage or structural problem (Fig 2.3.51, Fig 2.3.52).

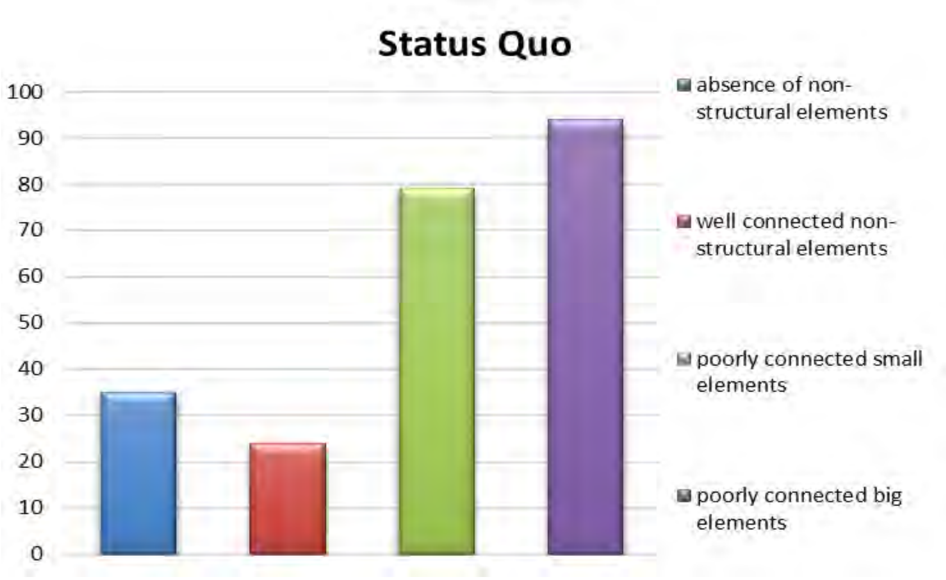


Fig. 2.3.51 Number of buildings for each status quo range

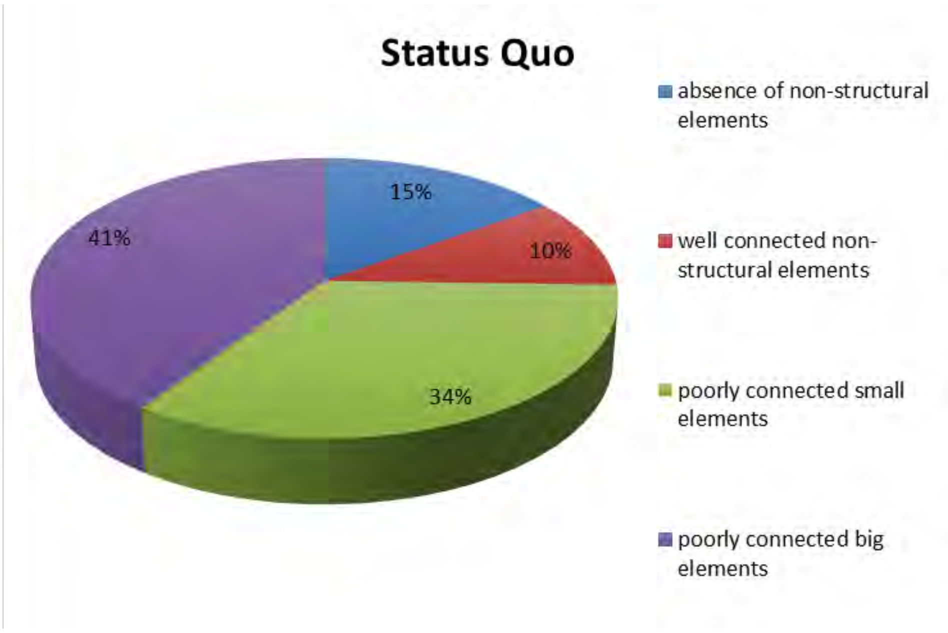


Fig. 2.3.52 Percentage of buildings for each status quo range

2.4 TYPOLOGIES IDENTIFICATION

The in situ survey aimed to identify the main characteristics of the buildings, such as horizontal and vertical structures, wall thickness, roof structure and interstorey high, to spot which ones are the most common in the city. The collected data were organized and statistically analyzed to group units with similar characteristics and structures. This process brought to typologies definition.

The study of the seismic behavior of a building typology brings to the definition of its vulnerability assessment. When the typology study is completed, it would be possible to define the vulnerability of a singular building in other parts of the city, identifying its most representative structural typology. Plus, the same results can be applied to other historical centers that show the same architectural and structural characteristics. The vulnerability assessment for building typologies is helpful in case of a rapid survey on large scale, when just an external or partially inside inspection is possible.

The final definition of eight macro typologies is the result of a three-step reduction. This subdivision was adopted to simplify the classification procedure, reducing building information to the main elements, especially regarding the prevalent vertical and horizontal structures and the roof typology. In some cases the lack of data makes the identification of the building typology quite difficult; for structural units in which data were missing, missing information have been assumed on the basis of statistical analysis and comparison with similar cases. Three typologies are called "unicum" because they included just few buildings, but are still considered in the analysis to have a complete vision on the studied area. Monumental buildings, such as Hospital, Castle, barracks and Court, were not included in the typological division and RC buildings are not considered.

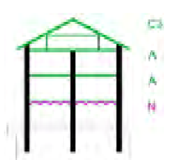
2.4.1 First step

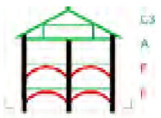
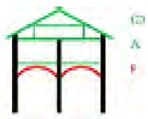
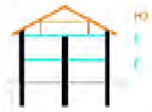
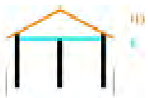
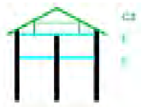
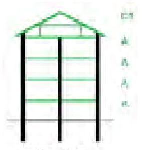
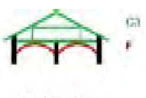
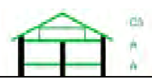
The first step defines 78 typologies, of which 11 are located in Iosefin. In the first phase the considered parameters are:

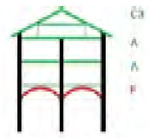
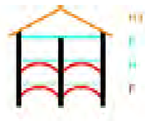
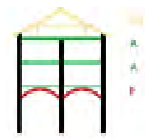
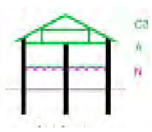
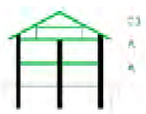
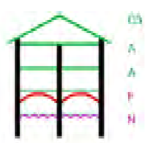
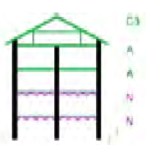
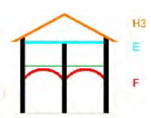
- vertical structures

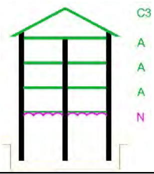
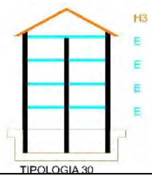
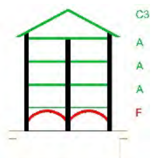
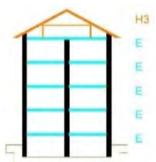
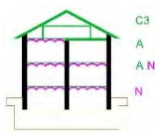
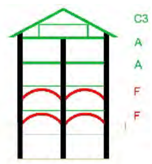
- horizontal structures
- roof
- stories number
- presence of basement of underground

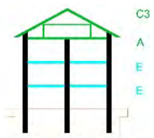
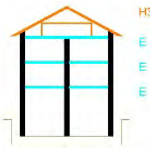
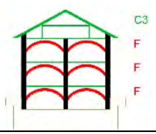
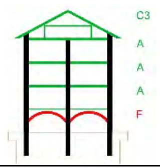
The considered vertical structure are masonry walls, masonry pillars and metal columns. Only US 3 has an RC diaphragm and it is considered as an “unicum”. The main horizontal structures are timber, concrete slab, vaults, metal beams with concrete slab, iron beam and little brick vaults, iron beam and very low brick vaults.
Considering the roof, only two different typologies are identified: the first one is the most widespread in the center and it is characterized by timber trusses on which wooden rafters settle. The second one is present in all the later interventions of roof renewal such as the introduction of a practicable attic and it is characterized by concrete and timber trusses connected by a concrete ring beam.

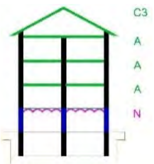
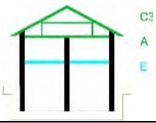
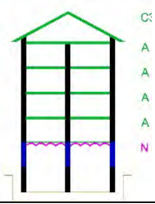
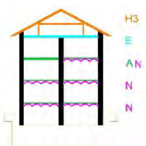
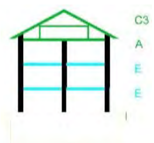
NUMBER TYPOLOGY	REPRESENTATION	VERTICAL STRUCTURE	HORIZONTAL STRUCTURE - UNDERGROUND	HORIZONTAL STRUCTURE - GROUND FLOOR	HORIZONTAL STRUCTURE - OTHER PLANS	ROOF STRUCTURE	STOREIS NUMBER
1		Masonry wall	Iron beams and little brick vaults	Vaults	Timber Concrete slab	Concrete and timber trusses	3+B
2		Masonry wall RC diagraphm	Iron beams and little brick vaults	Timber	Timber Timber	Concrete and timber trusses	3+B
3		Masonry wall	–	Iron beams and little brick vaults	Timber	Timber trusses	3

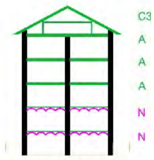
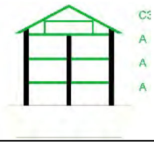
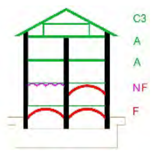
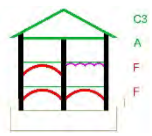
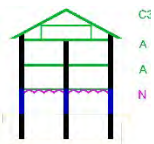
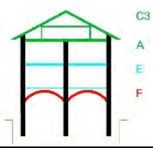
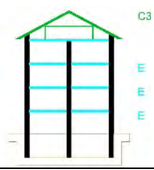
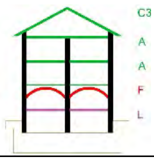
4		Masonry wall	Vaults	Vaults	Timber	Timber trusses	2+B
5		Masonry wall	_	Vaults	Timber	Timber trusses	2
6		Masonry wall	Vaults	Vaults	Timber	Timber trusses	3+BI
7		Masonry wall	_	Concrete slab	Concrete slab	Concrete and timber trusses	2
8		Masonry wall	_	Concrete slab	_	Concrete and timber trusses	1
9		Masonry wall	_	Concrete slab	Concrete slab	Timber trusses	2
10		Masonry wall	_	Timber	Timber	Timber trusses	4
11		Masonry wall	Vaults	Concrete slab	_	Concrete and timber trusses	1+B
12		Masonry wall	Iron beams and little brick vaults Vaults	Concrete slab	Concrete slab	Timber trusses	2+B
13		Masonry wall	_	Vaults	_	Timber trusses	1
14		Masonry wall	_	Timber Iron beams and little brick vaults	_	Timber trusses	1
15		Masonry	Iron beams and little brick	Timber	Timber	Timber trusses	2+B

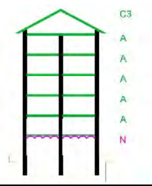
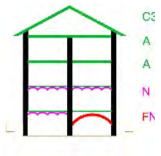
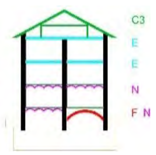
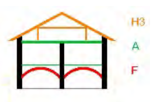
		wall	vaults Vaults				
16		Masonry wall	–	Vaults	Timber	Timber trusses	3
17		Masonry wall	Vaults	Vaults	Concrete slab	Concrete and timber trusses	2+B
18		Masonry wall	–	Vaults	Timber	Concrete and timber trusses	3
19		Masonry wall	–	Iron beams and little brick vaults	Timber	Timber trusses	2
20		Masonry wall	–	Timber	Timber	Timber trusses	2
21		Masonry wall	Iron beams and little brick vaults	Vaults	Timber	Timber trusses	3+B
22		Masonry wall	Iron beams and little brick vaults Vaults	Iron beams and little brick vaults	Timber	Timber trusses	2+B
23		Masonry wall	Iron beams and little brick vaults	Iron beams and little brick vaults	Timber	Timber trusses	3+B
24		Masonry pillars	–	Iron beams and little brick vaults	Timber	Timber trusses	3
25		Masonry wall	–	Vaults	Concrete slab	Concrete and timber trusses	2

26		Masonry wall	_	Vaults	Timber	Concrete and timber trusses	2
27		Masonry wall	_	Timber	Timber	Concrete and timber trusses	2
28		Metal reticular column	_	Iron beams and little brick vaults	Timber	Timber trusses	3
29		Masonry wall	_	Iron beams and little brick vaults	Timber	Timber trusses	4
30		Masonry wall	_	Concrete slab	Concrete slab	Concrete and timber trusses	4
31		Masonry wall	_	Vaults	Timber	Timber trusses	4
32		Masonry wall	_	Vaults	Timber	Timber trusses	2
33		Masonry wall	Concrete slab	Concrete slab	Concrete slab	Concrete and timber trusses	4+B
34		Masonry wall	Iron beams and little brick vaults	Iron beams and little brick vaults	Iron beams and little brick vaults Timber	Timber trusses	3+B
35		Masonry wall	Vaults	Vaults	Timber	Timber trusses	3+B

36		Masonry wall	Vaults	Vaults	Timber	Concrete and timber trusses	3+B
37		Masonry wall	Iron beams and little brick vaults	Concrete slab	Concrete slab	Timber trusses	3+B
38		Masonry wall	_	Concrete slab	Concrete slab Timber	Timber trusses	3
39		Masonry wall	_	Concrete slab	Concrete slab	Concrete and timber trusses	3
40		Masonry wall	_	Vaults	Concrete slab	Concrete and timber trusses	3
41		Masonry wall	_	Vaults	Timber	Concrete and timber trusses	2
42		Masonry wall	_	Vaults	Vaults	Concrete and timber trusses	2
43		Masonry wall	_	Vaults	Vaults Concrete slab	Concrete and timber trusses	3
44		Masonry wall		Vaults	Vaults	Timber trusses	2+B
45		Masonry wall	_	Vaults	Concrete slab	Concrete and timber trusses	2
46		Masonry wall	_	Vaults	Timber	Timber trusses	4
47		Masonry wall	_	Vaults	Concrete slab	Concrete and timber	3

					Timber	trusses	
48		Masonry pillars	–	Iron beams and little brick vaults	Timber	Timber trusses	4
49		Masonry wall	–	Concrete slab	Timber	Timber trusses	2
50		Masonry wall	–	Concrete slab	Concrete slab	Concrete and timber trusses	2
51		Masonry wall		Vaults	Vaults Timber	Concrete and timber trusses	3+B
52		Masonry wall	–	Timber	–	Timber trusses	1
53		Masonry wall	Vaults	Iron beams and little brick vaults	Timber	Timber trusses	3+B
54		Masonry pillars	–	Iron beams and little brick vaults	Timber	Timber trusses	5
55		Masonry wall	Iron beams and little brick vaults	Iron beams and little brick vaults	Iron beams and little brick vaults Timber Concrete slab	Concrete and timber trusses	3+B
56		Masonry wall	–	Concrete slab	Timber Concrete slab	Timber trusses	3

57		Masonry wall	Vaults	Timber	Timber	Timber trusses	3+B
58		Masonry wall	Iron beams and little brick vaults	Iron beams and little brick vaults	Timber	Timber trusses	4+B
59		Masonry wall	–	Timber	Timber	Timber trusses	3
60		Masonry wall	Vaults	Iron beams and little brick vaults Vaults	Timber	Timber trusses	3+B
61		Masonry wall	Vaults	Iron beams and little brick vaults Vaults	Timber	Timber trusses	2+B
62		Masonry pillars	–	Iron beams and little brick vaults	Timber	Timber trusses	3
63		Masonry wall	–	Vaults	Timber Concrete slab	Timber trusses	3
64		Masonry wall	–	Concrete slab	Concrete slab	Timber trusses	4
65		Masonry wall	Metal beams with concrete slab	Vaults	Timber	Timber trusses	3+B
66		Masonry wall	–	Vaults	Timber	Timber trusses	3

67		Masonry wall	–	Iron beams and little brick vaults	Timber	Timber trusses	6
68		Masonry wall	Iron beams and little brick vaults Vaults	Iron beams and little brick vaults	Timber	Timber trusses	3+B
69		Masonry wall	Iron beams and little brick vaults Vaults	Iron beams and little brick vaults	Concrete slab	Timber trusses	3+B
70		Masonry wall	Vaults and horizontal diaphragm	Iron beams and little brick vaults	Timber	Non-thrusting mixed roof	3+B
71		Masonry wall	Iron beams and little brick vaults	Timber	–	Timber trusses	1+B
72		Masonry wall	Vaults	Timber	–	Timber trusses	1+B
73		Masonry wall	Iron beams and little brick vaults Vaults	Concrete slab	–	Concrete and timber trusses	1+B
74		Masonry wall	–	Timber	–	Concrete and timber trusses	1
75		Masonry wall	Vaults	Timber	–	Concrete and timber trusses	1+B

76		Masonry wall	–	Iron beams and little brick vaults	Timber	Timber trusses	2
77		Masonry wall	–	Timber	–	Timber trusses	1
78		Masonry wall	Concrete slab	Concrete slab	–	Concrete and timber trusses	1+B

Tab. 2.4.1 Typologies identified in the first step of the procedure

2.4.2 Second step

The second step combines the horizontal structures considering only three categories: the ones with light structure, with heavy structures and with vaults. The vertical structure remains the same, cause almost all the typologies have masonry walls. Iron beam and little brick vaults and iron beams and very low brick vaults become light horizontal structures, while concrete slab and metal beams with concrete slab are considered as heavy horizontal structures. The vaults are considered separately due to their different weight. The subdivision in two classes, light and heavy, depends by the permanent loads of the structures: loads defined in the Vulnus manuals are taken as a reference. The roof maintains the division in two typologies, the timber trusses and the concrete and timber trusses structure but the initial division about the practicability of the attic is no more considered because it does not influence the structural behavior.

For the definition of the stories number, out of ground stories are considered. Buildings with underground were associated to the ones without, because the seismic behavior of the building interests out of ground stories, regardless of the presence of none or more floors underground. Buildings with basement are still divided from the previous ones for the same reason, because part of the basement is out of ground and so valuable for the seismic behavior.

Due to this simplifications, the number of typologies has been reduced from 78 to 33.

The typologies of this second step have been intersected with thickness of ground floor walls, the medium ground floor height and the medium interstorey height. walls thickness has defined considering the values that were observed in situ: they can be 45 cm, 60 cm, 75 cm, 90 cm of 105 cm thick. (See chapter 1.9.1)

The intersection between structure characteristics and wall thickness creates some micro-typologies, identified by a number and an alphabet letter.

MACROTYPOLOGY	VERTICAL STRUCTURE	HORIZONTAL STRUCTURE	ROOF	TYPOLOGY	MICRO-TYPOLOGY	STORIES NUMBER	US	GROUND FLOOR THICKNESS	AVERAGE INTERSTOREY HEIGHT
A	Masonry	Light horizontal structure and prevalently vaults at ground floor	Timber trusses	1	1A	1	36, 37	45	3,8
					1B		152, 241	60	
				2	2A	2	7, 44, 74, 75, 98	45	4
					2B		8, 18, 19, 20, 29, 30, 31, 55, 58, 59, 73, 77, 84, 97, 115, 120, 121, 151, 175, 193 202, 228, 301	60	
					2C		71, 80, 117, 145, 154	75	
					2D		94, 95, 167, 185	90	
				3	3A	3	49, 50, 51	45	4,3
					3B		4, 5, 53, 76, 96, 103, 104,105, 116, 118, 123, 124, 140, 174, 178, 192	60	
					3C		81, 109, 130, 134	75	
					3D		79, 86, 87, 88, 137, 138, 141, 166, 169B, 182, 183, 188, 189, 190	90	
				4	4B	4	25, 56, 133	60	4,2
					4C		129	75	
					4D		90, 92, 93, 128, 135, 155, 156, 187	90	
				5		5	157	90	4,8
				6		6	195	90	3,5

				7	7B	2+B	35, 38, 41, 61, 62, 64, 149, 150, 172, 211, 212, 213, 229, 230, 231, 233, 236, 237	60	4
					7C		68	75	
					7D		28, 100, 114, 217	90	
				8	8A	3+B	46	45	4,2
					8B		9, 24, 26, 43, 47, 54, 57, 60, 65, 168, 173, 184, 200, 201, 203, 207, 221, 234, 235	60	
					8C		69, 70, 143, 153	75	
					8D		101, 170	105, 90	
				9		4+B	169A	90	4,7
IOSEFIN				10	10A	1+B	224, 238, 239, 242	45	4,7
					10B		208, 209, 210, 215, 216, 218, 219, 220, 232	60	
				11		3+B	204	60	3,4
B	Metal reticular column	Light horizontal structure	Timber trusses	12		3	89, 127	60	4,9
				13		4	132, 144, 158	90	4,8
C	Masonry	Moderately heavy horizontal structures	Timber trusses	14		2	139	60	4
D	Masonry	Moderately heavy and light horizontal structures	Timber trusses	15		3	72	60	4,7
				16		3+B	181	105	4,4

E	Masonry	Light horizontal structure and prevalently vaults at ground floor	Concrete and timber trusses	17	17A	2	82	45	4,2
					17B		83, 85, 111, 186	60	
				18		3	52, 105,125, 131	60	4,3
				19		1+B	32	60	4,45
				20	20B	3+B	2	60	4
					20C		148	75	
					20D		102, 159	90	
IOSEFIN				21		1+B	225	45	4,5
F	Masonry and RC wall	Moderatly heavy and light horizontal structures	Concrete and timber trusses	22		3+B	3	60	4,1
G	Masonry	Moderatly heavy horizontal structures	Concrete and timber trusses	23		1	22	60	3,8
				24	24B	2	17, 21, 23, 63, 119	60	4
					24C		146	75	
				25	25A	3	107	45	3,6
					25B		108, 110, 164, 179	60	
				26	26B	4	91	60	3,7
					26D		180	90	
				27		2+B	48	45	3,7
				28		4+B	99	60	3,5
IOSEFIN				29		1	223	45	4,2
				30	30A	1+B	226, 227, 243	45	3,9
					30B		222, 245	60	
H	Masonry	Moderatly heavy and light horizontal structures	Concrete and timber trusses	31		2+B	34, 206	60	4
				32		3+B	106	45	3,6

IOSEFIN				33	3+B	205	60	3,8

Tab. 2.4.2 Macro typologies(A-H, typologies(1-33) and micro-typologies (1A, 1B, etc…) identified in the second step of the procedure

2.4.3 Third step

These micro-typologies have finally been grouped into eight macro-typologies. Some generalizations have been made in order to simplify the analysis. For each structural unit (US) only prevalent horizontal structures has been considered while, for the vertical ones, the structural units with masonry pillar at the ground floor have been associated to masonry walls. The macro-typologies do not consider stories number, wall thickness and interstorey height.

About roof typologies, the cases of timber trusses as roof structure and concrete slab as last horizontal structure have been united with the case of mixed and rigid roof, because the stiffen function of the concrete slab and its greater weight bring the roof structure to have a seismic behavior similar to mixed and rigid roof category.

The first macro typology, **A**, is the more widespread and includes 11 typologies, 2 of witch in Iosefin, with 157 structural units on a total of 242. Masonry walls, light horizontal structures, vaults at the ground floor and timber trusses roof characterize it.

In this macro-typology, buildings with one storey (Typology 1) characterized by an average interstorey height of 3.8 m and a wall thickness that can assume the values of 45 cm (micro-typology A) and 60 cm (micro-typology B are included).

Typology 2 includes buildings with two stories, with an average interstorey height for the ground floor of 4 m and 4.3 m for the upper floors, divided in four micro-typologies, A, B, C, D, for each wall thickness.

Typology 3 includes buildings with three stories, with an average interstorey height for the ground floor of 4.3 m and 4.2 m for the upper floors, divided in four micro-typologies, A, B, C, D, for each wall thickness.

Typology 4 includes buildings with four stories, with an average interstorey height for the ground floor of 4.2 m and 4 m for the upper floors, divided in three micro-typologies, B for the thickness of 60 cm, C for the 75 cm and D for the 90 cm.

Typologies 5 and 6 represent only two buildings of the city with 5 and 6 stories having both a wall thickness of 90 cm.

Typology 7 includes buildings with two stories and basement, with an average interstorey height for the ground floor of 4 m, 5 m for the upper floors and 1,3 m for the basement. It is divided into three micro-typologies, B for the thickness of 60 cm, C for the 75 cm and C for the 90 cm.

Typology 8 includes buildings with three stories and basement, with an average interstorey height for the ground floor of 4.2 m, 4.2 m for the upper floors and 1,1 m for the basement. It is divided into four micro-typologies, A, B, C, D, for each wall thickness.

Typology 9 is the last of the city center and contains a single unit, 169 A, with four stories and the basement, an average interstorey height of the ground floor of 4.7 m, 4.6 m for the upper floors and 1,6 m for the basement.

Typology 10 and 11 are present only in the district of Iosefin.

The second one, **B**, characterized by masonry walls above reticular column, light horizontal structures and timber trusses roof, is founded in few structural units and includes only two typologies, 12 and 13 of three and four stories.

The third, **C**, represent one of the three “unicum”. The unit 139 is the only building of typology 14, defined by masonry walls, moderate heavy horizontal structures and timber trusses roof, characterized by two out of ground stories, with a total height of 7.9 m and a thickness of 60 cm at the ground floor.

The second “unicum” is the fourth macro-typology, **D**, composed by two structural units and two typology, number 15 and 16, both with three out of ground stories but the second one also with the basement. Masonry walls, moderate heavy and light horizontal structures and timber trusses as roof characterize them.

The other four macro-typologies have in common the concrete and timber trusses roof.

The fifth macro typology, **E**, has masonry walls and light horizontal structure and prevalently vaults at the ground floor. It represents thirteen units and five typologies.

Typology 17 includes buildings with two stories, with an average interstorey height for the ground floor of 4.2 m and 4.7 m for the upper floors, divided into two micro-typologies, A for a thickness of the wall of 45 cm and B for a thickness of 60 cm.

Typology 18 includes buildings with three stories, with an average interstorey height for the ground floor of 4.3 m and for the upper floors 4.2 m, with a thickness of the ground floor of 60 cm.

Typology 19 represents the unit 32 with one storey and basement.

Typology 20 includes buildings with three stories and basement, with an average interstorey height for the ground floor of 4 m, 4,2 m for the upper floors and 1,4 m for the basement. It is divided into two micro-typologies, A for a thickness of the wall of 45 cm and B for a thickness of 60 cm.

Typology 21 has one storey and it is present only in the district of Iosefin.

The last “unicum” , macro-typology **F**, is characterized by the presence on a RC wall in the middle of the building and a concrete slab under the roof. It represent only one unit, number 3, defined by typology 22.

The seventh macro typology, **G**, is the more widespread between the ones with the concrete and timber trusses roof, its horizontal structures are moderately heavy and represents eleven typologies, two of them present only in Iosefin, with a global of twenty-two structural units.

Typology 23 represent the unit 22 with only one storey, while typologies 27 and 28 represent respectively units 48 and 99 with two and four stories respectively, both with basement.

Typology 24 includes buildings with two stories, with an average interstorey height for the ground floor of 4 m and for the upper floors 4.5 m, divided in two micro-typologies, B for a wall thickness of 60 cm and C for a thickness of 75 cm.

Typology 25 includes buildings with three stories, with an average interstorey height for the ground floor of 3.6 m and for the upper floors 3.3 m, divided in two micro-typologies, A for a wall thickness of 45 cm and B for a thickness of 60 cm.

Typology 26 includes buildings with four stories, with an average interstorey height for the ground floor of 3.7 m and for the upper floors 4 m, divided in two micro-typologies, B for a thickness of the wall of 60 cm and D for a thickness of 90 cm.
Finally the last macro typology, **H**, has moderate heavy and light horizontal structures, and defined three typologies 31, 32, and 33. The last one is present just in Iosefin, with five structural units.

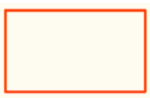
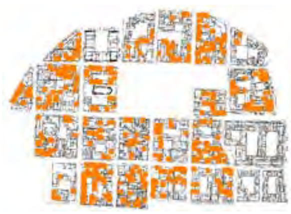
2.4.4 Plan module

During the on-site activities a plan of almost the entire city center has been found.[10] This map shows the ground floor of all the aggregates at north of the tram line in the historical center and it was hand-draw around 1980. The quality of the plan is not very detailed, in particular about the wall thickness and the windows and doors dimension, but the representation of internal rooms and walls allows the analysis of internal spaces to define recurrent plan modules. One module must cover the entire room area and its sides must overlap the internal walls. The first modules were defined for the first unit and then copied in the following unit where the measures coincide. If a room was not represented by a module of the previous unit, a new module was introduced. This procedure was repeated for all the represented aggregates, excluding monumental buildings, due to the fact that their particular characteristics exclude them from ordinary constructions. In some cases, if the room has huge dimensions or a particular shape, more modules combined were used to describe it or new "unicum" modules were introduced.
In few cases it was possible to observe that the plan map does not correspond exactly to the real disposition of internal rooms, due to possible interventions on the building in the latest years, but it is still a precious instrument to study the typical internal structure of buildings.

[10] Material provided by Arch. Bogdan Demetrescu

Fig.2.4.1: Block 23represented in the map (left) and after the module analysis (right)

modules for internal rooms are:

MODULE	SHAPE	MEASURES [m]	QUANTITY	SPREAD
1		12,20 x 4,60	39	
2		10,30 x 5,80	77	
3		5,8 x 4,10	424	

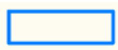
4		5,15 x 2,9	384	
5		10,10 x 8,70	3	
6		6,30 x 5,80	142	
7		4,15 x 3,80	312	
8		6,75 x 3,40	85	
9		4,86 x 4,15	121	
10		3,55 x 2,3	262	

11		2,5 x 1,35	19	
12		26 x 4,80	5	
13		14,90 x 4,30	8	
14		8,85 x 3,60	19	
15		8,05 x 5,00	23	

Tab. 2.4.3: Room modules

The analysis shows that the modules having the biggest dimensions are less common than the ones with small dimensions. In fact the most common module is Module 3, followed by Module 4 and Module 7. Modules tend to be bigger near the façade and to become smaller near the internal courtyard.

A different type of modules has been set up for the under-crossing to internal courtyard. Just five modules are representative of this element and with one module or a combination of more than one it is possible to represent the entire casuistry:

MODULE	SHAPE	MEASURES [m]	QUANTITY	SPREAD
A		17,60 x 5,45	5	
B		13,25 x 3,7	16	
C		9,65 x 2,85	12	
D		6,15 x 2,65	102	
E		5,85 x 1,45	44	

Tab. 2.4.4: Passage to the courtyard modules

The most common modules are D and E ones and as in the case of room modules they are the smallest ones. Module D in particular is the most widespread and it often appears as two modules one above the other.

Room modules and crossing modules have been combined to search for a common pattern or scheme to refer to building typologies, but even if there are some combinations that appears more often than others, no evident outline has been found. Anyway, the most common combinations are:

- Module D flanked by Modules 2, 3, 6 and/or 9;
- Module E flanked by Modules 2, 3, 6 and/or 9;
- Module B flanked by Modules 2 and/or 6.

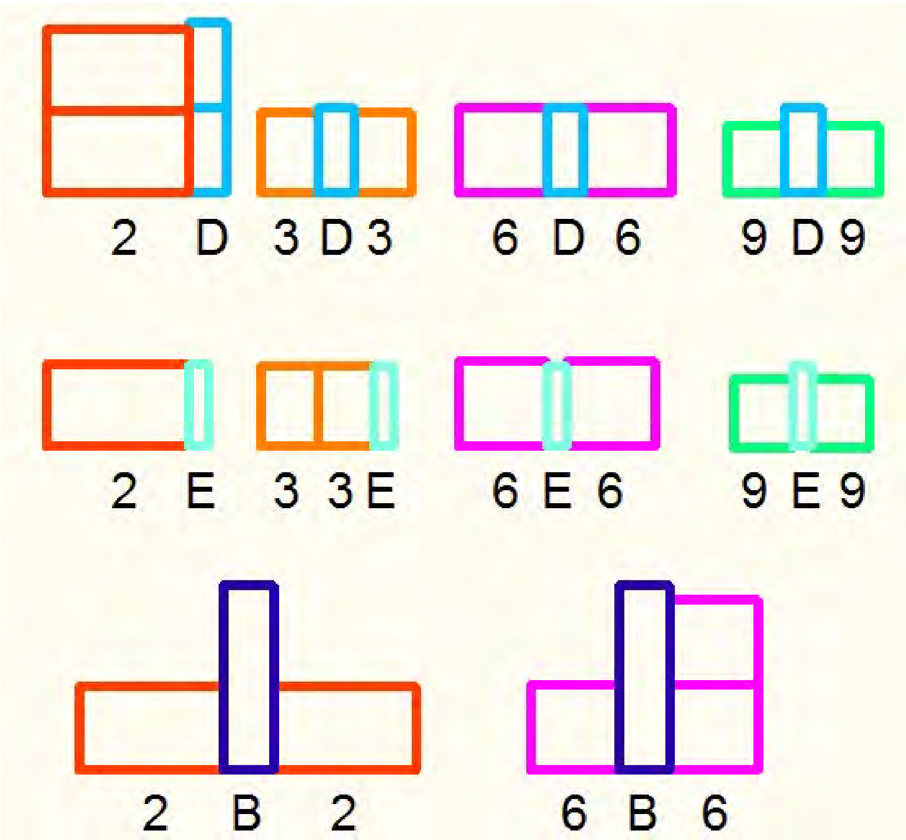


Fig. 2.4.1: Most common combination of room and crossing modules

2.4.5 Façade module

The buildings façades of Timisoara are characterized by a strong symmetry and a rhythmic pattern of the same module. The definition of a typological profile is necessary to analyze the in plane vulnerability of the buildings (see chapter 5.6). The considered units are the corner ones, because they are the more exposed to the in plane damage.

The characterization of the façade typologies considers the corner units. Each structural unit belongs to a micro-typology defined in chapter 2.4.1, in which the ground floor and upper floor interstorey height, together with their maximum and minimum values, has been defined. These height are the same used to define the typological facades in this chapter.

The 76 units considered have been divided in two big groups characterized by two roof typologies: the mixed roof with concrete and timber trusses and the roof with only timber trusses. This characteristic influences the weight that insist on the analyzed wall in the mechanism studied in chapter 5.6.

The units of the two roof groups have been further subdivided in relation of their stories number and their initial typology was recognized, as can be seen in Tab. 2.4.5. As a result of this first division, 4 classes have been defined for the mixed roof and 5 classes for the timber roof. In classes III, IV, V, VI and VIII, all the units in the same class belong to the same building typology, defined in chapter 2.4, and their interstorey height and its variation are the same already defined in Tab. 2.4.2. In classes I, II, VII and IX instead, the units belong to different initial typologies so in the second step of the identification process, the values of the interstorey height are the average ones of the initial typologies values.

ROOF TYPE	CLASSES	US	STORIES NUMBER	TYPOLOGY	INTERSTOREY HEIGHT GROUND FLOOR [m]	VARIATION [m]	INTERSTOREY HEIGHT UPPER FLOOR [m]	VARIATION [m]	BASEMENT [m]
CONCRETE AND TIMBER TRUSSES	I	21, 119	2	24	4	±0,5	4,5	±1	-
		82, 186		17	4,2	±0,5	4,7	±1	
	II	52, 125, 190	3	18	4,3	±0,5	4,2	±0,5	-
		108		25	3,6	±0,5	3,3	±0,5	
	III	91	4	26	3,7	-	4	±0,5	-
	IV	99	4+SI	28	3,5	-	3,5	-	1,5
TIMBER TRUSSES	V	7, 8, 18, 30, 55, 71, 74, 115, 117, 151, 154, 167, 185	2	2	4	±0,5	4,3	±0,5	-
	VI	28,35, 41, 62, 64, 68, 149, 150	2+B	7	4	±0,5	5	±0,5	1,3
	VII	4, 5, 42, 49, 50, 51, 53, 76, 79, 81, 86, 96, 103, 109, 130, 137, 138, 141, 166, 169B, 174, 178, 192	3	3	4,3	±1	4,2	±0,5	-
		89, 127		12	4,9	-	5	-	-
		105		18	4,3	±0,5	4,2	±0,5	-
	VIII	24, 26, 43, 46, 47, 54, 57, 60, 101, 153 168, 170	3+B	8	4,2	±0,5	4,4	±0,5	1,1
	IX	93, 128, 133	4	4	4,2	±0,5	4	±0,5	-
		132, 144, 158		13	4,8	±0,5	4,2	-	-

Tab. 2.4.5: Considered structural units

All the pictures of the building facade were processed through the perspective rectification carried out by RDF program and they have been analyzed to define the medium dimension of the windows openings and their position. In the following figures is shown the perspective rectification.

Fig. 2.4.2: Perspective rectification of US 30 and US 86

For each typology a large number of photos was straighten: the windows dimensions were compared and a medium value was determined. The French door were always located in correspondence of the floor, while the smaller windows were located at 1 m from the floor. All the classes are characterized by only one façade type, and consequently by one of these window positions, and only the class V has both possibilities: the windows located 1 m from the floor and the one located in correspondence of the floor.

With the same process, the length of the span was evaluated referring to the available pictures of the buildings and to the facades length of the structural units measured in the dwg files. With the picture rectification a first span dimension has been defined, following the windows pattern. This dimension has been later compared and adjusted with the plan modules defined in chapter 2.4 for each class. All the classes are indeed subdivided in micro-classes, according to the spans number and consequently to the façade length.

The characteristics considered to describe the facades can be seen in Fig.2.4.2 and are:

- Last storey height (h_{TOT}): it is the height of the last storey, defined from Tab. 2.4.5 as the interstorey height of upper floors;

- Height from the ground (Z): it is the sum of the interstorey height of the ground floor, the upper floors and the basement (if present) until the last floor. For example, it can be the height of the ground floor, for a 2 stories building. The position of the windows must be considered too and if the window is located at 1 m from the floor, this meter must be added to the highness of the floors below;
- Windows dimensions (bxh): the dimension of the windows is based on the facades analysis carried out with the rectification process. The dimensions are the average values of the analyzed cases for each class;
- Span length (b_{bay}): it is defined for each class referring to the most appropriate plan module.

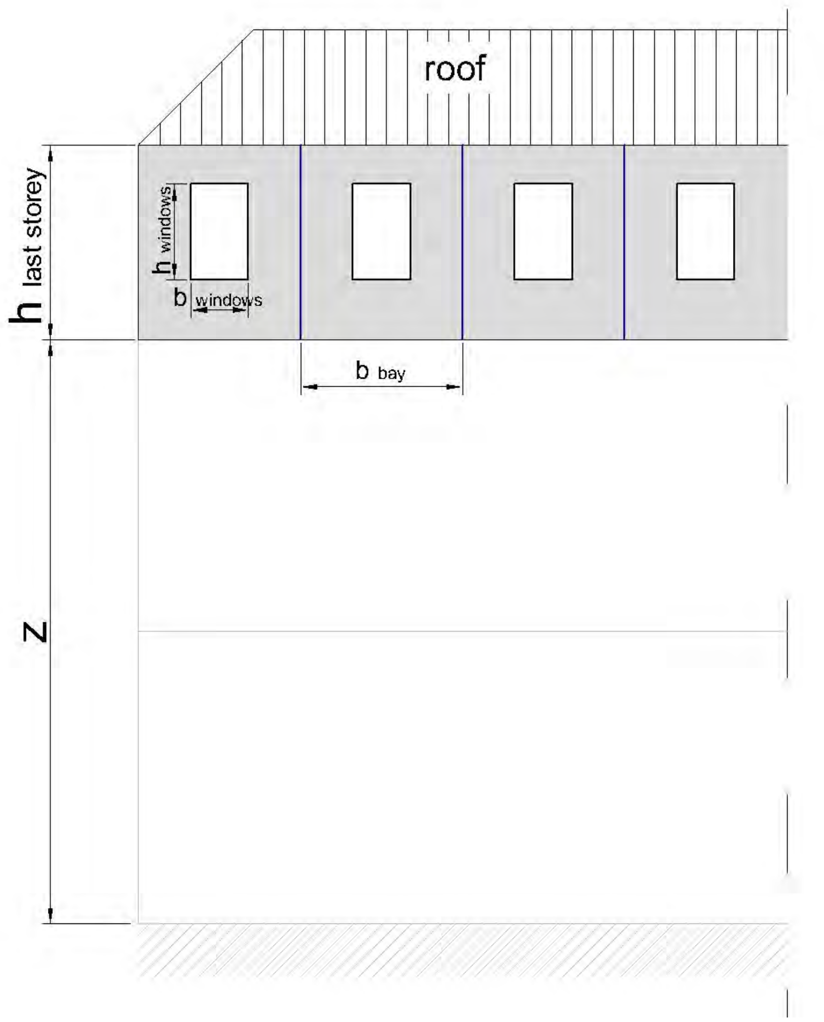


Fig. 2.4.3: Identification of the façade characteristics

The Tab. 2.4.6 resumes the analyzed classes and micro-classes characteristics. Classes III and IV are not considered in the analysis because they includes only one structural unit.

The buildings characterized by concrete and timber trusses are divided in two classes according to the stories number. It total this class includes 11 units.

The I class includes buildings with two stories and all the structural units belong to typologies 17 and 24. The dimensions that characterize this class are:

- Last plan height: 4.6 m;
- Height from the ground: 4.1 m;
- Windows dimensions (bxh): 1.15 x 2.9 m;
- Span length (connected with plan module 7): 3.8 m.

The micro-classes *a, b, c* and *d* correspond to 2, 5, 7 and 11 spans respectively.

The II class represents the buildings with three stories which belong to typologies 18 and 25. The dimensions that characterized this class are:

- Last plan height: 2.75 m;
- Height from the ground: 8.7 m;
- Windows dimensions (bxh): 1.15 x 2 m;
- Span length (connected with plan module 4): 2.9 m.

The micro-classes *a, b, c*, *d* and *e* correspond to 3, 6, 8, 10 and 13 bays.

The group characterized by timber trusses is composed by 5 classes and includes 65 structural units.

The V class represents the buildings with two stories which all belong to typology 2. The dimensions that characterized this class are:

- Last plan height: 4.3 m;
- Height from the ground: 4 m;
- Windows dimensions (bxh): 1.3 x 2.3 m;
- Span length (connected with plan module 7): 3.8m.

The micro-classes *a, b, c* and *d* correspond to 5, 7, 9 and 11 bays.

The VI class represents the buildings with two stories and basement, which all belong to typology 7. The dimensions that characterized this class are:

- Last plan height: 5 m;
- Height from the ground: 5.3 m;

- Windows dimensions (bxh): 1 x 2.8 m;
- Span length (connected with plan module 7): 3.8m.

The micro-classes *a, b* and *c* correspond to 5, 7 and 9 bays.

The VII class represents the buildings with three stories, which majority belongs to typology 2 and only two buildings belong to typology 12. The dimensions that characterized this class are:

- Last plan height: 3.2 m;
- Height from the ground: 10.5 m;
- Windows dimensions (bxh): 1.3 x 2 m;
- Span length (connected with plan module 8): 3.4 m.

The micro-classes *a, b, c*, *d* and *e* correspond to 4, 6, 8, 9 and 11 bays.

The VIII class represents the buildings with three stories and basement, which all belong to typology 8. The dimensions that characterized this class are:

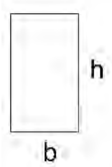
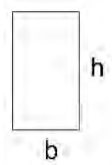
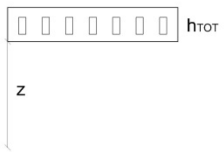
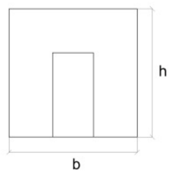
- Last plan height: 3.4 m;
- Height from the ground: 10.7 m;
- Windows dimensions (bxh): 1.1 x 2 m;
- Span length (connected with half plan module 4): 2.575 m.

The micro-classes *a, b* and *c* correspond to 8, 12 and 15 bays.

The IX class represents the buildings with four stories, which all belong to typologies 4 and 13. The dimensions that characterized this class are:

- Last plan height: 4.1 m;
- Height from the ground: 16.8 m;
- Windows dimensions (bxh): 1.2 x 2.2 m;
- Span length (connected with plan module 3): 4.1 m.

The micro-classes *a, b* and *c* correspond to 5, 7 and 10 bays.

ROOF	CLASSES	MICRO-CLASSES	SPANS NUMBER	STORIES NUMBER	WINDOW DIMENSIONS (bxh) [m]	MODULE (bxh) [m]	FACADE DIMENSIONS (h_{TOT};z) [m]		SPAN (bxh) [m]
							h_{TOT}	z	
									
CONCRETE AND TIMBER TRUSSES	I	a	2	2	1,15x2,9	7 (3,8x4,15)	4,6 ±1	4,1 ±0,5	3,8x4,6
		b	5						
		c	7						
		d	11						
	II	a	3	3	1,15x2	4 (2,9x5,15)	3,75 ±1	8,7 ±0,5	2,9x2,75
		b	6						
		c	8						
		d	10						
		e	13						
TIMBER TRUSSES	V	a	5	2	1,3x2,3	7 (3,8x4,15)	4,3 ±0,5	4 ±0,5	3,8x4,3
		b	7						
		c	9						
		d	11						
	VI	a	5	2+B	1x2,8	7 (3,8x4,15)	5 ±0,5	5,3 ±0,5	3,8x5
		b	7						
		c	9						
	VII	a	4	3	1,3x2	8 (3,4x6,75)	4,2 ±0,5	10,5 ±1	3,4x3,2
		b	6						
		c	8						
		d	9						
		e	11						
	VIII	a	8	3+B	1,1x2	4 (5,15x2,9)	4,4 ±0,5	10,7 ±1	2,575x3,4
		b	12						
		c	15						
	IX	a	5	4	1,2x2,2	3 (4,1x5,8)	4,1 ±0,5	16,8 ±1	4,1x3,1
		b	7						
		c	10						

Tab. 2.4.6:In plane classes

3 CODES

3.1 EVOLUTION OF ROMANIAN DESIGN CODES[1]

The major earthquake events with large impact on human life caused different modifications of technical provisions for buildings seismic design. After the major earthquake of Vrancea in 1940, a first reference regarding a seismic design code is done in the Romanian design guideline “Temporary instructions for preventing the deterioration of buildings due to earthquakes and restoration of the degraded ones” from 1941. In the seismic zoning map of 1952, that was the first established for Romania, Timisoara was considered a very low seismic risk zone.

In the 1963 the design code has been modified and Timisoara has been included in a zone having a degree of intensity VI on the MSK scale, while the southeast of the country maintained more or less the same zonation and degree.

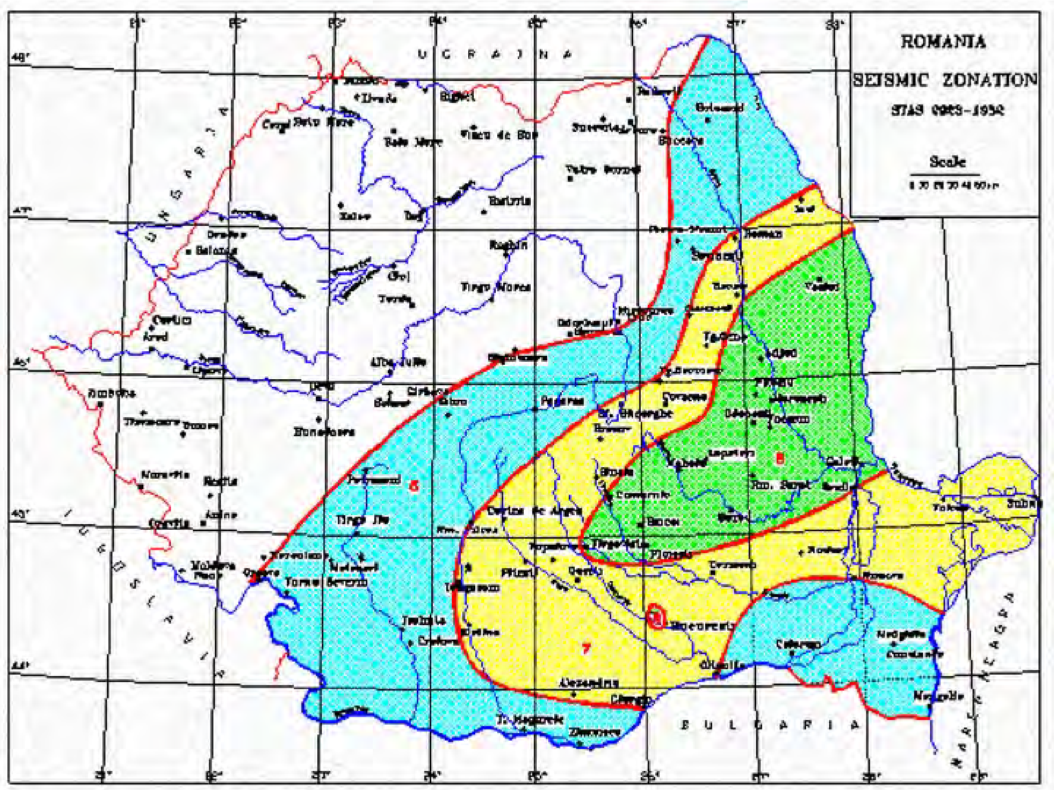


Fig.3.1.1: STAS 2923-52 - Seismic zonation in 1952 in terms of degrees in MSK scale.

REFERENCE: STAS 2923-52, Macrozonarea teritoriului R. P. Romane (Inforix)

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[1] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarcaet al., 2014)

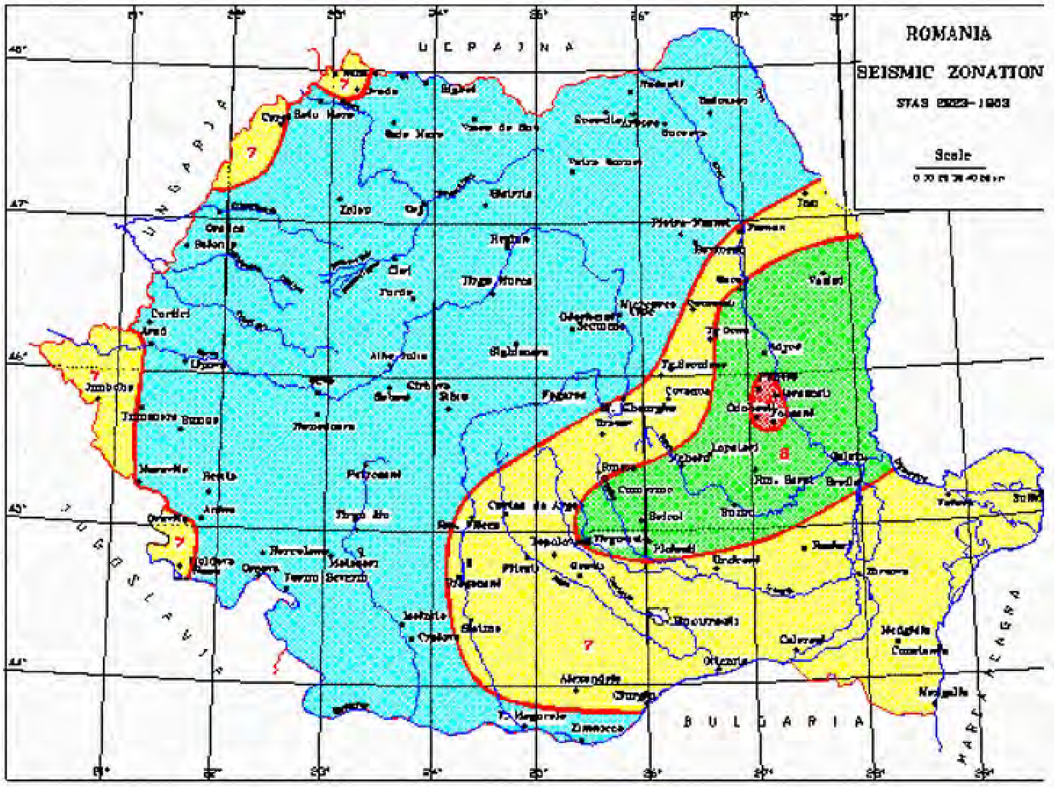

Fig.3.1.2: STAS 2923-63 - Seismic zonation in 1963 in terms of degrees in MSK scale.
REFERENCE: STAS 2923-63, Macrozonarea teritoriului R. P. Romane (Inforix)

Unfortunately, between 1963 and 1977, "a lot of buildings from Timisoara were designed according to a degree of intensity 7 on the MSK scale"[2]. In the design code **P13-63**, calculation of the seismic action was based on a response spectrum for crustal earthquakes, having a control period of Tc = 0.3 sec, let to 0.4 sec with **P13-70**.

After the Vrancea earthquake of 1977, in design codes **P100-78** and **P100-81** the control period value was modified to Tc=1.5 sec and the ductility rules for RC shear wall & frame structures were introduced. "In order to better cover the response of crustal earthquakes from the Banat region, a corner period of Tc = 0.3 and 0.4 sec was implemented."[3]

[2] *Seismic risk of buildings with RC frames and masonry infills from Timisoara, Banat region, Romania* (Mosoarca et al., 2014, p.5)

[3] *Ibidem*

After the 1977 event, new ductility rules for RC structures were imported from US practice and incorporated into Romanian seismic codes P100 and design rules were significantly improved after 1989, according to the EUROCODE 8 requirements[4].

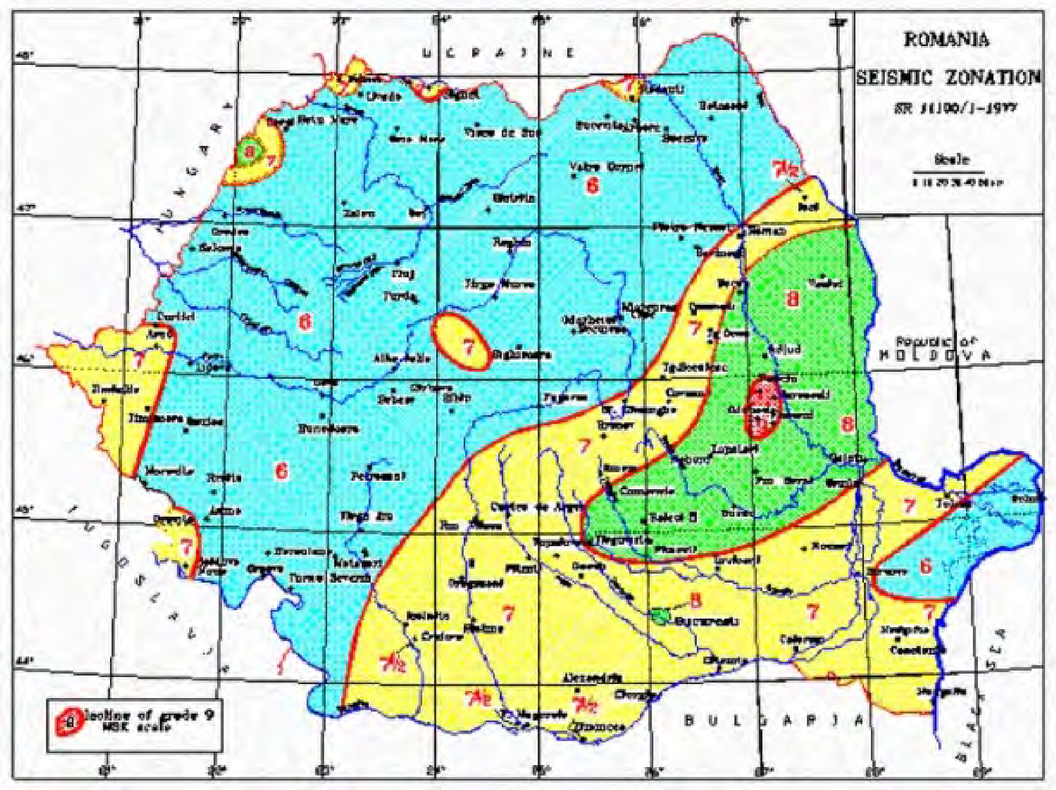

Fig.3.1.3: STAS 1100/1-77 - Seismic zonation in 1977 in terms of degrees in MSK scale.
REFERENCE: STAS 1100/1-77, Macrozonarea teritoriului R. P. Romane (Inforix)

The later **P100-92** seismic design code introduced advanced ductility rules for RC shear wall and frame structures and for steel structures and new values for Timisoara seismic intensity, considering an increased degree of intensity from VII to VII and half on the MSK scale. These modifications also influenced the PGA and the dynamic amplification factor β. On the new seismic zoning map, Timisoara reaches a level of seismic intensity of 7.5 with a peak ground

[4] *Study on seismic design characteristics of existing buildings in Bucharest, Romania* (Postelnicu et al., 2004 pp. 12-20)

acceleration of a_g=0.16g, but it is very close to the limit with the area of 8 degree and a_g=0.20g.

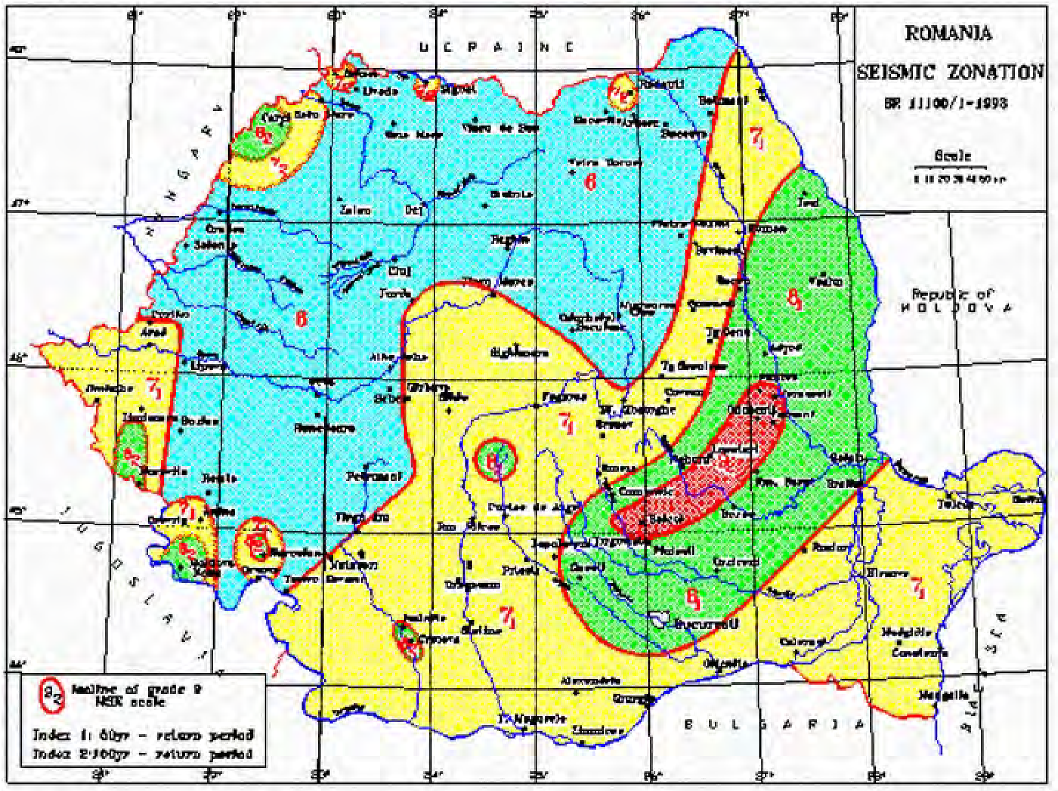

Fig.3.1.4: STAS 1100/1-93- Seismic zonation in 1993 in terms of degrees in MSK scale. REFERENCE: STAS 1100/1-93, Macrozonarea teritoriului R. P. Romane (Inforix)

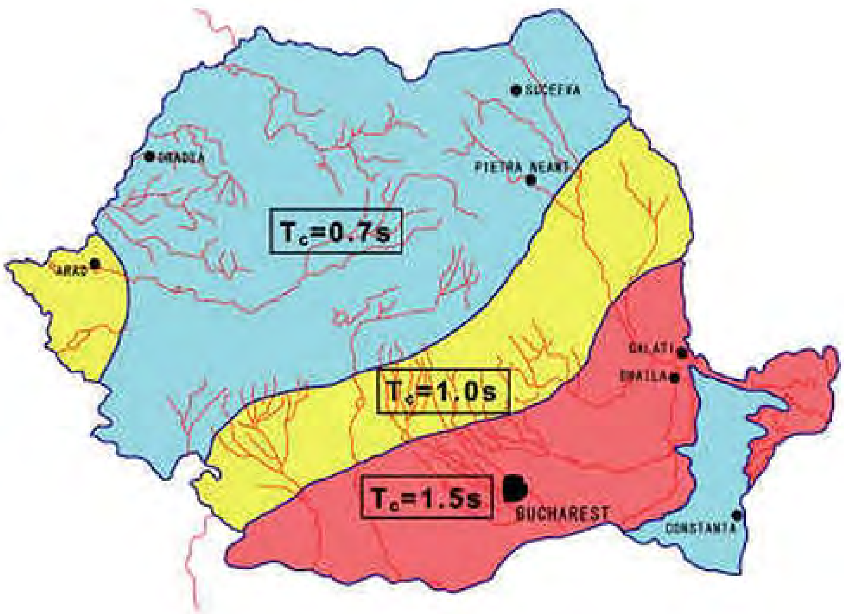

Fig.3.1.5: Seismic zonation map of Romania in 1993 – T_c corner period

REFERENCE: Romania - Code for aseismic design of residental buildings, agrzootechnical an industrial structures (Isee, 1992)

The 2006 seismic design code **P100-2006** can be considered as the first one of the new generation of seismic design codes based on the expected seismic performance and it follows the European code EN 1998-1 considering the requirements performances of life safety (SV) and limit damage (LD); the hazard was significantly lower than the one European code[5].

A separate spectrum with $\beta_0 = 3$ and $T_C = 0.7$ s is given for the crustal sources in the Banat area after the series of significant seismic events occurred in 1991[6].

The 2006 code focused on the level of peak ground acceleration $\mathbf{a_g}$ and**,** on the zonation map, the Banat region had values ranging from $a_g = 0.08g$ to $a_g = 0.20g$, while the value for the town of Timisoara is a_g=0.16g.

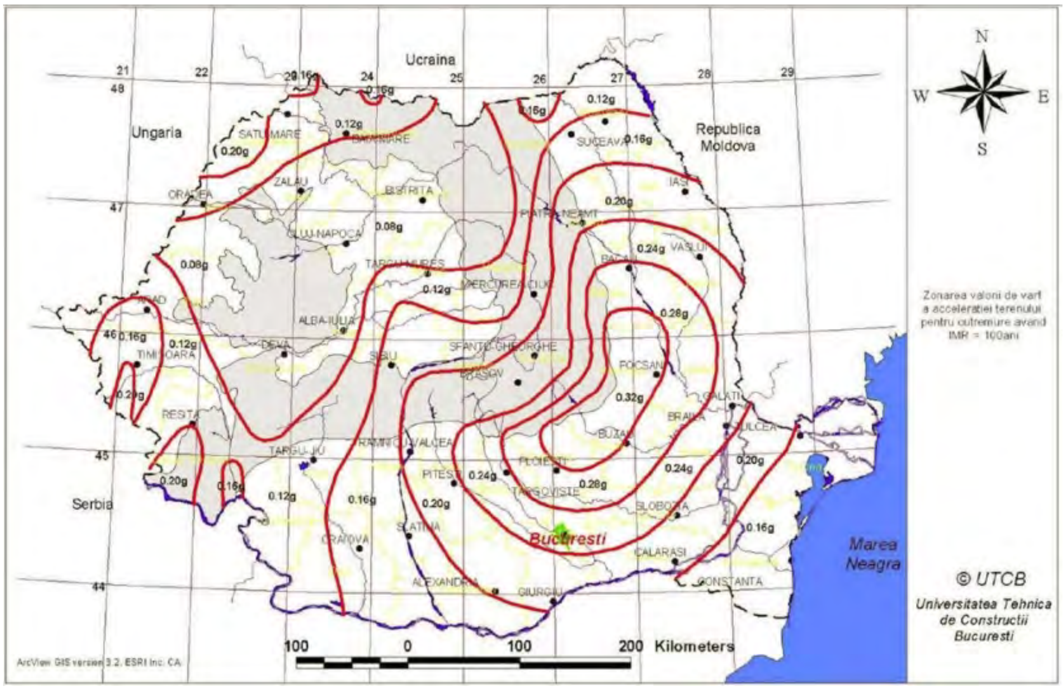

Fig.3.1.6: Zonation of Romanian territory in terms of peak ground acceleration values for earthquakes with average recurrence interval IMR = 100 years. Code design P100-1 / 2006

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[5] *Cod de proiectare seismică p100, Partea I - p100-1/2011 prevederi de proiectare pentru clădiri* (Postelnicu et al., 2011)

[6] *A comparison between the requirements of present and former Romanian seismic design codes, based on the required structural overstrength* (Craifaleanu, 2008, p. 3)

REFERENCE: Cauzele seismelor. Zone seismice (Inforix)

Fig.3.1.7: Romania in terms of zoning control period (corner), T_C response spectrum. Code design P100-1 / 2006

REFERENCE: Cauzele seismelor. Zone seismice (Inforix)

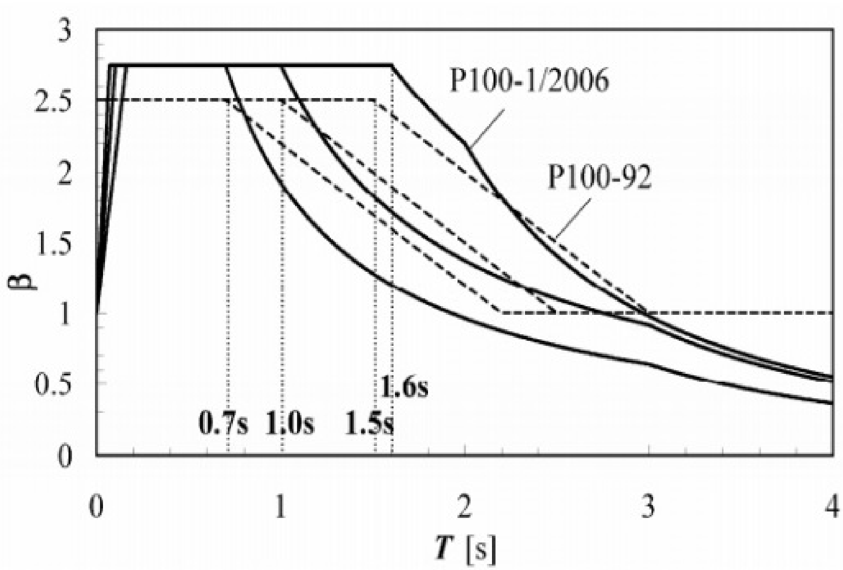


Fig.3.1.8: Comparison between the normalized elastic response spectra in the two releases of P100

REFERENCE: A comparison between the requirements of present and former Romanian seismic design codes, based on the required structural overstrength, (Craifaleanu, 2008, p. 3)

P100-2013 introduced other modifications of these values, increasing the peak ground acceleration to values between a_g = 0.10g to a_g = 0.25g: for Timisoara the value of a_g is a_g=0.20g. The zonation in terms of control period of the national territory is almost identical to P100 -2006 code.

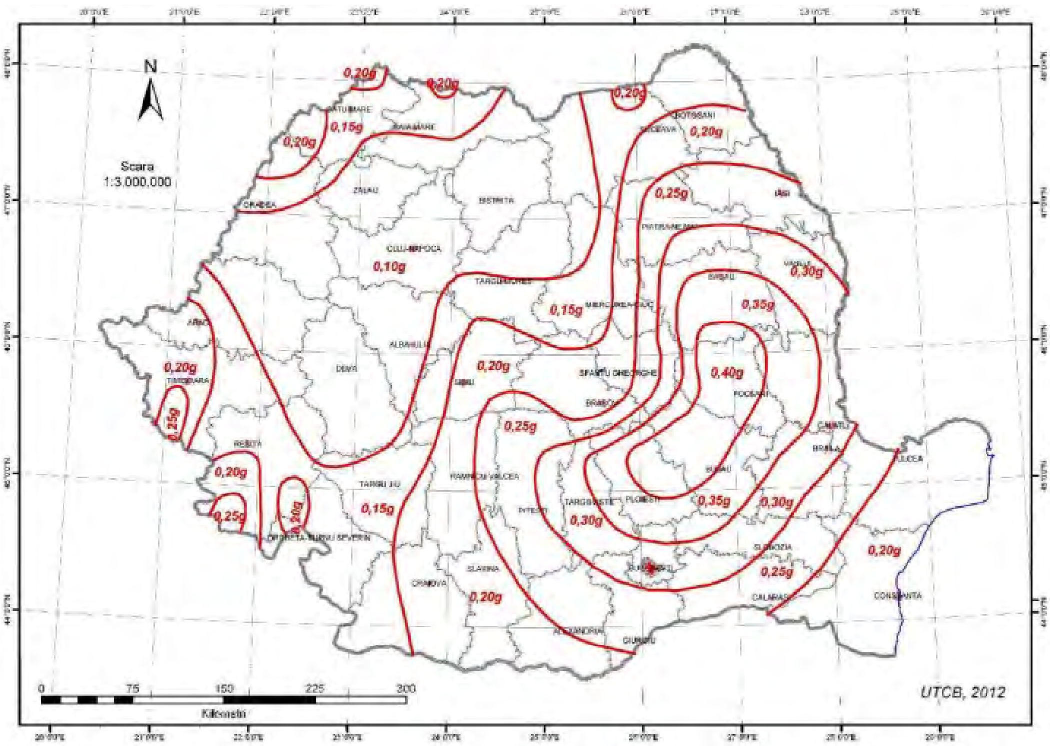


Fig.3.1.9: Zonation of Romania in terms of peak ground acceleration values for ag with IMR = 225 years and 20% probability of exceedance in 50 years

REFERENCE: Cod de proiectare seismică. **(**Siugrc)

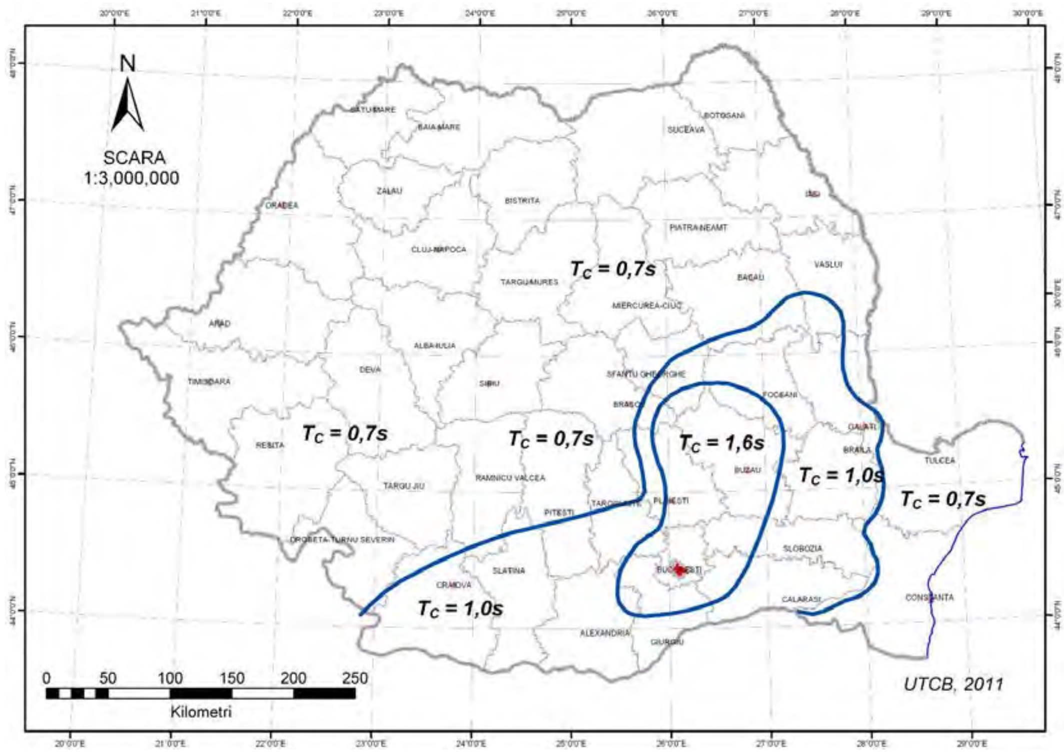


Fig.3.1.10: Romania in terms of zoning control period (corner), T_C response spectrum. Code design P100-1 / 2013

REFERENCE: Cod de proiectare seismică - P100-1 / 2013

Figure 1.4.12 rapresents the evolution of the seismic design coefficent in Bucharest from the design code P13-63 to P100-92. Before the editing of P100-78 and P100-81 design codes buildings ductility was not considered, and the value of the dynamic coefficent β increases progressively making standards more restrictive.

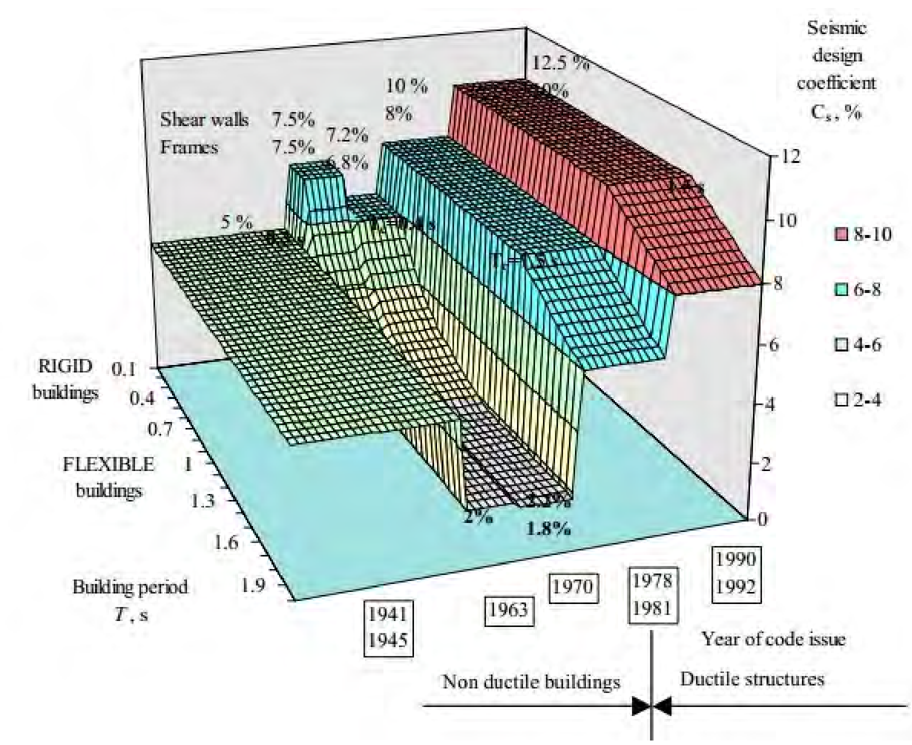


Fig.3.1.11: Evolution of seismic design coefficient in Bucharest during period 1940-2003

REFERENCE: Earthquake protection of historicalbuildings by reversible mixed technologies, (Lungu, Arion and Vacareanu, 2005, p.10)

3.2 HORIZONTAL RESPONSE SPECTRUM

The seismic action parameters used for the vulnerability analysis are defined according to the Eurocode 8[7] and the Romanian Code[8]. The horizontal response spectrum is different for these two codes, so both will be analyzed.

3.2.1 Romanian Code[9]

The Romanian Code subdivides the national territory in seismic zones, in which the hazard level is considered constant. The hazard seismic design is described by the horizontal peak ground acceleration a_g, determinated by the average return period *IMR*. With the so defined a_g is possible to determinate the characteristic value of seismic action, A_{Ek}. The design value of seismic action is A_{Ed} and it is the result of the multiplication of the characteristic value for the importance and exposure factor of the construction:

$$A_{Ed} = \Upsilon_{1,e} \cdot A_{Ek} \tag{3.1}$$

Values of a_g are given in Fig. 3.1.9 and they correspond to *IMR* = 225 years with 20% probability of exceedance in 50 years. For Timisoara the value is 0.20 a_g.

The horizontal component of the elastic response spectrum $S_e(T)$, expressed in m/s^2, is defined as:

$$S_e(T) = a_g \cdot \beta(T) \tag{3.2}$$

where a_g is expressed in m/s^2 and $\beta(T)$ is the normalized elastic response spectrum. The normalized spectrum $\beta(T)$, for a conventional value of critical damping ξ=0.05 and as a function of control periods T_B, T_C and T_D, is given by these relations:

$$0 \le T \le T_B \qquad \beta(T) = 1 + \frac{(\beta_0 - 1)}{T_B} \cdot T \tag{3.3}$$

$$T_B \le T \le T_C \qquad \beta(T) = \beta_0 \tag{3.4}$$

[7] *Design of structures for earthquake resistance* (Eurocode 8 – ENV 1998-1)

[8] *Cod de proiectare seismică* (P100-1 / 2013)

[9] *§3.1 Reprezentarea actiunii seismice pentru proiectare* **(**P100-1 / 2013 , pp. 43-39)

$$T_C \leq T \leq T_D \qquad \beta(T) = \beta_0 \cdot \frac{T_C}{T} \qquad (3.5)$$

$$T_D \leq T \leq 5s \qquad \beta(T) = \beta_0 \cdot \frac{T_C \cdot T_D}{T^2} \qquad (3.6)$$

where:

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

β_0 is the dynamic amplification factor for a maximum ground acceleration of a linear single-degree-of-freedom system, its value is $\beta_0 = 2{,}5$;

T_B, T_C and T_D define the spectrum shape and T_C value is given in Fig. 3.1.10. From its value it is possible to define also T_B and T_D values, due to Table 3.1:

T_C	0,70 s	1,00 s	1,60 s
T_B	0,14 s	0,20 s	0,32 s
T_D	3,00 s	3,00 s	2,00 s

Tab. 3.2.1: Control period T_B, T_C, T_D for horizontal component of response spectrum

REFERENCE: §3.1 Reprezentarea actiunii seismice pentru proiectare (P100-1 / 2013, p. 45)

The normalized elastic response spectrum $\beta(T)$ for the city of Timisoara is the following.

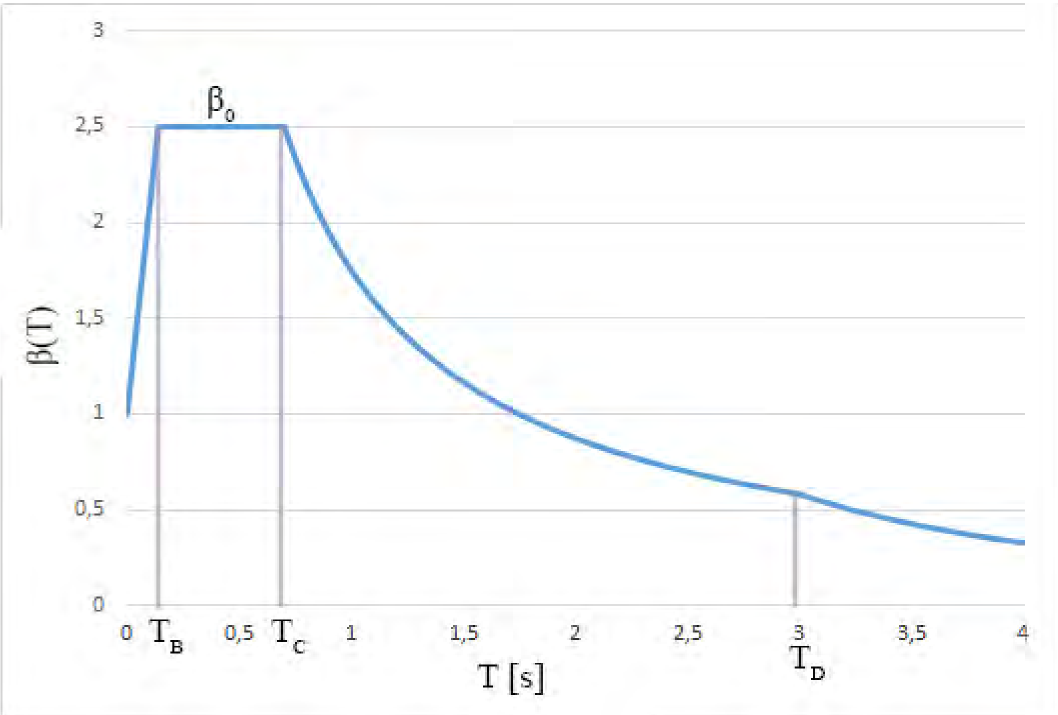


Fig. 3.2.1: Normalized elastic response spectrum according to Romanian Code for the city of Timisoara

3.2.2 Eurocode 8[10]

The Eurocode describes seismic hazard "in terms of a single parameter, which is the value of the reference peak ground acceleration on type A ground, a_{gR}. […] The reference peak ground acceleration, chosen by National Authorities for each seismic zone, correspond to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference of exceedance in 50 years, P_{NCR}) chosen by National Authorities."[11]

An importance factor Υ_I=1 is assigned to the reference return period and it depends on the types of building in terms of consequences of failure. The design ground acceleration on type A ground a_g is defined as:

$$a_g = \Upsilon_I \cdot a_{gR} \tag{3.7}$$

The identification of ground types regards the influence of local ground condition on the seismic action. Ground types are described by the stratigraphic profiles and parameters given in Tab. 3.2. The average shear wave velocity $v_{s,30}$ for the city of Timisoara is about 200 m/s (see §1.2.4 p.15) so the ground type is C.

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ [m/s]	N_{SPT} [blows/30cm]	C_u [kPa]
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250

10 *§3 Ground conditions and seismic action* (Eurocode 8 – ENV 1998-1, pp. 33-44)

11 *§3.2.1 Seismic zones* (Eurocode 8 – ENV 1998-1 ,p. 35)

D	Deposits of lose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominately soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft alys/silts with high plasticity index (Pl > 40) and high water content.	<100 (indicative)	-	10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A-E or S_1			

Tab. 3.2: Ground types

REFERENCE: § 3.2.2 *Identification of ground types* (Eurocode 8 – ENV 1998-1 ,p. 34)

In the Eurocode 8 "the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an 'elastic response spectrum'".[12] The shape of this spectrum is the same for the no-collapse requirement and for the damage limitation requirement. The horizontal elastic response spectrum $S_e(T)$ is defined by the following expression:

$$0 \leq T \leq T_B \qquad S_e(T) = a_g \cdot S \cdot [\, 1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5\text{-}1)] \tag{3.8}$$

$$T_B \leq T \leq T_C \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \tag{3.9}$$

$$T_C \leq T \leq T_D \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \tag{3.10}$$

$$T_D \leq T \leq 4s \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \left[\frac{T_C \cdot T_D}{T^2}\right] \tag{3.11}$$

[12] *§3.2.2.1 Basic representation of the seismic action–General* (Eurocode 8 – ENV 1998-1, p. 36)

where:

$S_e(T)$ is the elastic response spectrum;

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

a_g is the design ground acceleration on type A ground ($a_g = \Upsilon_1 \cdot a_{gR}$);

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

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

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

S is the soil factor;

η is the damping correction factor with a reference value of η=1 for 5% viscous damping.

The values of the periods T_B, T_C and T_D and the soil factor S depend on the ground type and they describe the shape of the elastic response spectrum.

The Code defines two types of spectra, which depend from the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment: if the earthquake has a surface-wave magnitude M_s not greater than 5,5 it is recommended that the Type 2 spectrum is adopted, if it is greater than 5,5 the Type 1 spectrum has to be adopted[13]. The historical earthquakes that interested the area of Timisoara had a surface-wave magnitude usually lower than 5,5, but few of them exceeded this value, so both spectra have been evaluated but Type 2 has been used.

Ground type	S	T_B [s]	T_C [s]	T_D [s]
A	1	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Tab. 3.2.3: Values of the parameters describing the recommended Type 1 elastic response spectra

REFERENCE: §3.2.2.2 Horizontal response spectrum (Eurocode 8 – ENV 1998-1, p. 38)

[13] *§3.2.2.2 Horizontal response spectrum* (Eurocode 8 – ENV 1998-1, p. 38)

Ground type	S	T_B [s]	T_C [s]	T_D [s]
A	1	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Tab. 3.2.4: Values of the parameters describing the recommended Type 2 elastic response spectra
REFERENCE: §3.2.2.2 Horizontal response spectrum (Eurocode 8 – ENV 1998-1, p. 39)

The Type 1 and Type 2 elastic response spectrum for the city of Timisoara are so the following:

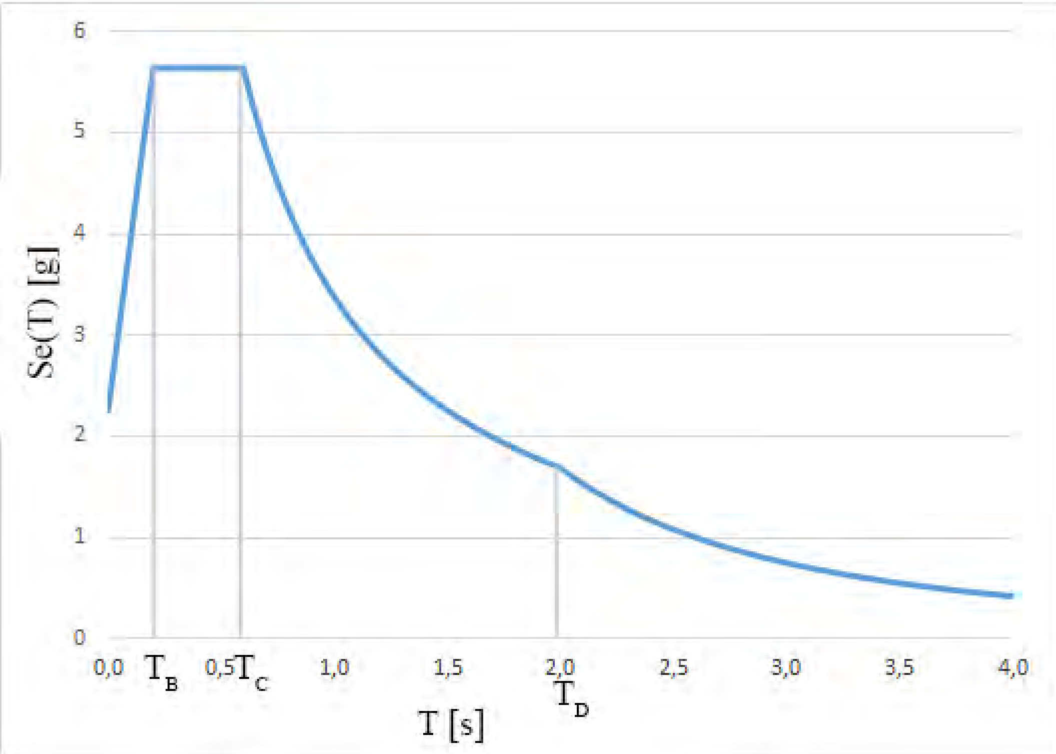


Fig. 3.2.2: Type 1 elastic response spectrum according to the Eurocode 8 for the city of Timisoara

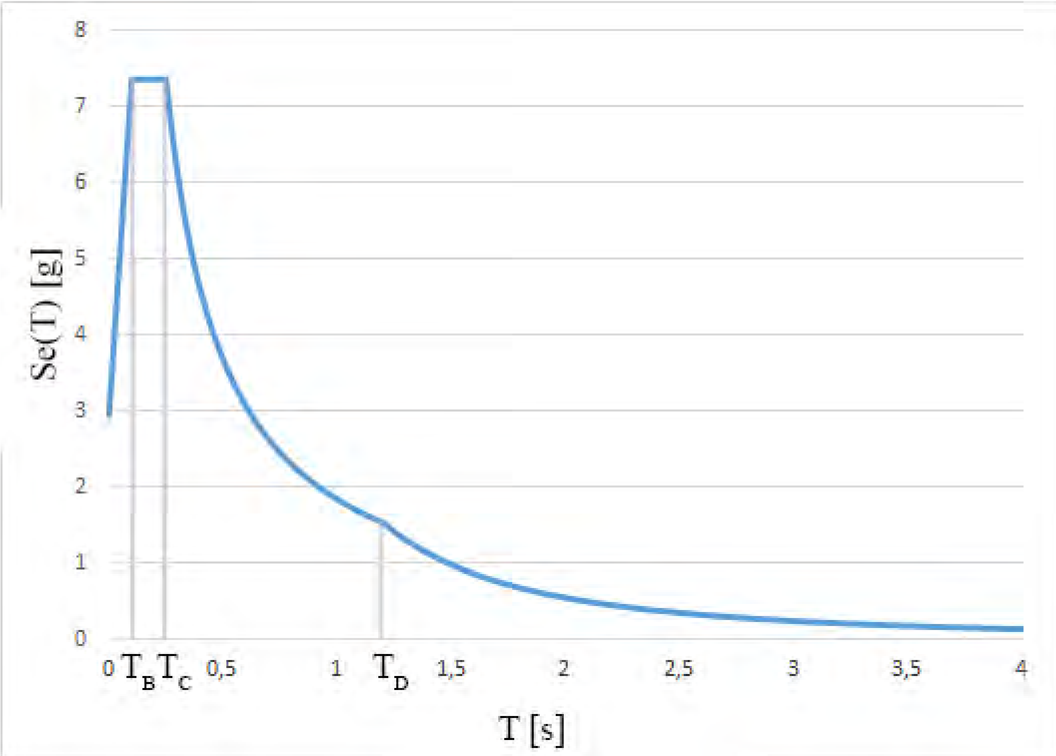


Fig. 3.2.3: Type 2 elastic response spectrum according to the Eurocode 8 for the city of Timisoara

3.3 MATERIALS AND LOAD ANALYSIS

For the vulnerability analysis it is necessary to know material characteristics and it was chosen to adopt values defined by the Italian regulation, NTC 2008.[14] For masonry buildings the Italian Code defines three knowledge levels, according to the quality of information obtained. For this analysis the assumed level is *LC1,* that is reached when a geometric survey and limited in situ inspections are made.[15] The knowledge level defines, for each wall typology, the values of mechanical parameters. For the masonry compressive strength and shear strength the minimum value has been taken, between 240 and 400 N/cm^2 for the first parameter and between 6 and 9,2 N/cm^2 for the second one. For the modulus of elasticity and the modulus of tangential elasticity the medium value has been taken, between 1200 and 1800 N/mm^2 for the first parameter and between 400 and 600 N/mm^2 for the second one. The indicated value of 18 kN/m^3 has been taken for the specific weight of masonry.

In Timisoara the vertical structures are made of solid bricks and lime mortar masonry, so the mechanical parameters for this wall typology, for a knowledge level *LC1,* are:

f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
240	6	1600	500	18

Tab. 3.2.5: Material properties of solid bricks and lime mortar masonry

REFERENCE: §C8A.2 Tipologie e relative parametri meccanici delle murature (Circolare NTC 2008, p. 403)

where:

f_m is the average value of masonry compressive strength

τ_0 is the average value of masonry shear strength

[14] *Nuove Norme Tecniche per le Costruzioni* (Decreto del Ministero delle Infrastrutture 14/01/2008);
Istruzioni per l'applicazione delle Norme Tecniche per le Costruzioni (Circolare esplicativa n. 617 02/02/09)

[15] *§C8A.1.A.4 Costruzioni in muratura: livelli di conoscenza* (Circolare NTC 2008, p. 391)

E is the average value of modulus of elasticity

G is the average value of modulus of tangential elasticity

w is the average specific weight of masonry

Regarding horizontal structures, architectural details have been hypothesized to estimate permanent loads as follow:

Horizontal structure typology	Constituent elements	Dimensions [cm]	ϒ [kN/m³]	Permanent load G_k [kN/m²]
Adjoining type timber structure	Plaster	1,5	0,19	2,31
	Fir beams	20	6	
	Soil and shards filling	3	15	
	Wooden planking	6	6	
	Wood finish	2,5	6	
	Parquet	2,5	6	
R.C. slab	Plaster	1,5	0,3	4,70
	Concrete slab	15	24	
	Screed	7	14	
	Floor finish	2	6	
Iron beams and little brick vaults	Iron beam (IPE 200) i = 1 m	0,284	78,5	3,91
	Solid bricks	15	17	
	Soil and shards filling	3	15	
	Loamy soil filling	3	13	
	Wooden finish	2,5	6	
	Parquet	2,5	6	

Tab. 3.2.6: Horizontal structures load analysis

REFERENCE: §3.1.2 Pesi propri dei materiali strutturali (NTC 2008, p. 11)

To evaluate permanents loads of brick vaults, the Gelfi program *Arco* has been used. The characteristics considered were:

Ring geometry	Span	4,5 m
	Rise	1,5 m
	Thickness	0,15 m
	Segments number	30
Unitary weight of volume γ	Bricks	17 kN/m^3
	Soil and shards filling	15 kN/m^3
	Timber structure	6 kN/m^3
Timber structure dimension		0,1 m
Soil and shards filling dimension		0,5 m
Ring width		1 m

Tab. 3.2.7: Vaults characteristics considered on permanent loads evaluation

This program gives horizontal and vertical reactions for each spring in case of barrel vaults. In case of cross vaults the reaction can be halved to obtain the searched results. The permanent load for brick vaults and concrete horizontal diaphragms are obtained from a combination of brick vault loads and R.C slab loads.

The accidental loads follow the Italian regulation and are described above:

Building category	Environment	q_k [kN/m^2]	Ψ_{2j}
A	Residential environment	2,00	0,3
B	Open to public offices	3,00	0,3
C	Environments susceptible to crowding	3,00	0,6
D1	Shops	4,00	0,6
D2	Commercial centers and libraries	5,00	0,6
E1	Libraries, archives, deposits, etc…	6,00	0,8
H1	Not-walkable roof and attics	0,5	0,0

Tab. 3.2.8: Accidental loads and values of combination factors for different building categories

REFERENCE: §2.5.3 Combinazione delle azioni (NTC 2008, p. 8); *§3.1.4 Carichi variabili* (NTC 2008, p. 12);

4 VULNERABILITY ASSESSMENT

The vulnerability assessment has been used in two different applications. The first one regards the study of three blocks of the historical center, with the purpose of defining the vulnerability level of the entire aggregate. The second application regards the study of four singular structural units, to compare the results obtained for two different level of information: the real case, obtained by detailed plans and sections, and the survey case, using the city plan of 1980 and information collected in situ.

4.1 THE VULNERABILITY ASSESSMENT

An aggregate of buildings is constituted by an assembly of linked parts that are the result of a complex and not uniform genesis, due to many factors such as the construction sequence, different materials, changing needs, owners alternation, etc[1]. An historical aggregate is indeed composed by adjoining buildings that interact to each other. The vulnerability assessment of these blocks must consider the buildings both in their entirety and in relation with adjacent corps with different characteristics. All the elements of the same aggregate are indeed connected, creating a complex set in which the structural behavior of a building can not ignore the adjacent units structural behavior.[2] The software *Vulnus,* developed by the University of Padua, performs an expeditious analysis of buildings seismic vulnerability as a whole and allows statistical evaluations on the aggregate results[3].

[1] *§C8A.3 Aggregati edilizi* (Circolare esplicativa NTC 2008, p. 406)

[2] *Manuale d'uso del programma, Vulnus 4.0* (Università degli Studi di Padova, 2009, p. 5)

[3] *Ibidem*

4.1.1 The Vulnus methodology[4]

The Vulnus methodology works on the evaluation of the critical level of average horizontal acceleration applied to the building masses, correspondent to the activation of out of plan mechanisms of individual walls and in plan of the two system of parallel walls connected with the horizontal structures.

Empirical observation of the earthquake effects and laboratory simulations demonstrated that masonry buildings can break up in two distinct modes:

- Masonry walls characterized by reasonable quality and effectively confined from orthogonal walls and rigid stories can manifest shear failure of the walls;
- Masonry walls characterized by low quality walls and not adequate enclosure can manifest complex failure mechanisms for perimeter walls.

The vulnerability model of this methodology depends on these indexes:

- I1: it is the ratio between the summation of the shear strengths in the middle plan of parallel walls in the weaker direction between the principal two of the building and the total building weight, that can be corrected to consider eventual irregularities in plan or in elevation. The I1 index is dimensionless and it can be seen also as the critical ratio between the average acceleration of the masses A and the gravity acceleration g. The evaluation of this parameter needs an estimation of the average resistance to diagonal traction, that can be obtained from laboratory data experimented on similar typologies.
- I2: it is the ratio between the average acceleration that activates the out of plan mechanisms in the most critical conditions and the gravity acceleration. Through limit analysis of different kinematic models, the resistance of vertical masonry panels of external walls of the building, connected to horizontal structure with only confinement forces, and horizontal panels, connected to perpendicular walls through nodal areas, are separately evaluated. The local acceleration at different floors is evaluated assuming a distribution proportional to the building height. It is

[4] *Ivi (p.6)*

also required an evaluation of the tensile and compressive strength of the masonry walls and of the containment forces that may develop at the horizontal diaphragms level, due to the presence of effective tie rods or due to other resistance mechanisms. The analysis of just the external walls is justified from empirical observation on building damage during seismic events: out of plan mechanisms are not observed in case of internal septs or perimeter septs adjacent to higher.

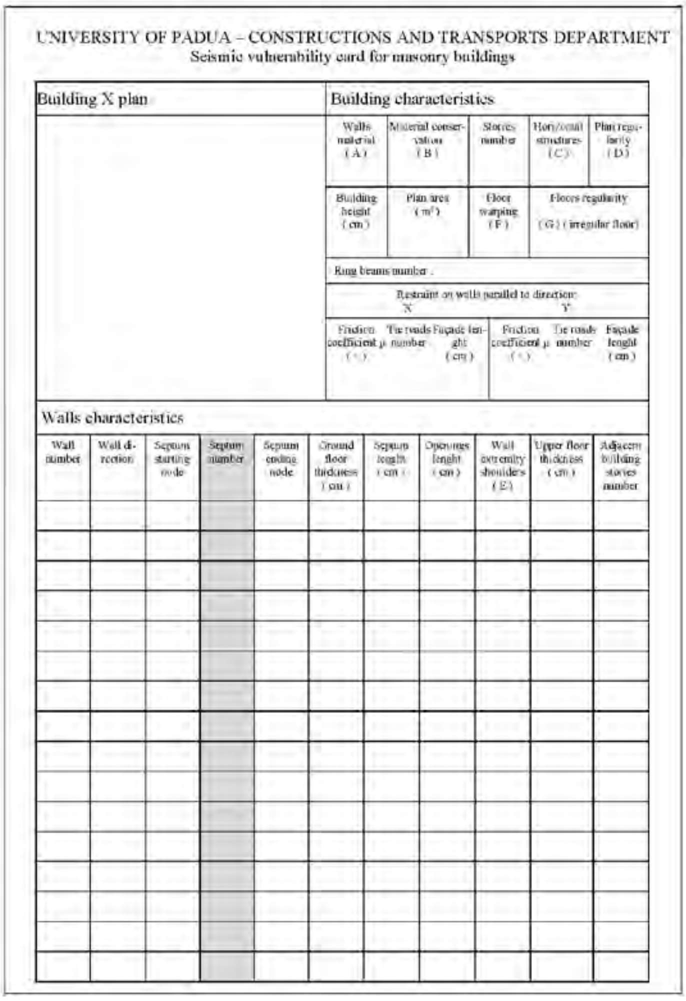
- I3: it is the weighted sum of the scores of the partial vulnerability factors expressed in the second level GNDT form: this value is normalized between 0 (construction built in a workmanlike manner or according to seismic codes) and 1. It is an empirical parameter that considered qualitative factors not considered by the parameters calculation and that draws vulnerability index defined by Benedetti and Petrini (1984), discarding the parameters jet valued by the previous two indices.

The indices I1, I2 and I3, properly combined, are the basis for the formulation of a comprehensive assessment of the seismic vulnerability, which also takes into account the quality of the information gathered at the base of the calculation.

The vulnerability rating may referred to the single structural unit, to the entire block, or to all the blocks in question. A later stage leads to the calculation of the expected values of serious damage through the construction of fragility curves and comparison between results obtained and those expected from the scale of macro seismic intensity EMS98.

4.1.2 The survey form[5]

The input data derive from the compilation of the “Seismic vulnerability form for masonry buildings” (Fig 4.1).



UNIVERSITY OF PADUA – CONSTRUCTIONS AND TRANSPORTS DEPARTMENT
Seismic vulnerability card for masonry buildings

Building X plan

Building characteristics

Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
Building height (cm)	Plan area (m²)	Floor warping (F)	Floors regularity (G) (irregular floor)	

Ring beams number :

Restraint on walls parallel to direction:

X			Y		
Friction coefficient μ (-)	Tie rods number	Façade lenght (cm)	Friction coefficient μ (-)	Tie rods number	Façade lenght (cm)

Walls characteristics

Wall number	Wall direction	Septum starting node	Septum number	Septum ending node	Ground floor thickness (cm)	Septum length (cm)	Openings lenght (cm)	Wall extremity shoulders (E)	Upper floor thickness (cm)	Adjacent building stories number

Fig. 4.1.1 Seismic Vulnerability survey form for masonry buildings
REFERENCE: Mauale d’uso del programma Vulnus 4.0 (Università degli studi di Padova , 2009)

Structural units must be homogeneous respect to the following building characteristics:

- Period of construction;
- Height and volume;
- Materials and preservation state;
- Construction techniques;
- Foundations type and soil characteristics.

[5] *Manuale d’uso del programma, Vulnus 4.0* (Università degli Studi di Padova, 2009, pp. 11-18)

The form is divided in three parts that concerns buildings and walls characteristics.

The first section is a space reserved to a schematic drawing of the building plan where nodes, walls and septa shall be identified by an index. The chosen plan has to be the more representative of the building. The ground floor plan is usually considered because the resistance verification to horizontal seismic forces is generally more onerous than in other floors.

A wall identifies a straight portion of masonry that can be divided into more septa, pinpointed by the initial and the final node, determined by walls intersections. The second section summarizes the general characteristics of the building, that are:

Constituent material: this parameter considers the prevalent material that constitute the walls or in case of unevenness the material with the worst characteristics.

MATERIAL	**RESISTANCE** [MPa]		**SPECIFIC DENSITY** [kg/m^3]
	COMPRESSION	TRACTION	
1) not identified	1.5	0.08	2100
2) stone	2.6	0.14	2100
3) bricks	4.0	0.22	1800
4) RC blocks	4.0	0.36	1200
5) tuff block	3.2	0.20	1800

Tab. 4.1.1: Wall material and relative mechanical properties in the case of good quality masonry

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 13)

M**aterial conservation:** it describes the state of the building, and it is defined by four classes, from not identified to bad conservation.

CONSERVATION STATE	
1) not identified	Mechanical characteristics multiplied by 0.75
2) good	Mechanical characteristics multiplied by 1.00
3) mediocre	Mechanical characteristics multiplied by 0.75
4) bad	Mechanical characteristics multiplied by 0.50

Tab. 4.1.2: Conservation state

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 14)

Stories number: this parameter takes count of the building stories number, including the basement but not the underground.

Horizontal structure typologies: it takes into account the loads per unit surface, plan rigidity and horizontal and vertical structure gripping.

PERMANENT LOADS		
1) not identified		G + Q= 3.7 kN/m^2
2) very light	wood (even stiffened), iron beams and little brick vaults	G + Q= 2.2 kN/m^2
3) light		G + Q= 3.7 kN/m^2
4) medium	Concrete	G + Q= 5.2 kN/m^2
5) heavy		G + Q= 6.7 kN/m^2
6) very heavy		G + Q= 8.2 kN/m^2

Tab. 4.1.3: Horizontal structures permanent loads

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 14)

Plan regularity: it considers the presence of holes and mass distribution.

PLAN REGULARITY
1) not identified
2) regular
3) not regular

Tab. 4.1.4: Plan regularity

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 15)

Building height: this parameter is measured from ground level to the gutter line

Building surface: it represents the building area and is measured considering the external walls line.

Warping of horizontal structures: it consider the prevalent warping present in the building.

HORIZONTAL STRUCTURE WARPING
1) not identified
2) X prevalent
3) Y prevalent
4) both directions

Tab. 4.1.5: Horizontal structures warping
REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 15)

F**loor regularity:** it identified the presence of thrusting floors or overweight.

FLOOR REGULARITY
1) not identified
2) regular
3) inactive at (_) floor on the walls parallel to X direction
4) inactive at (_) floor on the walls parallel to Y direction
5) inactive at (_) floor on the walls parallel to X andY direction
6) overweight at floor (_)

Tab. 4.1.6: Floor regularity

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, p. 16)

Walls restraint: this parameter denotes the continuity between horizontal and vertical structures in the parallel direction to X and Y. The friction coefficient, the number of tie rods and the dimension of the principal façade in both directions X Y are part of the parameter.

The last section summarize the septa characteristics:

- Wall index;
- Wall angle;
- Initial and final node;
- Septum number;
- Ground floor thickness;
- Septum length;
- Holes dimension;

- Extremity shoulders, that gives information about nodes: they are considered weakened if the distance between the hole and the node is less than half of the dimension of the hole. They are identified by a number:
  1) not identified
  2) regular shoulders
  3) initial shoulder not regular
  4) final shoulder not regular
  5) both shoulders not regular.
- Last storey thickness;
- Stories number of the adjacent building with the value of:
  - 0 for isolated septa;
  - -1 for internal septa or septa in common with other buildings;
  - stories number *nc* of the adjacent building for septa common to other buildings with lower height.

If the building shows elevation and/or plan irregularity it can be subdivided in more structural units.

4.1.3 Elementary kinematic models[6]

Vulnus considered two groups of elementary kinematic models, as said in chapter 4.1.1: the first group includes in plane mechanisms of collapse (shear strength) and the second one considers out of plane mechanism of collapse. Usually, it is the second group of mechanisms that leads to the building collapse.

The hypotheses considered by the program are:

- The distribution of the masses (including floors) is uniform along the whole height of the structure;
- The acceleration distribution is proportional to the building height;
- The walls parallel to the direction of the earthquake, in favor of security, absorbs the entire horizontal action transferred to them through mechanisms of flexural strength of orthogonal walls connected through the ceilings.

[6] *Manuale d'uso del programma, Vulnus 4.0* (Università degli Studi di Padova, 2009, pp. 19-21)

4.1.4 In plane mechanism of collapse

For each of the two main directions of the building, Vulnus evaluates the relationship between the critical value of the average horizontal acceleration on the building masses and the acceleration of gravity as ratio of the base shear and the total weight of the construction. The parameter I1 is given by the minimum value.

Considering the vertical load uniformly distributed over the walls, the error introduced for the typology is small. The vertical stress σ_0 depends mainly from the masonry weight, even considering the warping floors unidirectional (which does not correspond to reality) and neglecting the transmission capacity of the tangential stresses between the masonry at different levels of vertical tension.

In the case of clustered buildings, the septa in common between two buildings must contribute to the shear strength of both: the distribution of shear strength is assumed proportionally to the load applied to them by the adjoining buildings, and that is approximately connected to the floor number.

Therefore, the thickness of the wall, for adjacent walls in buildings with equal or lesser height, it is reduced proportionally to the load of responsibility. In case of lower contiguous buildings should be indicated the real total thicknesses for ground floor and top floor.

4.1.5 Out of plane mechanism of collapse

For out of plane mechanism of collapse, kinematic mechanisms concerning vertical and horizontal masonry strips are identified. For each mechanisms, the relationship between the average horizontal acceleration, which activates the mechanism, and the acceleration of gravity was calculated. A wall can be considered composed by a number of vertical stripes or by a number of horizontal stripes, so two resistance contributions has been considered:

- I2' considers the resistance of 1m masonry vertical stripes simply supported (with no tensile strength) on the foundation and transversal walls or connected to the floors;
- I2 " considers the arc or beam resistance of horizontal masonry stripes connected by the transverse walls (parallel to the earthquake direction).

The I2 index, representative of the out of plane building resistance, is given by the lesser of the sums I2 '+ I2" calculated for the various walls.

The new version of the program was integrated with other mechanism that add the presence of a full curb on the masonry, since it can improve the box-shaped structure effect and can increase the forces that counteract the earthquake action. The action of containment is transmitted to the masonry thanks to the friction, which is formed from the contact between masonry and curb.

The stabilizing effect of the floors have different values depending on the floor warping. In case of presence of a prevailing direction of floor warping $\mu = 0.15$ is used, while for orthogonally septa $\mu = 0.05$ is assumed.

The friction force caused by the presence of the curb (or the friction between only horizontal and vertical structure with no curb) is inversely proportional to the height, because the weight of the overlying load decreases with the increase of the stories.

Both the tie rods and the restraining effect of the curbs are subjected to deformations at high altitudes, so they respond to the seismic action with bigger forces on the given area.

Thanks to the introduction of this model, it is possible to calculate the force produced by the curb presence and the force caused by the horizontal-vertical structures interaction.

4.1.6 Kinematic mechanisms for 1 m deep vertical stripes

As regards the one meter vertical strips mechanisms, the program considers the analysis to the global overturning of the wall (this analysis independent from the masonry resistance value) and the overturning or bending failure of the wall on the upper floor for high containment pressures (for simplicity it is considered that acceleration, and so distributed load on the last floor, is constant and equal to the average of the floors).

4.1.7 Kinematic mechanism for 1 m high horizontal stripes

The horizontal stripes are divided into spans of 1 meter height and length l for each wall, restricted by transversal walls in the nodal points, but not by the horizontal structures and resistant to orthogonal actions:

- As beams embedded to nodes until the tensile strength limit of masonry (critical acceleration value a_1);
- According to the arc mechanism with displacement in the thickness of the wall, until the collapse limit by compression or by overturning of the arc shoulders (the critical acceleration value a_2).

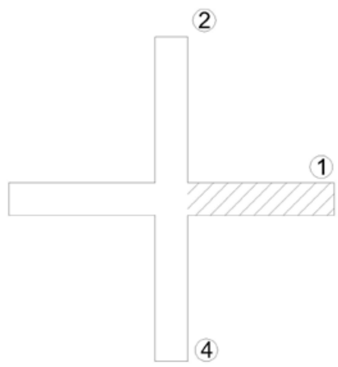
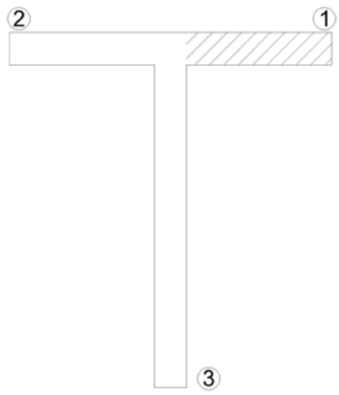
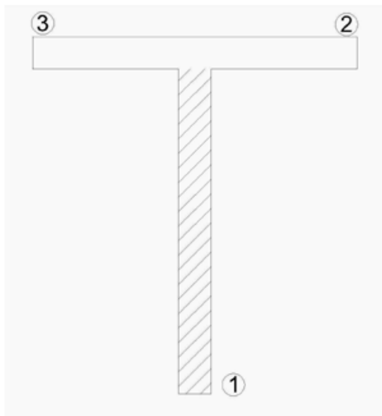
In both cases, for each node, the resistance to separation of the strips by transverse walls must be verified (critical acceleration value a_3). The shear strength of the horizontal stripes does not appear limitative, with geometric values and materials strength practically significant.

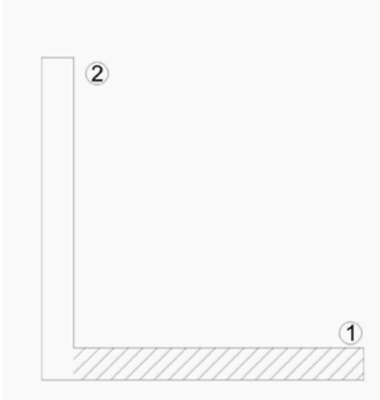
4.1.8 Effects of interaction between adjacent buildings

After the definition of the elementary out of plane kinematic mechanisms, Vulnus calculates the indices I2' and I2" values for each considered mechanisms, for all the septa. It is also possible to display all the values calculated for each mechanism and septum.

For the evaluation of I2 index not all the calculated values are considered: some mechanisms are considered by the procedure as not activatable. For example, in I2' calculation only the free external walls are considered. To calculate I2" all the shoulder nodes must be analyzed in order to understand which kind of mechanism can be activated (bending failure, compressive failure of the arch, local overturning due to the thrust of the arch, detachment of the septum from the perpendicular wall).

Vulnus considers the nodes shown in Tab 4.1.7:

NODE TYPE	
CROSS NODE Possible breaking mechanisms: • Bending failure • Failure of the arch Mechanisms that cannot be activated: • Separation of the wall perpendicular to the septum • Local overturning due to the thrust of the arch	
INTERMEDIATE “T” NODE Possible breaking mechanisms: • Bending failure • Compression breaking of the arch • Separation of the wall perpendicular to the septum Mechanisms that cannot be activated: • Overturning of the wall shoulder perpendicular to the septum due to the thrust of the arch	
ENDING “T” NODE Possible breaking mechanisms: • Bending failure • Compressive failure of the arch • Local overturning due to the thrust of the arch Mechanisms that cannot be activated: • Separation of the wall perpendicular to the septum	

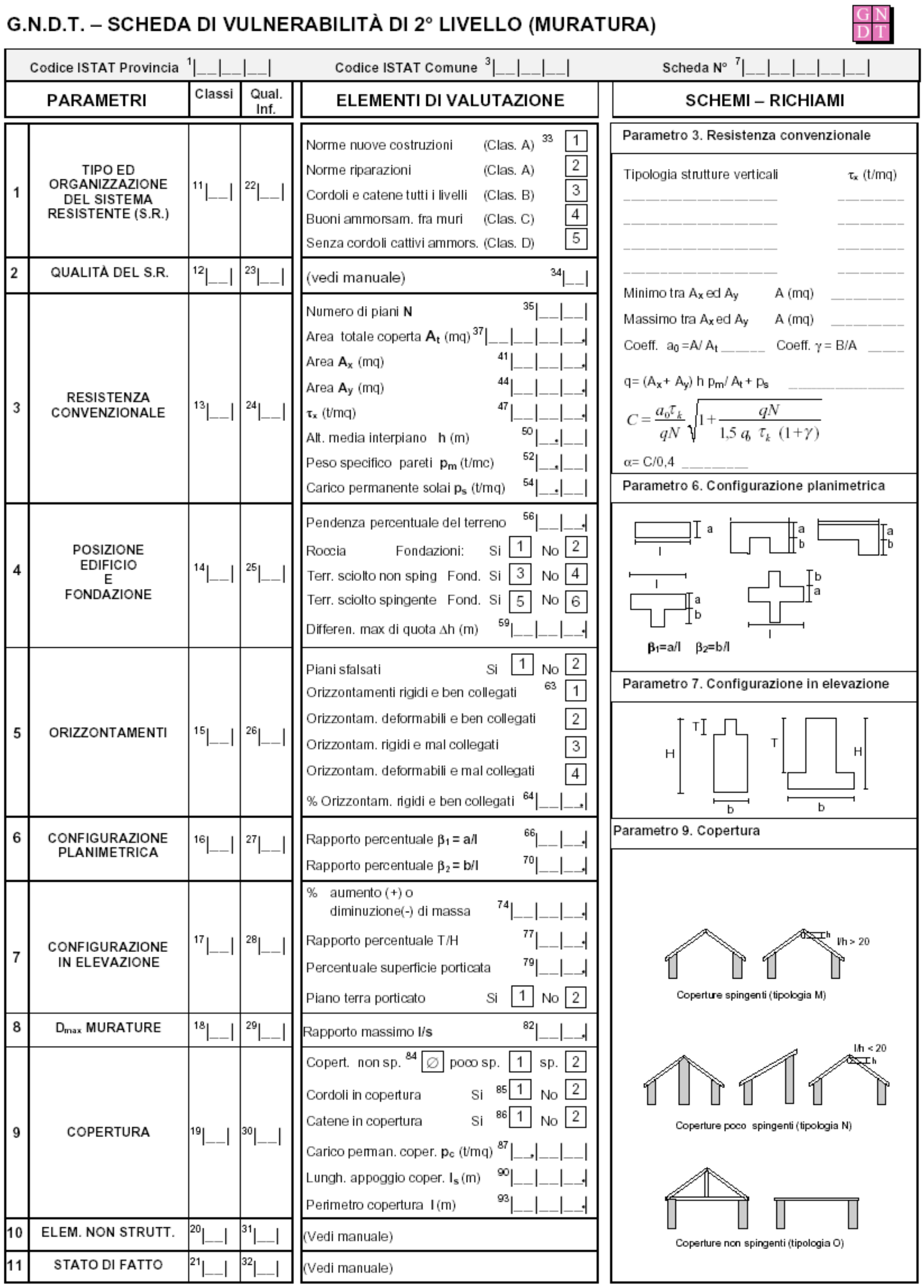
"L" NODE Possible breaking mechanisms: • Bending failure • Compressive failure of the arch • Local overturning due to the thrust of the arch • Separation of the wall perpendicular to the septum	

Tab. 4.1.7: Effects of interaction between adjacent buildings

REFERENCE: Manuale d'uso del programma, Vulnus 4.0 (Università degli Studi di Padova, 2009, pp. 35-36)

4.1.9 I3 Index calculation

The I3 index considers the factors, positive and negative, overlooked in the previous resistant mechanisms. These additional factors are identified through the compilation of the second level GNDT form, for each analyzed building. Below the form is shown.



G.N.D.T. – SCHEDA DI VULNERABILITÀ DI 2° LIVELLO (MURATURA)

Codice ISTAT Provincia 1 |__|__|__| Codice ISTAT Comune 3 |__|__|__| Scheda N° 7 |__|__|__|__|__|__|

	PARAMETRI	Classi	Qual. Inf.	ELEMENTI DI VALUTAZIONE
1	TIPO ED ORGANIZZAZIONE DEL SISTEMA RESISTENTE (S.R.)	11	22	33 Norme nuove costruzioni (Clas. A) 1; Norme riparazioni (Clas. A) 2; Cordoli e catene tutti i livelli (Clas. B) 3; Buoni ammorsam. fra muri (Clas. C) 4; Senza cordoli cattivi ammors. (Clas. D) 5
2	QUALITÀ DEL S.R.	12	23	(vedi manuale) 34
3	RESISTENZA CONVENZIONALE	13	24	Numero di piani N 35; Area totale coperta A_t (mq) 37; Area A_x (mq) 41; Area A_y (mq) 44; τ_x (t/mq) 47; Alt. media interpiano h (m) 50; Peso specifico pareti p_m (t/mc) 52; Carico permanente solai p_s (t/mq) 54
4	POSIZIONE EDIFICIO E FONDAZIONE	14	25	Pendenza percentuale del terreno 56; Roccia Fondazioni: Si 1 No 2; Terr. sciolto non sping Fond. Si 3 No 4; Terr. sciolto spingente Fond. Si 5 No 6; Differen. max di quota Δh (m) 59
5	ORIZZONTAMENTI	15	26	Piani sfalsati Si 1 No 2; 63 Orizzontamenti rigidi e ben collegati 1; Orizzontam. deformabili e ben collegati 2; Orizzontam. rigidi e mal collegati 3; Orizzontam. deformabili e mal collegati 4; % Orizzontam. rigidi e ben collegati 64
6	CONFIGURAZIONE PLANIMETRICA	16	27	Rapporto percentuale $\beta_1 = a/l$ 66; Rapporto percentuale $\beta_2 = b/l$ 70
7	CONFIGURAZIONE IN ELEVAZIONE	17	28	% aumento (+) o diminuzione(-) di massa 74; Rapporto percentuale T/H 77; Percentuale superficie porticata 79; Piano terra porticato Si 1 No 2
8	D_{max} MURATURE	18	29	Rapporto massimo l/s 82
9	COPERTURA	19	30	Copert. non sp. 84 ∅ poco sp. 1 sp. 2; Cordoli in copertura Si 85 1 No 2; Catene in copertura Si 86 1 No 2; Carico perman. coper. p_c (t/mq) 87; Lungh. appoggio coper. l_s (m) 90; Perimetro copertura l (m) 93
10	ELEM. NON STRUTT.	20	31	(Vedi manuale)
11	STATO DI FATTO	21	32	(Vedi manuale)

SCHEMI – RICHIAMI

Parametro 3. Resistenza convenzionale

Tipologia strutture verticali τ_x (t/mq)

Minimo tra A_x ed A_y A (mq)

Massimo tra A_x ed A_y A (mq)

Coeff. $a_0 = A/A_t$ Coeff. $\gamma = B/A$

$q = (A_x + A_y)\, h\, p_m / A_t + p_s$

$$C = \frac{a_0 \tau_k}{qN} \sqrt{1 + \frac{qN}{1{,}5\, a_0\, \tau_k\, (1+\gamma)}}$$

$\alpha = C/0{,}4$

Parametro 6. Configurazione planimetrica

$\beta_1 = a/l$ $\beta_2 = b/l$

Parametro 7. Configurazione in elevazione

Parametro 9. Copertura

Fig. 4.1.2 Second level GNDT form

REFERENCE: Criteri per l'esecuzione delle indagini, la compilazione delle schede di vulnerabilità II livello GNDT/CNR e la redazione della relazione tecnica (Regione Toscana, 2004)

The form collects information about eleven parameters that describe the building. For each parameter the form requires to indicate the class and the quality of the information used to define the class. For each category, the manual describes the method to evaluate correctly the parameter.

The second level G.N.D.T. form was initially used to calculate the Normalized GNDT index (between 0 and 1), which determines the vulnerability of a single building as a function of the eleven representative parameters, considering the propensity of the building to be damaged from a seismic event.

Actually, Vulnus considers only seven of the eleven parameters collected through the forms to calculate the I3 index, discarding those that are implicitly evaluated by I1 and I2 (Tab. 4.1.8). The I3 index is a normalized measure of the structural weaknesses of the building. If I3 = 0 the building respects the earthquake standards and it is characterized by good state of preservation. The I3 calculation is the same for isolated buildings and clustered buildings.

PARAMETERS	RELATION WITH I1 AND I2	I3 WEIGHT
Resistant system type and organization	I2	0.00
Resistant system quality	partially I1 and I2	0.15
Conventional resistance	I1	0.00
Building and foundation position	No	0.75
Horizontal structures	partially I2	0.50
Planimetric configuration	I1	0.00
Altimetric configuration	partially I2	0.50
D_{MAX} walls	I2	0.00
Roof	partially I2	0.50
Non-structural elements	No	0.25
Status quo	partially I1 and I2	0.50

Fig. 4.1.8: Parameters to calculate the I3 index

4.1.10 Procedure for vulnerability calculation

Calculated I1, I2 and I3 indices, it is possible to progress with the vulnerability analysis. These parameters are transformed into fuzzy subsets of their definition range. This way the uncertainty related to the estimation of quantities that are difficult to measure directly (for example the foundations depth), the variability of physical parameters characterizing the materials (for example the compressive strength) and any inaccuracies or errors in the detection process are considered.

Special data structures called fuzzy variables are introduced. The fuzzy set theory allows to treat concepts without exact boundaries where the transition between an element belonging to the set and one that does not belong to it is not clear, but gradual. A deterministic model is used to calculate the vulnerability, applied to fuzzy quantities. The model calculates with hyperbolic function the probability of survival f_s, or the complementary probability of collapse Vu (vulnerability). Fig 4.1.3 represents the function that describes the vulnerability: it divides the plane into the " certainly safe zone ", with Vu = 0, and the "certainly insecure zone" with Vu = 1. There is a transition zone in which the value of Vu, variable between 0 and 1, become the “probability of collapse” of the construction compared to I1, I2 and A variables. In this zone the curves characterized by a constant Vu value constitute a family of hyperbolas for which the parameter u ($0<u<1$) or Vu are constant between the boundary of the safe and unsafe zones. The vulnerability function implies a perfect symmetry of the effects of I1 and I2 and therefore of the strength compared to the condition I1 = I2.

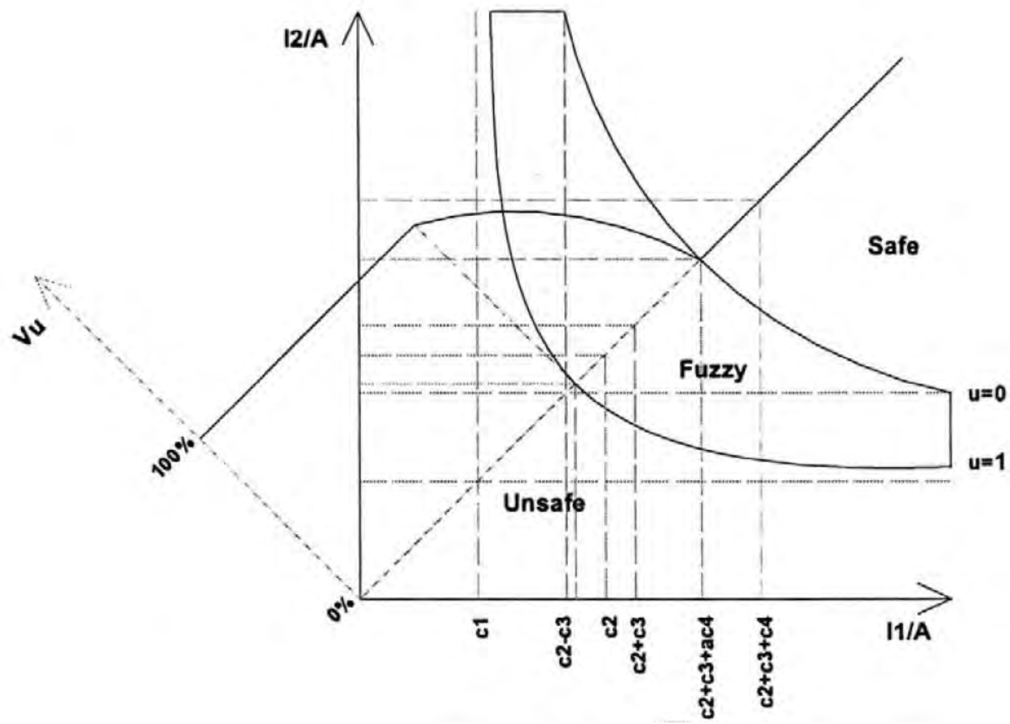


Fig. 4.1.3 Representation of the vulnerability function

REFERENCE: Manuale d'uso del programma Vulnus 4.0 (Università degli studi di Padova , 2009, p. 50)

The parameter *a*, defined in the range variable between 0 and 1, determines the amplitude of the transition zone, summarizing the influence of building qualitative factors (the parameters of the second level GNDT form). Even the same values of Vu depend on *a*: with constant I1 and I2, the function Vu grows monotonously with *a*.

From the cooperation between the estimated Vulnus results and the real damage observed on the buildings, the model was calibrated:

- c1 = 0.5 fixed the asymptotes of the hyperbolic function and therefore implies that, when A> min (I1 / 2, I2 / 2), it is certainly the appearance of serious damage;
- c2 =1.0 and c3 =0.1 imply that, in the condition of symmetry (I1 = I2), if A is equal to the values witch trigger the collapse mechanism (with an uncertainty of the model equal to 10%), this is the central values that separates the safety from the insecurity range;

- c4 = 1.0 implies that, always in condition of symmetry (I1 = I2), when A is less than half of the values which trigger the collapse mechanisms, is certainly to exclude the presence of serious damage.

The hypothesis of interaction of hyperbolic type extends the conclusions to the case where I1 and I2 assume different values.

4.2 APPLICATION OF THE METHODOLOGY TO THE BLOCKS

4.2.1 Description of the blocks

The Vulnus methodology has been applied to three blocks of the historical center of Timisoara: C3, C4 and C23 (Fig. 4.1.4). Blocks C3 and C4 are both masonry buildings and RC buildings while block C23 is composed by all masonry buildings, excepting for US 134 that is characterized by a bearing structure of concrete pillars. Vulnus software does not evaluate RC buildings so they are not included in this analysis. The relationship of the evaluated units with the omitted ones is still considered with the parameter "adjacent building stories number", in which is indicated the relative height of the adjacent unit respect to the analyzed unit, and with the parameter "wall thickness", that can be calibrated to consider adjoining or common walls between two neighboring units.

Block C3 has 11 buildings, of which 4 are masonry buildings and 7 are RC buildings. Block C4 has 10 buildings and just one is a RC building. Block 23 has 8 buildings and just US 134 has an RC structural system that can not be evaluated in Vulnus.

No reinforcing elements are considered in the analysis, due to both their uncertain activation and to choose the worst situation possible, in favor of safety.. The same building can be subdivided in more structural units if it has parts with different stories number or if it has big dimensions.[7]

[7] The maximum number of walls that ca be put in the program is 21. If a building has more than 21 walls it has been divided in more structural units.

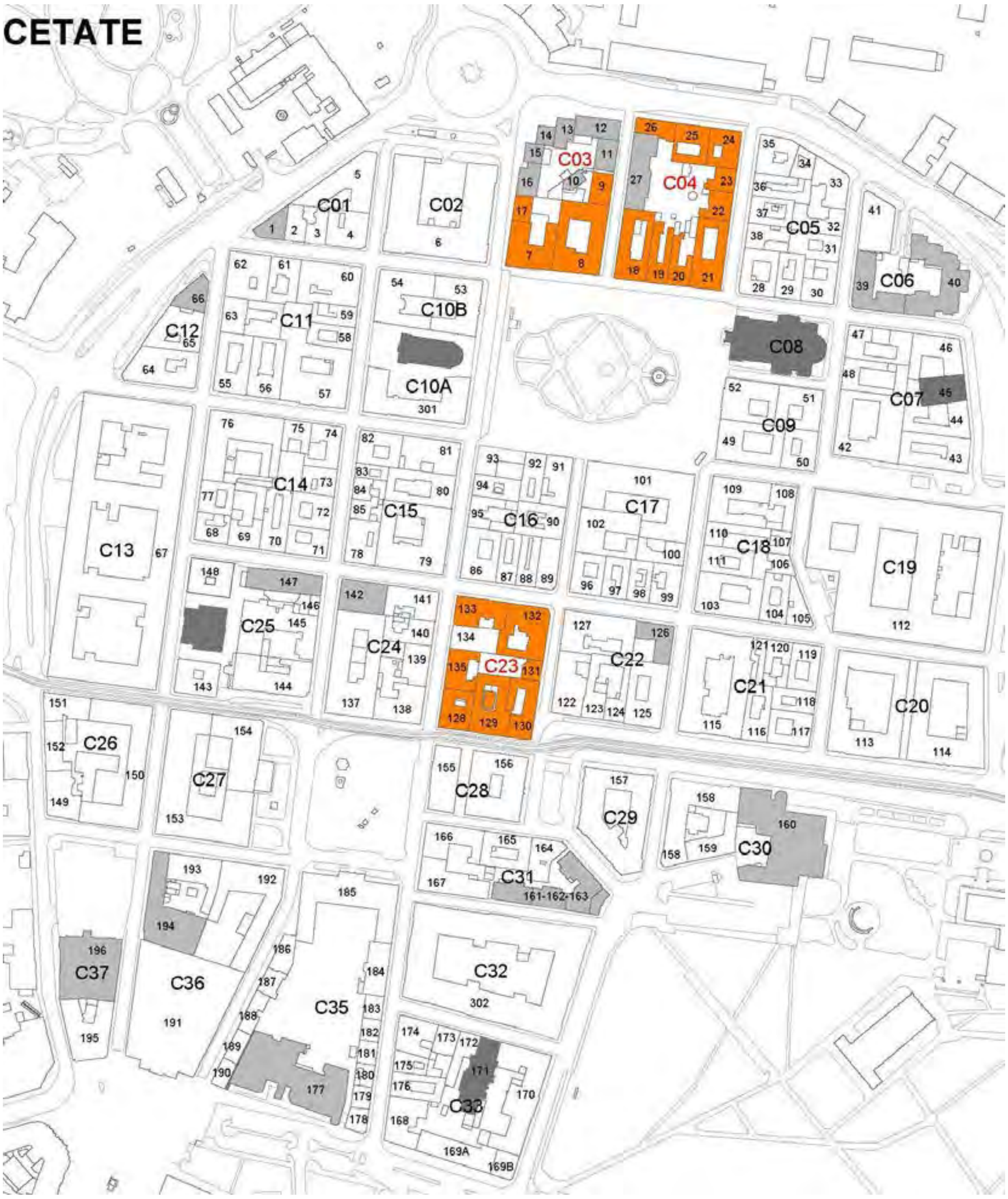


Fig. 4.1.4: Analyzed blocks

To apply Vulnus methodology it is important to have a plan of the building to define walls, septa and nodes which are necessary to represent the analyzed structural units. For this purpose the map of the historical center, already used to define plan modules in chapter 2.4.4, has been used. The definition of walls and septa is calibrated following the plan modules already defined in chapter 2.4.4, to better understand the structural system of the building. Septum length, wall direction and number, plan area and façade length are so defined. Openings length and walls extremity shoulders are defined using the same map, even if the representation is not very detailed. All the others parameters are defined using the data collected in situ, like building height, horizontal structures and wall thickness. The study is indeed aimed to give a vulnerability evaluation starting from the building characteristics that can be relived with a rapid survey.

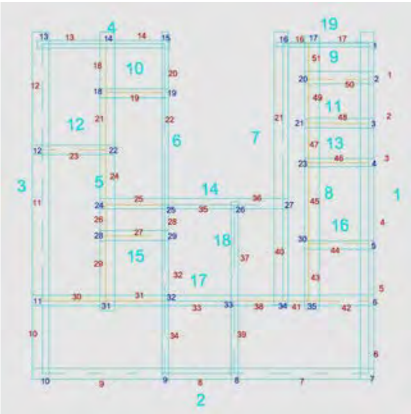
The assumptions made to complete the analysis are the following:

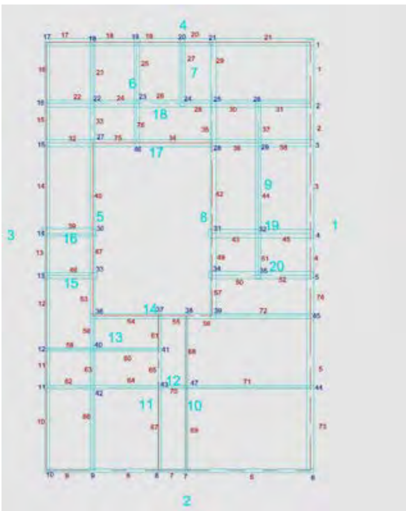
- Material conservation: A and B ratings in the survey form correspond to parameter 2 (good) in Vulnus, C corresponds to 3 and D corresponds to 4.
- Stories number: in this parameter the presence of basement must be considered.
- Permanent load on horizontal structure: the prevalent horizontal structure must be considered; timber structures and iron beams and little brick vaults structures correspond to a light load of 3 kN/m^2 while concrete slabs correspond to a medium load of 4.5 kN/m^2. The vertical and horizontal vault thrust are defined using *Arco* software, as explained in chapter 3.3;
- Floor warping: warping direction is hypothesized following the most probable in plan disposition in case of timber structure or iron beams and little brick vaults; it is considered following both direction in case of concrete slab; it is considered usually perpendicular to the perimeter walls.
- Floors regularity: if the structural unit is one or two stories high it is considered regular (parameter 2), if it has 3 or more stories it is considered unidentified (parameter 1). The floor regularity considers the possibility of different warping direction in different floors. It was choose to consider parameter 2 in case of two stories height buildings because it is more probable that the warping direction coincides for each floor. It is considered unidentified in case of more than two floors because it is more probable the presence of an horizontal structure with a different warping direction.
- Friction coefficient: for walls perpendicular to warping direction the coefficient has a value of 0.15 and for walls parallel to warping direction it has a value of 0.05; if the warping has both directions the friction coefficient has a value of 0.15 in both directions.
- Wall thickness: for perimeter walls, ground floor and upper floors thickness are defined considering the values of the survey form; for internal walls the thickness decreases of 15 cm, except for principal walls;

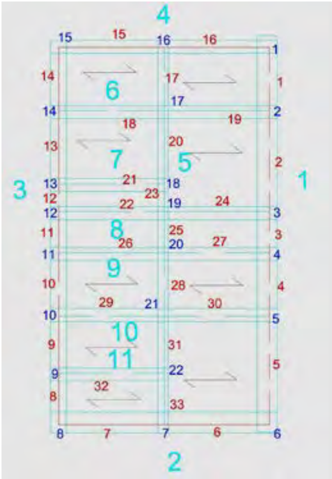
for internal walls the thickness decreases of 15 cm from ground floor to upper floors.

The following tables show the analyzed structural units for each block. They are part of the Vulnus form and the entire schedule can be found in annex C. The image represents the unit subdivision in walls (blue), septa (red) and nodes (dark blue), while the right part of the table contains informations about building characteristics and their evaluation following the criteria defined in chapter 4.1.2.

BLOCK C3

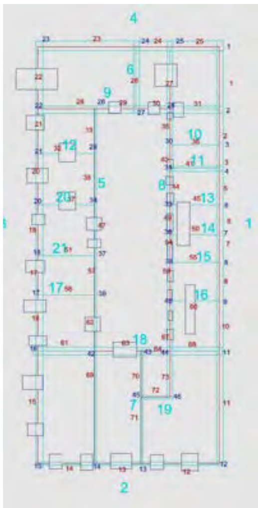
<table>
<tr><th>Building US 7 plan</th><th colspan="6">Building characteristics</th></tr>
<tr><td rowspan="6"></td><td>Walls material (A)
3</td><td>Material conservation (B)
2</td><td colspan="2">Stories number
2</td><td>Horizontal structures (C)
3</td><td>Plan regularity (D)
3</td></tr>
<tr><td>Building height (cm)
1110</td><td>Plan area (m^2)
618.8</td><td colspan="2">Floor warping (F)
2</td><td colspan="2">Floors regularity
2
(G) (irregular floor)</td></tr>
<tr><td colspan="6">Ring beams number :0</td></tr>
<tr><td colspan="6">Restraint on walls parallel to direction:
X Y</td></tr>
<tr><td>Friction coefficient µ (+)
0.05</td><td>Tie roads number</td><td>Façade lenght (cm)
2730</td><td>Friction coefficient µ (+)
0.15</td><td>Tie roads number</td><td>Façade lenght (cm)
2750</td></tr>
</table>

<table>
<tr><th>Building US 8 plan</th><th colspan="6">Building characteristics</th></tr>
<tr><td rowspan="6"></td><td>Walls material (A)
3</td><td>Material conservation (B)
2</td><td colspan="2">Stories number
2</td><td>Horizontal structures (C)
3</td><td>Plan regularity (D)
3</td></tr>
<tr><td>Building height (cm)
890</td><td>Plan area (m^2)
923</td><td colspan="2">Floor warping (F)
4</td><td colspan="2">Floors regularity
2
(G) (irregular floor)</td></tr>
<tr><td colspan="6">Ring beams number :0</td></tr>
<tr><td colspan="6">Restraint on walls parallel to direction:
X Y</td></tr>
<tr><td>Friction coefficient µ (+)
0.15</td><td>Tie roads number</td><td>Façade lenght (cm)
26411</td><td>Friction coefficient µ (+)
0.15</td><td>Tie roads number</td><td>Façade lenght (cm)
4227</td></tr>
</table>

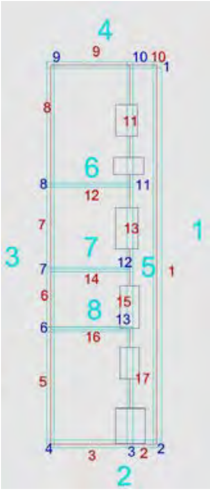
Building **US 9** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 2	Stories number 3	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1320	Plan area (m^2) 224.4	Floor warping (F) 2	Floors regularity 1 (G) (irregular floor)		
	Ring beams number :0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ 0.05	Tie roads number	Façade lenght (cm) 1119	Friction coefficient µ 0.15	Tie roads number	Façade lenght (cm) 2005

Building **US 17** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 2	Stories number 2	Horizontal structures (C) 4	Plan regularity (D) 3	
	Building height (cm) 850	Plan area (m^2) 123.7	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :2					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 897	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1519

BLOCK C4

Building **US 18** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 2	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 835	Plan area (m^2) 569	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor) 0		
	Ring beams number : 0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1834	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 4187

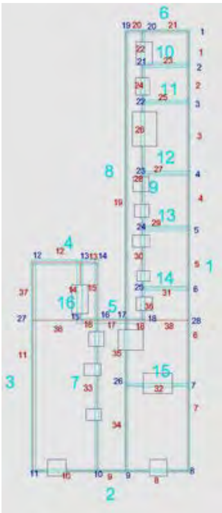
Building **19A** plan	Building characteristics



Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	1	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor) 0	
450	100	2		
Ring beams number :0				
Restraint on walls parallel to direction: X			Y	

Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.05		527	0.15		1890

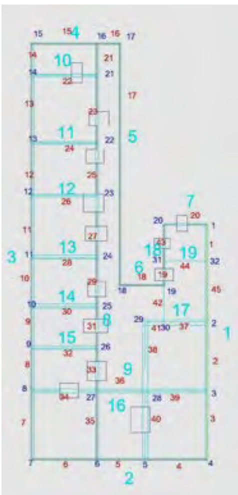
Building **US 19B** plan	Building characteristics



Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	2	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor) 0	
850	306.1	2		
Ring beams number :0				
Restraint on walls parallel to direction: X			Y	

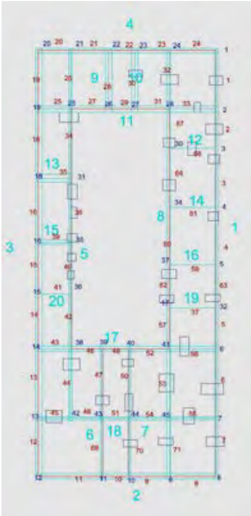
Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.05		1287	0.15		3598

Building **US 20** plan	Building characteristics

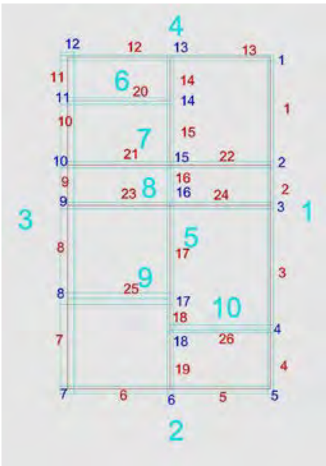


Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	2	3	3
Building heigth (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor) 0	
950	372.7	2		
Ring beams number :0				
Restraint on walls parallel to direction: X			Y	

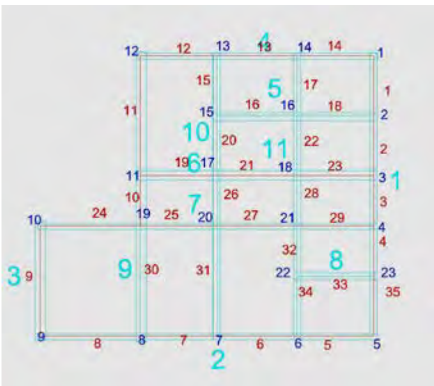
Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.05		1452	0.15		3435

Building **US 21** plan	Building characteristics				
	Walls material (A) 3	Material conservation (B) 2	Stories number 2	Horizontal structures (C) 4	Plan regularity (D) 3
	Building height (cm) 950	Plan area (m^2) 546.4	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor)	
	Ring beams number : 2				
	Restraint on walls parallel to direction:				

X: Friction coefficient μ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.15		1759	0.15		4187

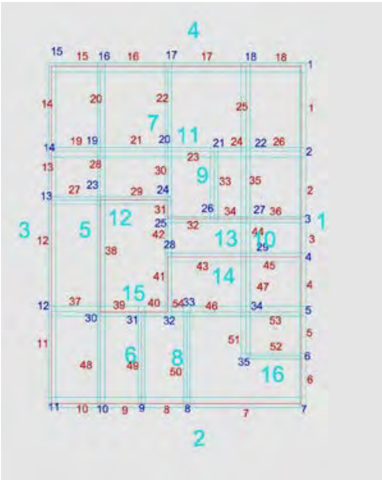
Building **US 22** plan	Building characteristics				
	Walls material (A) 3	Material conservation (B) 2	Stories number 1	Horizontal structures (C) 4	Plan regularity (D) 3
	Building height (cm) 380	Plan area (m^2) 170.1	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor)	
	Ring beams number :1				
	Restraint on walls parallel to direction:				

X: Friction coefficient μ	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ	Y: Tie roads number	Y: Façade lenght (cm)
0.15		1019	0.15		1669

Building **US 23** plan	Building characteristics				
	Walls material (A) 3	Material conservation (B) 2	Stories number 2	Horizontal structures (C) 4	Plan regularity (D) 3
	Building height (cm) 740	Plan area (m^2) 259.1	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor) 0	
	Ring beams number :1				
	Restraint on walls parallel to direction:				

X: Friction coefficient μ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.15		1759	0.15		1473

Building **US 24** plan



Building characteristics

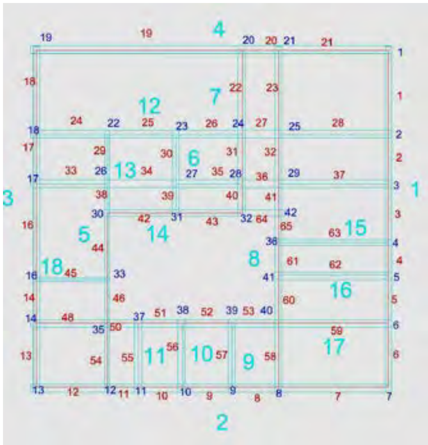
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	3	3	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) 1 (irregular floor) 0	
1400	313.3	4		

Ring beams number :0

Restraint on walls parallel to direction:

X: Friction coefficient µ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient µ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.15		1586	0.15		2144

Building **US 25** plan



Building characteristics

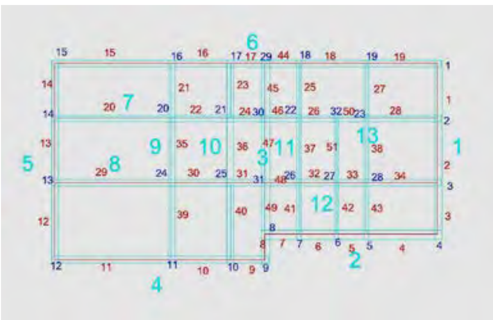
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	4	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) 1 (irregular floor) 0	
1530	482	3		

Ring beams number :

Restraint on walls parallel to direction:

X: Friction coefficient µ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient µ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.15		2248	0.05		2143

Building **US 26** plan



Building characteristics

Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	3	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) 1 (irregular floor) 0	
1270	295	3		

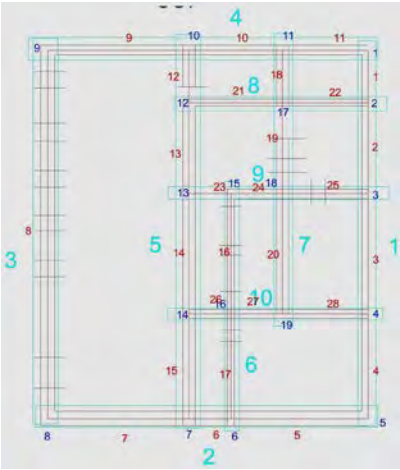
Ring beams number :0

Restraint on walls parallel to direction:

X: Friction coefficient µ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient µ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.15		2489	0.05		1261

BLOCK 23

Building **US 128** plan



Building characteristics

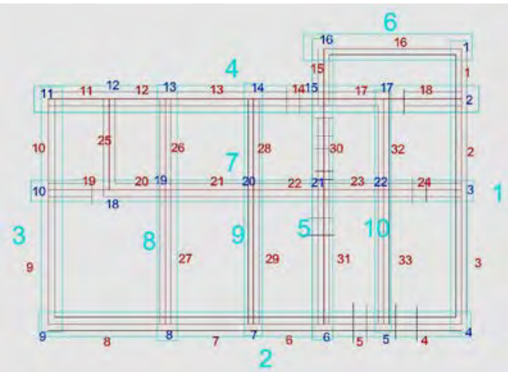
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	3	4	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor)	
1900	495.3	4		

Ring beams number :0

Restraint on walls parallel to direction:

X			Y		
Friction coefficient µ	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.05		2011	0.05		2324

Building **US 129A** plan



Building characteristics

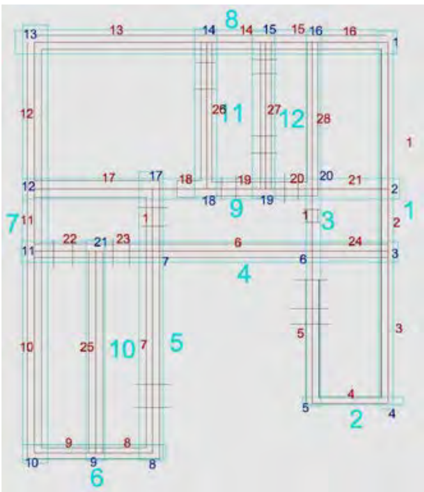
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	3	4	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor)	
1530	330.7	3		

Ring beams number :0

Restraint on walls parallel to direction:

X			Y		
Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.15		2286	0.05		1525

Building **US 129B** plan



Building characteristics

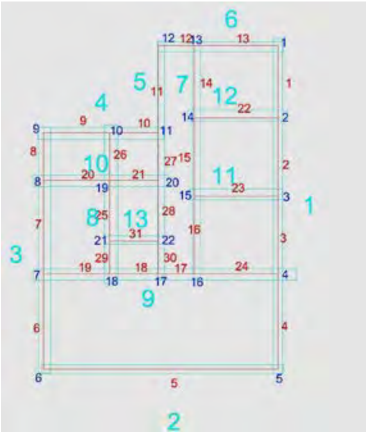
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	3	3	3	3
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity 2 (G) (irregular floor)	
1140	361.7	3		

Ring beams number :0

Restraint on walls parallel to direction:

X			Y		
Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)	Friction coefficient µ (+)	Tie roads number	Façade lenght (cm)
0.15		1941	0.05		2263

Building **US 130A** plan



Building characteristics

Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	4	3	3	3

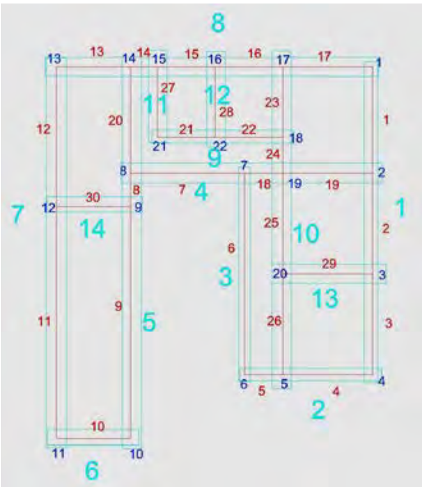
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) (irregular floor)
1160	245.4	2	2

Ring beams number :

Restraint on walls parallel to direction:

X: Friction coefficient μ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.05		1357	0.15		1865

Building **US 130B** plan



Building characteristics

Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	4	3	3	3

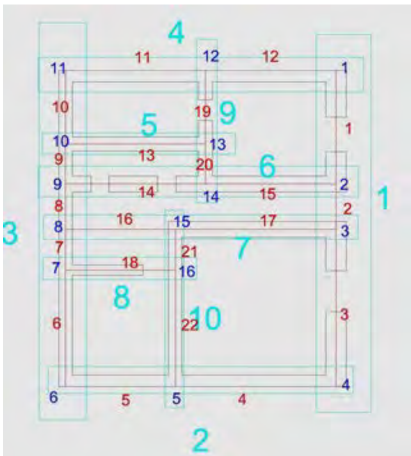
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) (irregular floor)
1160	249.3	2	2

Ring beams number :

Restraint on walls parallel to direction:

X: Friction coefficient μ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ (+)	Y: Tie roads number	Y: Façade lenght (cm)
0.05		1703	0.15		1988

Building **US 131** plan



Building characteristics

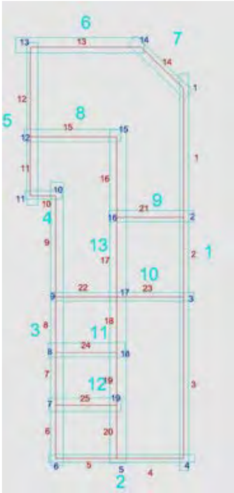
Walls material (A)	Material conservation (B)	Stories number	Horizontal structures (C)	Plan regularity (D)
3	2	3	3	3

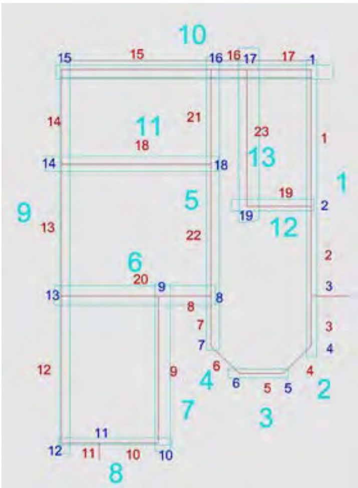
Building height (cm)	Plan area (m^2)	Floor warping (F)	Floors regularity (G) (irregular floor)
1160	174.5	1	1

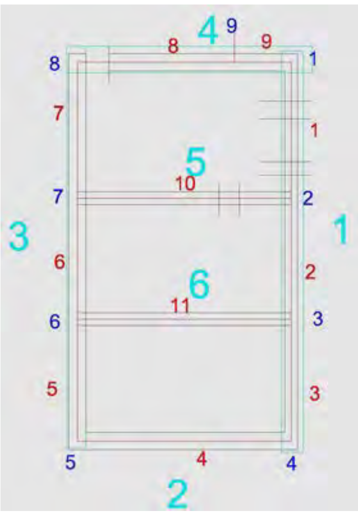
Ring beams number : 4

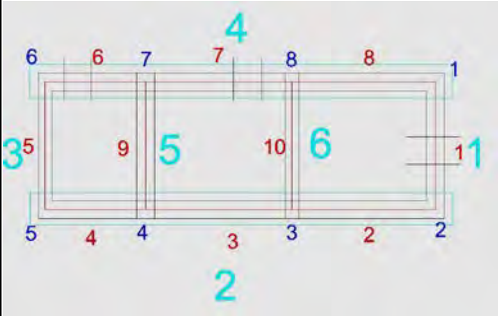
Restraint on walls parallel to direction:

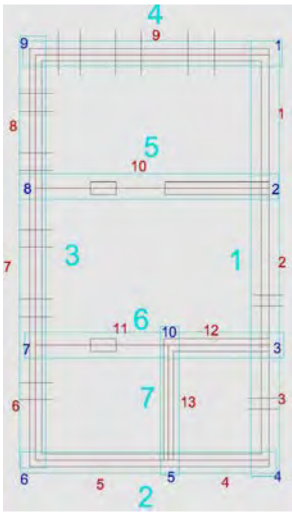
X: Friction coefficient μ (+)	X: Tie roads number	X: Façade lenght (cm)	Y: Friction coefficient μ (+)	Y: Tie roads number	Y: Façade lenght (cm)
		1188			1387

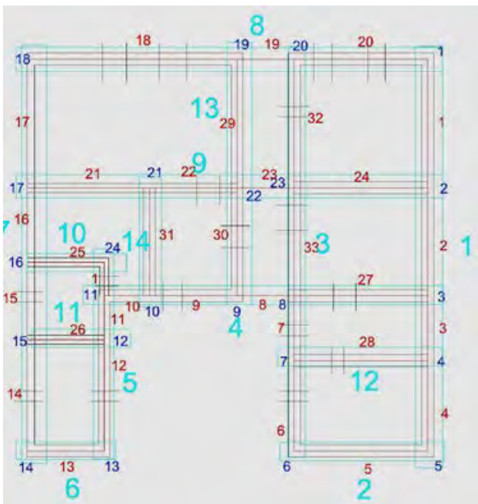
Building **US 132A** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 4	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1700	Plan area (m^2) 378	Floor warping (F) 2	Floors regularity 2 (G) (irregular floor)		
	Ring beams number : 0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 1220	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 3264

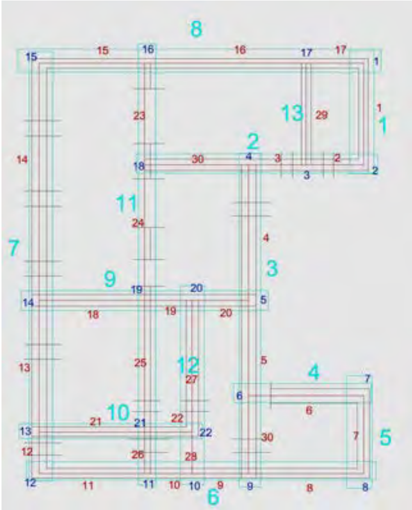
Building **US 132B** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 4	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1700	Plan area (m^2) 230	Floor warping (F) 3	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1294	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 1943

Building **US 132C** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 4	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1700	Plan area (m^2) 110.6	Floor warping (F) 2	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 748	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1321

Building **US 132D** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 5	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 2000	Plan area (m^2) 65.4	Floor warping (F) 3	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1296	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 425

Building **US 133A** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 4	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1390	Plan area (m^2) 233.2	Floor warping (F) 2	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 1134	Friction coefficient µ (+) 0.15	Tie roads number	Façade lenght (cm) 1943

Building **US 133B** plan	Building characteristics					
	Walls material (A) 3	Material conservation (B) 3	Stories number 4	Horizontal structures (C) 3	Plan regularity (D) 3	
	Building height (cm) 1390	Plan area (m^2) 339.4	Floor warping (F) 4	Floors regularity 2 (G) (irregular floor)		
	Ring beams number :0					
	Restraint on walls parallel to direction: X			Y		
	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 2006	Friction coefficient µ (+) 0.05	Tie roads number	Façade lenght (cm) 1943

<table>
<tr><td>Building US 135 plan</td><td colspan="6">Building characteristics</td></tr>
<tr><td rowspan="5"></td><td>Walls material (A)
3</td><td>Material conservation (B)
3</td><td>Stories number
4</td><td>Horizontal structures (C)
3</td><td colspan="2">Plan regularity (D)
3</td></tr>
<tr><td>Building height (cm)
1830</td><td>Plan area (m^2)
451.5</td><td>Floor warping (F)
2</td><td colspan="3">Floors regularity
2
(G) (irregular floor)</td></tr>
<tr><td colspan="6">Ring beams number :</td></tr>
<tr><td colspan="6">Restraint on walls parallel to direction:
X Y</td></tr>
<tr><td>Friction coefficient μ (+)
0.05</td><td>Tie roads number</td><td>Façade lenght (cm)
2011</td><td>Friction coefficient μ (+)
0.05</td><td>Tie roads number</td><td>Façade lenght (cm)
2567</td></tr>
</table>

4.2.2 Statistical analysis

The value of a_g is given by Romanian code and for the city of Timisoara is 0.20 a_g. Vulnus calculates automatically the value of I1 and I2 for each building of the aggregate (Tab. 4.2.1) and it executes a statistical analysis that is graphically visible using histograms (Fig. 4.2.2, Fig. 4.2.3, Fig 4.2.4). Finally, the program recapitulates the maximum and the minimum values of I1 and I2 and the buildings in which they are identified, the average value on the whole block, the mean square deviation and the variation coefficient (Tab. 4.4.2, Tab 4.4.3, Tab 4.4.4).

Block	US	I1	I2	I1/I2	Eq. specific density [kg/m^3]	Volume [m^3]	Weight [kg]
C3	7	0,384	0,11	3,468	2220	6869	2198
	8	0,589	0,222	2,646	2134	8215	3850
	9	0,519	0,262	1,975	2011	2962	2091
	17	0,678	0,326	2,08	2225	1051	623
C4	18	0,53	0,221	2,395	2072	4751	2856
	19 A	0,872	0,753	1,157	1968	450	379
	19 B	0,396	0,154	2,567	2056	3060	1903
	20	0,544	0,158	3,431	2017	3541	2250
	21	0,486	0,279	1,738	2087	5191	3635

	22	1,501	1,311	1,144	2172	646	459
	23	0,775	0,384	2,018	2316	1917	1100
	24	0,388	0,269	1,438	1990	4386	3225
	25	0,4	0,279	1,434	2144	7375	4086
	26	0,459	0,296	1,551	2012	3746	2730
C23	128	0,242	0,136	1,779	2020	9411	6272
	129 A	0,354	0,238	1,485	2045	5060	3888
	129 B	0,462	0,186	2,48	1993	4123	3885
	130 A	0,335	0,158	2,115	2024	2847	2000
	130 B	0,199	0,152	1,314	2121	2892	1639
	131	0,518	0,643	0,806	2020	2024	1605
	132 A	0,257	0,168	1,533	2227	6426	2832
	132 B	0,214	0,201	1,067	2124	4063	2198
	132 C	0,253	0,209	1,207	2150	1880	1067
	132 D	0,192	0,448	0,43	1987	1308	1268
	133 A	0,34	0,171	1,988	2335	3241	1428
	133 B	0,365	0,167	2,186	2141	4718	2948
	135	0,297	0,223	1,331	2042	8262	5299

Tab. 4.2.1: Basic data for the statistical analysis

As it can be seen in the histograms below, all the buildings of block C3 have an index I1 that is higher than index I2, that means that they have a greater vulnerability to out of plane mechanisms than to in plane mechanisms. This behavior is typical for historical clustered buildings. The US 17 has the maximum values of both I1 and I2: it is two stories high and very regular and compact both in plan and in elevation. The US 7 has the minimum values of both I1 and I2: it has long and widely open wall, which is vulnerable to in-plane mechanism and consequently with a lower I1, and it has large rooms and distanced internal walls, which decrease the out of plane resistance.

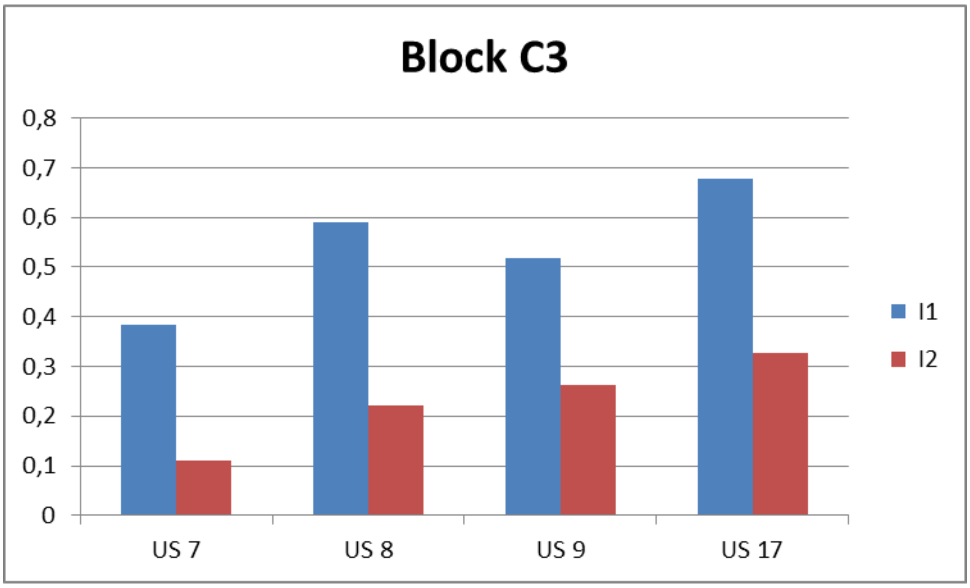


Fig. 4.2.2: Indexes I1 and I2 for Block C 3

Block C3		
	I1	I2
Maximum value	0,678	0,326
In building	US 17	US 17
Minimum value	0,385	0,111
In building	US 7	US 7
Average value	0,543	0,231
Weighted average value	0,51	0,194
Mean square deviation	0,107	0,078
Variation coefficient [%]	19,753	33,955

Tab. 4.2.2: Statistical analysis for block C3

As it can be seen in the histograms below, also all the buildings of block C4 have an index I1 that is higher than index I2 but the average value of the indices is higher than block C3, probably due to the reduced dimensions of the rooms compared with the previous block ones. In fact a building with smaller rooms is less vulnerable due to the major proximity of the resistant walls. Unit 22 has the highest values of the indices: it is one storey high and it has concrete horizontal structure and roof. These characteristics decrease its seismic vulnerability because their greater weight favors the stabilizing moment and the concrete ring beams favors the box behavior of the masonry structure. The minimum value of I1 corresponds to US 24, that it is characerized by widely holed walls and therefore by a low in plane resistance. The minimun value of I2 corresponds to US 19B, that it is characterized by a long (19m) and two stories high wall and therefore very vulnerable to out of plan mechanisms.

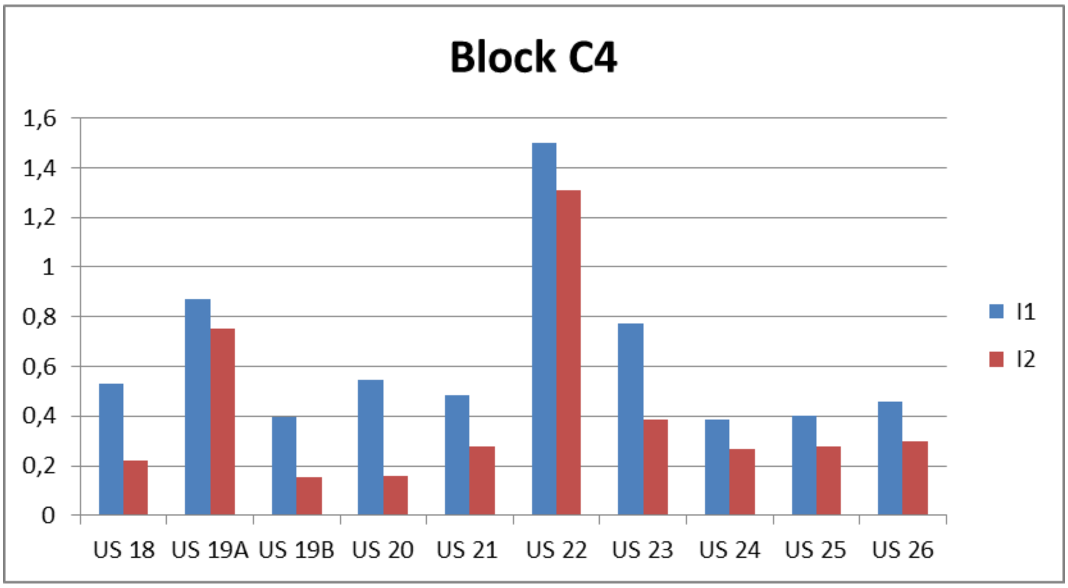


Fig. 4.2.3: Indexes I1 and I2 for Block C4

Block C4		
	I1	I2
Maximum value	1,502	1,311
In building	US 22	US 22
Minimum value	0,388	0,154
In building	US 24	US 19B
Average value	0,636	0,411
Weighted average value	0,497	0,267
Mean square deviation	0,327	0,341
Variation coefficient [%]	51,506	82,962

Tab. 4.2.3: Statistical analysis for block C4

Unlike blocks C3 and C4, not for all the buildings of block C23 the index I1 is bigger than index I2: US 131 and US 132 have an higher I2. US 131 has also the maximum values of both I1 and I2: it is the only building in the aggregate that is characterized by concrete horizontal structures. The minimum value of I1 corresponds to US 132D: it is five stories high and it has an elongated shape that decrease its in plan resistance. The minimum value of I2 corresponds to US 128: it is four stories high and at the ground floor it has a very large room that occupies almost half of the building area. The plan of the upper floors is not known but this large room probably does not repeat itself on other floors while the software evaluates the empty space as four stories high. In this situation the most vulnerable wall is four stories high and has a length of 23m with no perpendicular wall except for the perimeter ones. The extreme slender of the wall makes it very vulnerable to out of plane mechanisms. The average value is the lowest of the three blocks.

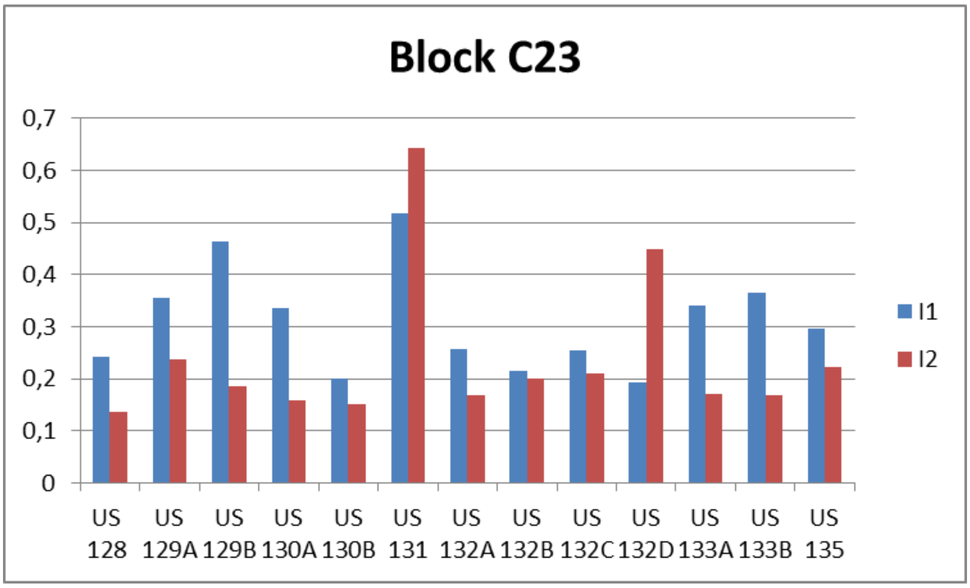


Fig. 4.2.4: Indexes I1 and I2 for Block C 23

Block C23		
	I1	I2
Maximum value	0,519	0,644
In building	US 131	US 131
Minimum value	0,193	0,136
In building	US 132D	US 128
Average value	0,31	0,239
Weighted average value	0,304	0,205

Mean square deviation	0,096	0,139
Variation coefficient [%]	30,824	58,329

Tab. 4.2.4: Statistical analysis for block C23

With Vulnus program it is also possible to approximately estimate the probability of survival in percentage of the buildings totality and to define the prevailing type of failure referring to I1 an I2 values (Tab. 4.2.5, Tab. 4.2.6, Tab. 4.2.7). Block C3, block C4 and block C23 have respectively a probability of survival of 75%, 80% and 38.46% for an a_g value of 0.2g. For blocks C3 and C4, the failure is for I2 (respectively with 25% and 20% of probability of collapse) while block 23 has the probability of failure both for I1 and I2. For this block, the probability of collapse for I1 corresponds to 7,69%, the probability of collapse for I2 corresponds to 46,15% and for both I1 and I2 to 7,7%.

Block C3				
Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
a/g = 0,02	75%	0%	25%	0 %

Tab. 4.2.5: Probabilistic analysis for block C3

Block C4				
Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
a/g = 0,02	80%	0%	20%	0%

Tab. 4.2.6: Probabilistic analysis for block C4

Block C23				
Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
a/g = 0,02	38,46%	7,69%	46,15%	7,7%

Tab. 4.2.7: Probabilistic analysis for block C23

4.2.3 Vulnerabilty analysis

The results of the II level GNDT form[8] are shown in Tab. 4.2.8. The results are necessary to determinate the value of I3 index (Tab. 4.2.9) and therefore for the seismic vulnerability analysis. Parameters about info quality of block C23 are

[8] *Criteri per l'esecuzione delle indagini, la compilazione delle schede di vulnerabilità II livello GNDT/CNR e la redazione della relazione tecnica* (Regione Toscana, 2004)

higher than the other two blocks thanks to the information given by local architect Bogdan Demetrescu about the structural typology of buildings of this block.

US	RESISTANT SYSTEM TYPE AND ORGANIZATION		RESISTANT SYSTEM QUALITY		CONVENTIONAL RESISTANCE		BUILDING AND FOUNDATION POSITION		HORIZONTAL STRUCTURES		PLANIMETRIC CONFIGURATION		ALTIMETRIC CONFIGURATION		D_{MAX} WALLS		ROOF		NON-STRUCTURAL ELEMENTS		STATUS QUO	
	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality
Block C3																						
7	D	B	C	B	D	M	C	M	D	B	D	M	A	M	D	M	B	B	D	M	B	B
8	D	B	C	B	C	M	C	M	D	B	B	M	A	M	D	M	B	B	D	M	B	B
9	D	B	C	M	C	M	C	M	D	B	C	M	A	M	A	M	B	B	C	M	B	M
17	D	B	C	M	A	M	C	M	D	B	C	M	A	M	A	M	B	B	C	B	B	B
Block C4																						
18	D	B	C	M	B	M	C	M	D	B	C	M	A	M	C	B	C	C	D	M	C	M
19A	D	B	C	B	B	M	C	M	D	B	D	M	A	B	A	M	B	B	D	B	A	B
19B	D	B	C	B	D	M	C	M	D	B	D	M	A	M	D	M	B	B	D	M	A	B
20	D	B	C	B	C	M	C	M	D	B	D	M	A	M	A	M	B	B	D	M	A	B
21	C	B	C	B	C	M	C	M	A	B	C	M	A	M	A	M	A	B	D	M	A	M
22	C	B	C	B	A	M	C	M	A	B	B	M	A	M	A	M	A	B	D	M	A	M
23	C	B	C	B	B	M	C	M	A	B	A	M	A	M	B	M	B	B	A	M	A	B
24	D	B	C	M	C	M	C	M	D	M	B	M	A	M	A	M	B	B	D	M	C	B
25	D	B	C	B	C	M	C	M	D	M	A	M	A	M	C	M	B	B	C	M	B	B
26	D	B	C	B	C	M	C	M	D	B	C	M	A	B	A	M	B	B	C	M	B	B
Block C23																						
128	D	E	C	M	A	E	C	M	D	E	A	E	A	E	D	E	B	E	C	E	C	E
129 A	D	E	C	M	C	E	C	M	D	E	B	E	A	E	A	E	B	E	C	E	B	E
129 B	D	E	C	M	C	E	C	M	D	E	D	E	A	E	A	E	B	E	C	E	B	E
130 A	D	E	C	M	C	E	C	M	D	E	D	E	C	E	B	E	B	E	C	E	D	E
130 B	D	E	C	M	D	E	C	M	D	E	D	E	A	E	D	E	B	E	C	E	D	E

131	B	E	A	E	B	E	C	M	B	E	A	E	A	E	B	E	B	E	A	E	A	E
132 A	D	E	C	M	D	E	C	M	D	E	D	E	A	E	C	E	B	E	A	E	B	E
132 B	D	E	C	M	D	E	C	M	D	E	D	E	A	E	B	E	B	E	A	E	B	E
132 C	D	E	C	M	D	E	C	M	D	E	A	E	A	E	B	E	B	E	A	E	B	E
132 D	D	E	C	M	D	E	C	M	D	E	D	E	A	E	B	E	B	E	A	E	B	E
133 A	D	E	C	M	B	E	C	M	C	E	A	E	A	E	B	E	B	E	C	E	B	E
133 B	D	E	C	M	C	E	C	M	D	E	D	E	A	E	B	E	B	E	C	E	B	E
135	D	E	C	M	C	E	C	M	D	E	C	E	A	E	B	E	B	E	C	E	B	E

Tab. 4.2.8: Results of the II level GNDT form

Block	US	I3	I GNDT	I GNDT Norm
C3	7	0,534392	262,5	0,686275
	8	0,534392	202,5	0,529412
	9	0,507937	217,5	0,568627
	17	0,507937	172,5	0,45098
C4	18	0,587302	221,25	0,578431
	19A	0,481481	191,25	0,5
	19B	0,481481	247,5	0,647059
	20	0,481481	213,75	0,558824
	21	0,269841	131,25	0,343137
	22	0,269841	78,75	0,205882
	23	0,243386	101,25	0,264706
	24	0,587302	228,75	0,598039
	25	0,507937	210	0,54902
	26	0,507937	217,5	0,568627
C23	128	0,613757	258,75	0,676471
	129A	0,613757	288,75	0,754902
	129B	0,26455	93,75	0,245098
	130A	0,455026	247,5	0,647059
	130B	0,455026	243,75	0,637255
	131	0,455026	221,25	0,578431
	132A	0,455026	243,75	0,637255
	132B	0,455026	168,75	0,441176
	132C	0,507937	228,75	0,598039
	132D	0,507937	221,25	0,578431
	133A	0,560847	183,75	0,480392

	133B	0,507937	210	0,54902
	135	0,507937	225	0,588235

Tab. 4.2.9: I3, I GNDT and I GNDT norm indices

4.2.4 Buildings vulnerability

The vulnerability assessment for each building is shown in Tab 4.2.10 and it depends from index I3 values. The qualitative scale that Vulnus program uses to define the vulnerability evaluation is[9]:

0 – very small

1 – small

2 – medium

3 – severe

4 – very severe

Block	US	a/g = 0.02
C3	7	VERY SEVERE
	8	MEDIUM
	9	MEDIUM
	17	SMALL
C4	18	MEDIUM
	19A	VERY SMALL
	19B	MEDIUM
	20	MEDIUM
	21	MEDIUM
	22	VERY SMALL
	23	VERY SMALL
	24	MEDIUM
	25	MEDIUM
	26	MEDIUM
C23	128	VERY SEVERE
	129A	MEDIUM
	129B	MEDIUM
	130A	MEDIUM
	130B	VERY SEVERE
	131	VERY SMALL
	132A	SEVERE

[9] *Manuale d'uso del programma Vulnus 4.0* (Università degli studi di Padova, 2009, p. 59)

	132B	SEVERE
	132C	MEDIUM
	132D	MEDIUM
	133A	MEDIUM
	133B	MEDIUM
	135	MEDIUM

Tab. 4.2.10: Vulnerability analysis for each building

The graphical representation (Fig. 4.2.5) shows that the vulnerability evaluation is very diversified. The evaluation is different both in different blocks and in same building of the same aggregate. In general the vulnerability judgment respects the buildings evaluations already made for maximum and minimum I indices: the vulnerability increases when the indices decrease.

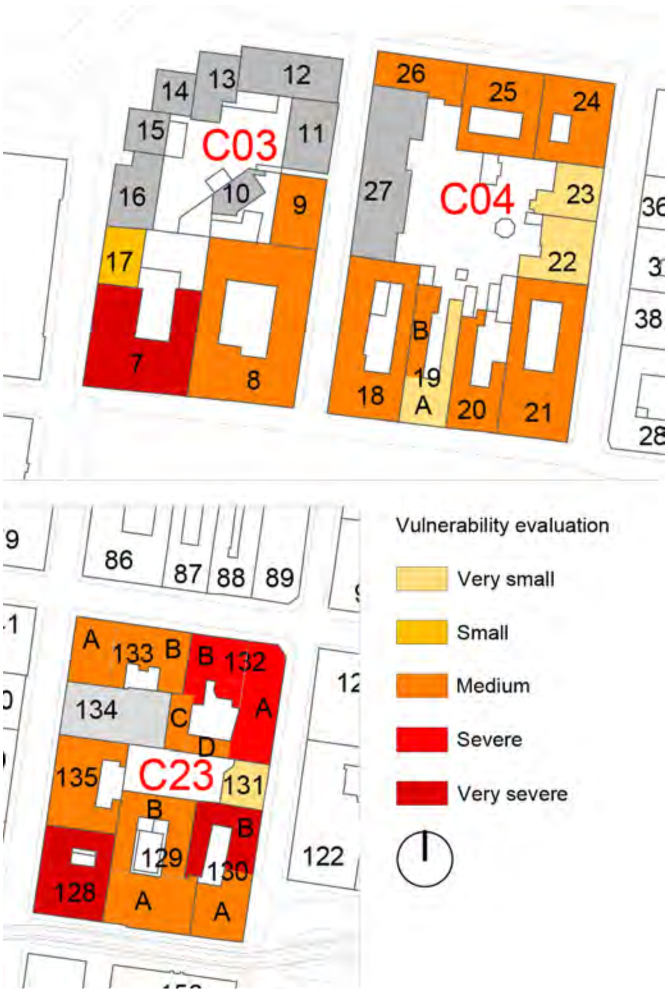


Fig. 4.2.5: Representation of the vulnerability assessment provided by Vulnus for a/g = 0.02

4.2.5 Group vulnerability

The group vulnerability is expressed by an identified function Vg and it is evaluated on a discrete scale from 0 to 100 with steps of 10%. Tables below show the probability that the group has to belong to a certain value of the vulnerability evaluation scale, referring to buildings (Tab. 4.2.11) or to volumes (Tab. 4.2.12).[10] The program provides a "medium" assessment for the vulnerability of blocks C3 and C23, while "small" to block C4. The vulnerability level referred to volumes is the same to the one referred to buildings for all the blocks.

Vulnerability identify level referred to buildings													
a/g	Block	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	C3	0	0	1	0,5	0,1	0	0	0	0	0	0	MEDIUM
	C4	0,5	1	0,899	0,5	0,1	0	0	0	0	0	0	SMALL
	C23	0	0	0,1	0,1	0,5	0,8	1	0,899	0,2	0	0	MEDIUM

Tab. 4.2.11: Group vulnerability for blocks C3,C4 and C23 referred to buildings

Vulnerability identify level referred to volumes													
a/g	Block	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	C3	0	0	1	0,5	0,1	0	0	0	0	0	0	MEDIUM
	C4	0,5	1	0,899	0,5	0,1	0	0	0	0	0	0	SMALL
	C23	0	0	0,1	0,1	0,5	0,8	1	0,899	0,1 18	0	0	MEDIUM

Tab. 4.2.12: Group vulnerability for blocks C3,C4 and C23 referred to volumes

[10] *Manuale d'uso del programma Vulnus 4.0* (Università degli studi di Padova, 2009, pp. 52-54)

4.2.6 Expected damage frequencies

The expected values of serious damage E[Vg] are defined in function of the different values of PGA/g ratio between peak ground acceleration and gravity acceleration, to obtain a further definition of the vulnerability. With E[Vg] values and the PGA/g (Tab. 4.2.13, Tab. 4.2.14, Tab. 4.2.15), Vulnus draws graphs constituted by fragility curves and in which is underlined the defined a/g value, that in this case corresponds to 0.2g (Fig. 4.2.6, Fig. 4.2.7, Fig. 4.2.8). Fragility curves are characterized by a lower, a central and an upper limit of the graph. The lower and upper curves delimit the area that defines the most probable values of expected damage frequencies of serious damage.

For a PGA/g value of 0.2, the average expectation of damage is 44% in block C3, with a range between 20% and 63%, 33% in block C4, with a range between 10% and 52%, and 67% in block C23, with a range between 46% and 83%.

C3	PGA/g				
	0	0,2	0,4	0,6	0,8
E[Vg] Low	0	0,199	0,718	0,99	1
E[Vg] White	0,02	0,437	0,87	1	1
E[Vg] Up	0,049	0,625	0,988	1	1

Tab. 4.2.13: Expectation values of damage for block C3

C4	PGA/g				
	0	0,2	0,4	0,6	0,8
E[Vg] Low	0	0,100	0,618	0,799	0,845
E[Vg] White	0,02	0,333	0,738	0,865	0,928
E[Vg] Up	0,049	0,524	0,839	0,915	0,961

Tab. 4.2.14: Expectation values of damage for block C4

C23	PGA/g				
	0	0,2	0,4	0,6	0,8
E[Vg] Low	0	0,463	0,936	0,981	1
E[Vg] White	0,02	0,671	0,962	0,988	1
E[Vg] Up	0,049	0,834	0,983	1	1

Tab. 4.2.15: Expectation values of damage for block C23

In the fragility curve graph (Fig. 4.2.6) for block C3 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: 0.01 ≤ PGA/g ≤ 0.1
  E[Vg] White: 0.01 ≤ PGA/g ≤ 0.05
  E[Vg] Up: 0.01 ≤ PGA/g ≤ 0.05
- Second phase: the lower and the central limits increase with similar and irregular slopes, while the upper limit grows with greater inclination compared to the other two curves.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: PGA/g ≥ 0.44
  E[Vg] White: PGA/g ≥ 0.58
  E[Vg] Up: PGA/g ≥ 0.64

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.

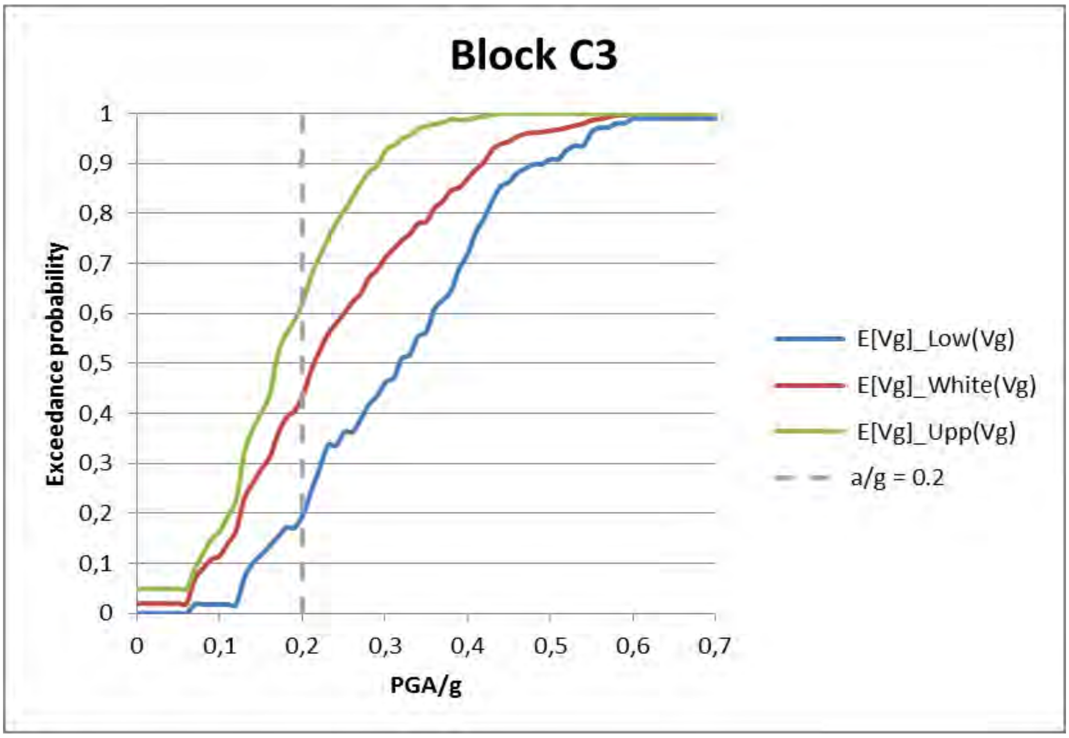


Fig. 4.2.6: Fragility curves for block C3

In the fragility curve graph (Fig. 4.2.7) for block C4 three phases can be identified:

- <u>First phase</u>: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: $0.01 \leq PGA/g \leq 0.1$
  E[Vg] White: $0.01 \leq PGA/g \leq 0.07$
  E[Vg] Up: $0.01 \leq PGA/g \leq 0.07$

- <u>Second phase</u>: the lower and the central limits increase with similar and irregular slopes, while the upper limit grows with greater inclination compared to the other two curves.

- <u>Third phase</u>: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after very high values of PGA/g:

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.

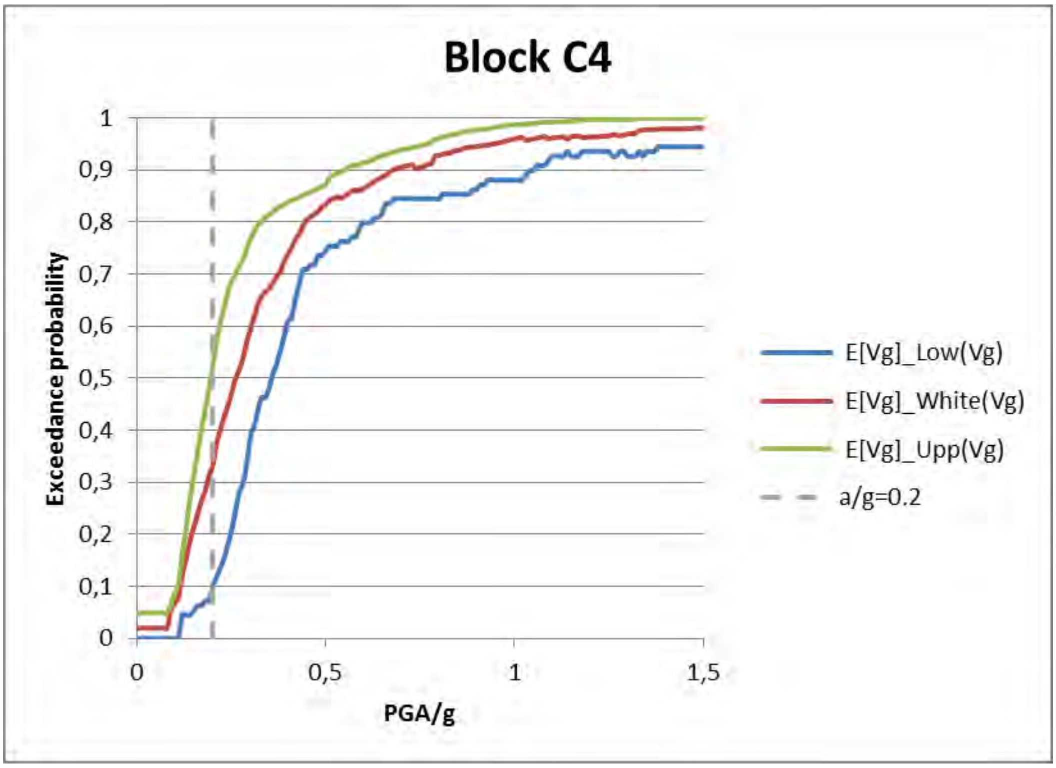


Fig. 4.2.7: Fragility curves for block C4

In the fragility curve graph (Fig. 4.2.8) for block C23 three phases can be identified:

- <u>First phase</u>: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: $0.01 \leq PGA/g \leq 0.06$
  E[Vg] White: $0.01 \leq PGA/g \leq 0.05$
  E[Vg] Up: $0.01 \leq PGA/g \leq 0.05$

- <u>Second phase</u>: the lower and the upper limits increase with similar and irregular slopes, while the central limit grows with lower inclination compared to the other two curves.

- <u>Third phase</u>: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: $PGA/g \geq 0.63$
  E[Vg] White: $PGA/g \geq 0.58$
  E[Vg] Up: $PGA/g \geq 0.48$

Therefore for the value of PGA/g of 0.2 the vulnerability is medium-high.

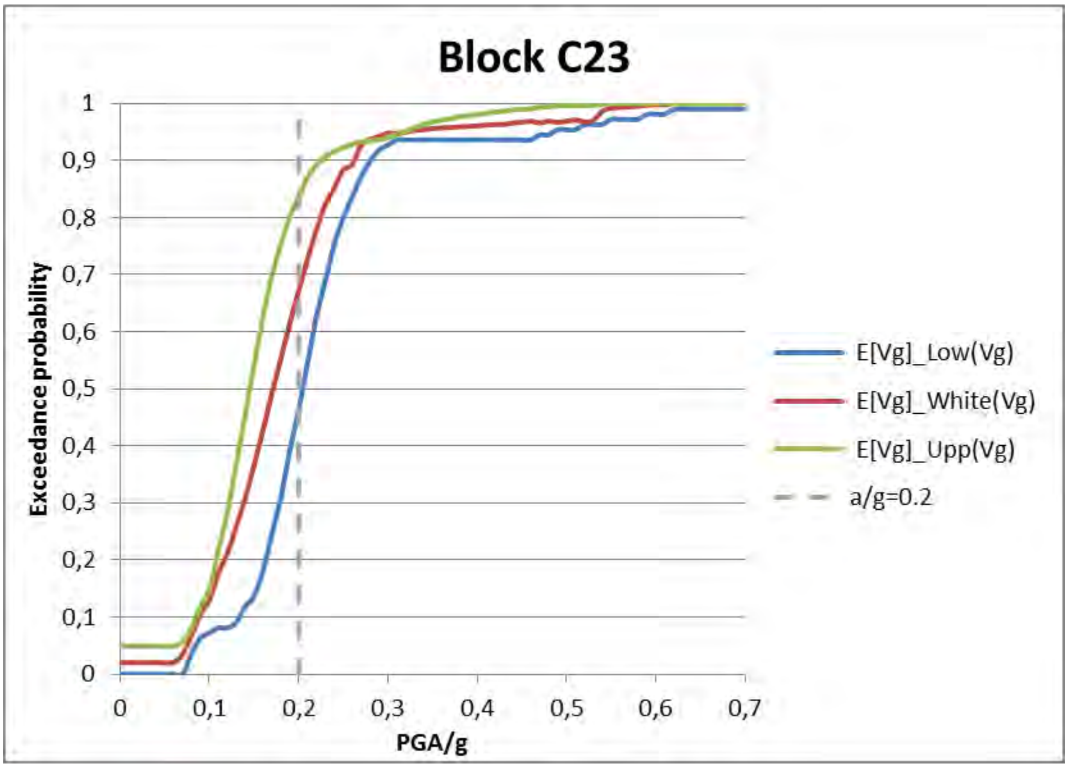


Fig. 4.2.8: Fragility curves for block C23

4.3 APPLICATION OF THE METHODOLOGY TO STRUCTURAL UNITS

4.3.1 Description of structural units

Vulnus methodology has been applied also to four structural units, of which we have detailed plans and sections, to compare the results obtained with different level of information. The applied level of information are:

- Real case: building dimensions and information about the structural typology and constructive details are obtained from plans and sections, usually in scale 1:50 or 1:100, of the building.[11]
- Survey case: information about structural typologies, wall thickness and interstorey high result from the in situ survey, while plan and openings dimension has been taken from the city plan of 1980 (see chapter 2.4.4)

The comparison between these two cases is finalized to underline eventual differences that can affect the results of the analysis and to valid the assumptions previously made to complete the Vulnus process. The informations collected in situ are indeed often incomplete, even about important building characteristics, and the city plan does not represent eventual interventions of the latest years. For this reason it is important to verifying that the adopted method does not deviate too much from the real case, comparing different cases with different level of information.

The analyzed units are: US 88, US 123, US 124 and US 130. The US 88 is located in the south side of block C16, between two buildings with the same stories number but one a little bit higher and one a little bit lower. US 123 and 124 are located side by side in the south side of block C22 and are contained between an higher building and a building of the same height. The US 130 is located in the south-east corner of block C23 between an higher building and a building of the same height. They all belong to typology 3, characterized by three stories, masonry vertical structures, deformable horizontal structures with brick vaults at the ground floor and timber roof. Despite that, they belong to three different micro-typologies: US 123 and US 124 belong to micro-typology 3B,

[11] The material has been provided by Arch. Bodgan Demetrescu

characterized by a wall thickness of 60 cm, US 130 belongs to micro-typology 3C, characterized by a wall thickness of 75 cm, and US 88 belongs to micro-typology 3D, characterized by a wall thickness of 90 cm. All the units have an average typological interstorey high of 4,3 m for the ground floor and 4,2 m for upper floors.

The assumptions made to complete the analysis are the same of chapter 4.2.1

Fig. 4.3.1: Analyzed structural units

The following images show the subdivision in walls, septa and nodes of the analyzed structural units. For each unit, the real plan is compared with the survey one and eventual differences can be observed.

As can be seen in Fig. 4.3.2, the survey case of US 88 has a sensible number of walls less than the real case, particularly around the internal courtyard. Also in US 123 (Fig. 4.3.3) the internal walls are objects of changings in position and number, while for US 124 (Fig. 4.3.4, Fig. 4.3.5) the scheme is almost identical. The US 130 (Fig. 4.3.6) has the same scheme for the real case and the survey case as well. Variations between the two cases are usually about the dimension and the position of openings and consequently about the shoulder regularity. Other important differences are the wall thickness both at the ground floor and at upper floors.

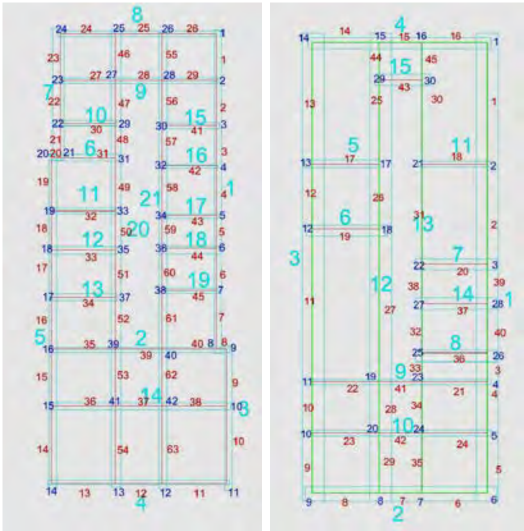


Fig. 4.3.2: Plan scheme of US 88 – Real (left) and survey (right)

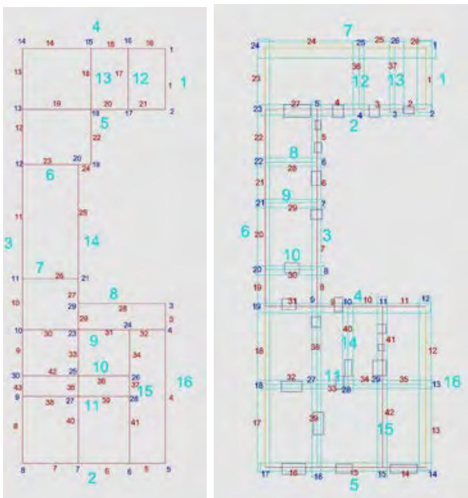


Fig. 4.3.3: Plan scheme of US 123– Real (left) and survey (right)

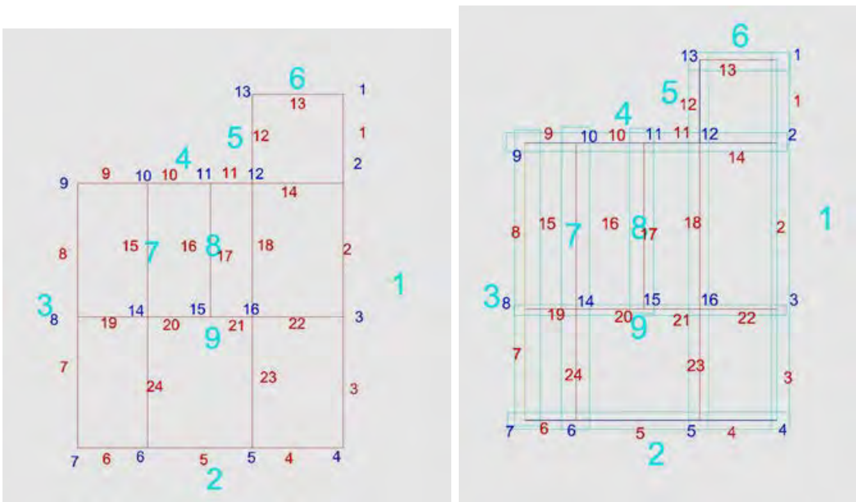


Fig. 4.3.4: Plan scheme of US 124A – Real (left) and survey (right)

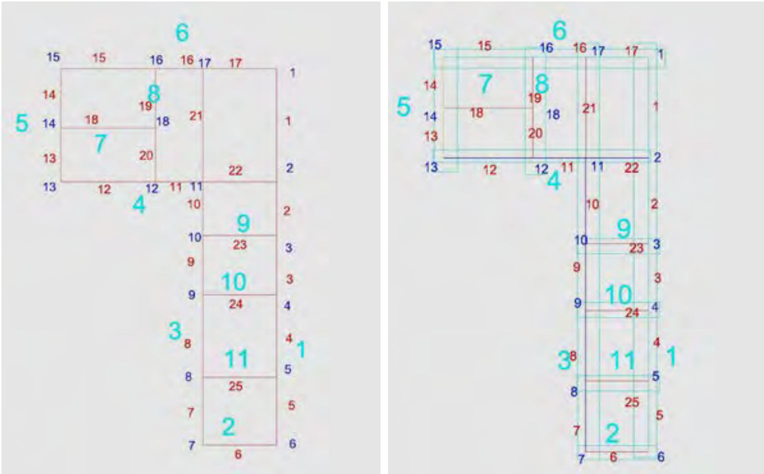


Fig. 4.3.5: Plan scheme of US 124B – Real (left) and survey (right)

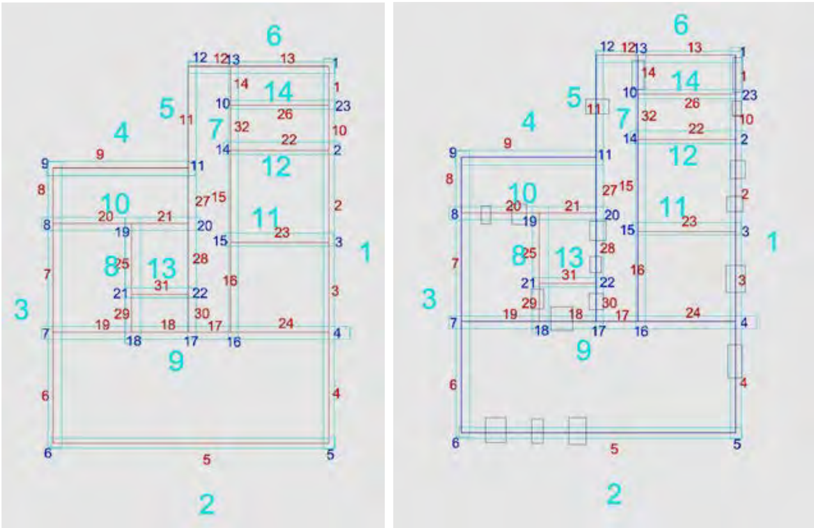


Fig. 4.3.6: Plan scheme of US 130A – Real (left) and survey (right)

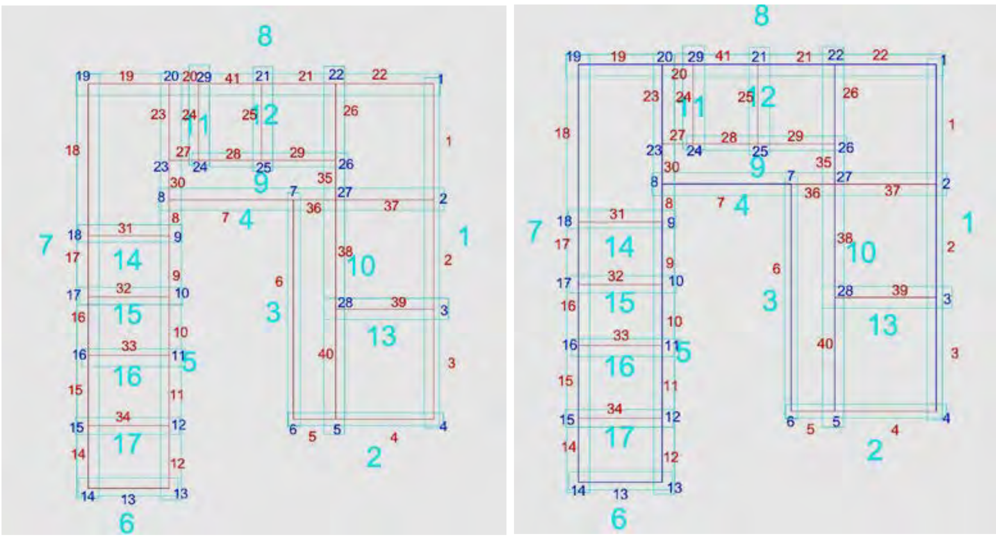


Fig. 4.3.7: Plan scheme of US 130B – Real (left) and survey (right)

4.3.2 Statistical analysis

As previously said, Vulnus calculates automatically the value of I1 and I2 for each building. Indices of the real case and the survey case have been compared for each structural unit (Tab. 4.3.1, Tab 4.3.2, Tab. 4.3.3, Tab. 4.3.4). The last line of tables, called “difference”, shows the difference between the real case and the survey case. With histograms (Fig. 4.3.9, Fig. 4.3.10, Fig. 4.3.11, Fig. 4.3.12) it is easily visible the trend of the graphs.

In the real case of US 88 the index I1 is smaller than the index I2 while in the survey case it is the opposite. I1 increases of 0.107 while I2 decrease of 0.318 from real to survey case. The real case is more vulnerable to in plane mechanisms and, on the opposite, the survey case is more vulnerable to the out of plane mechanisms. The differences between the two cases are:

- The thickness of external walls of the survey case is higher than the real one: 90 cm for the survey and 75 cm for the real case. This factor improve the in plan resistance of the survey case.
- The 1980 map of the city, that is the reference for survey plans, often does not represent internal walls that there are in reality, particularly in correspondence of vaults spans. This factor increases the vulnerability of the survey case to out of plan mechanisms.
- The dimension and the number of openings is higher in the real case, cause in the 1980 map the low quality of the representation tends to reduce the width of doors and windows. This factor increases the resistance to in plan mechanisms for the survey case.

Block	US	I1	I2	I1/I2	Eq. specific density [kg/m^3]	Volume [m^3]	Weight [kg]
C16	88 REAL	0,343	0,533	0,643	2066	3704	2161
	88 SURVEY	0,45	0,215	2,089	1955	3562	3466
	DIFFERENCE	+0,107	-0,318	+1,446	-111	-142	+1305

Tab. 4.3.1: Data for the statistical analysis-US 88

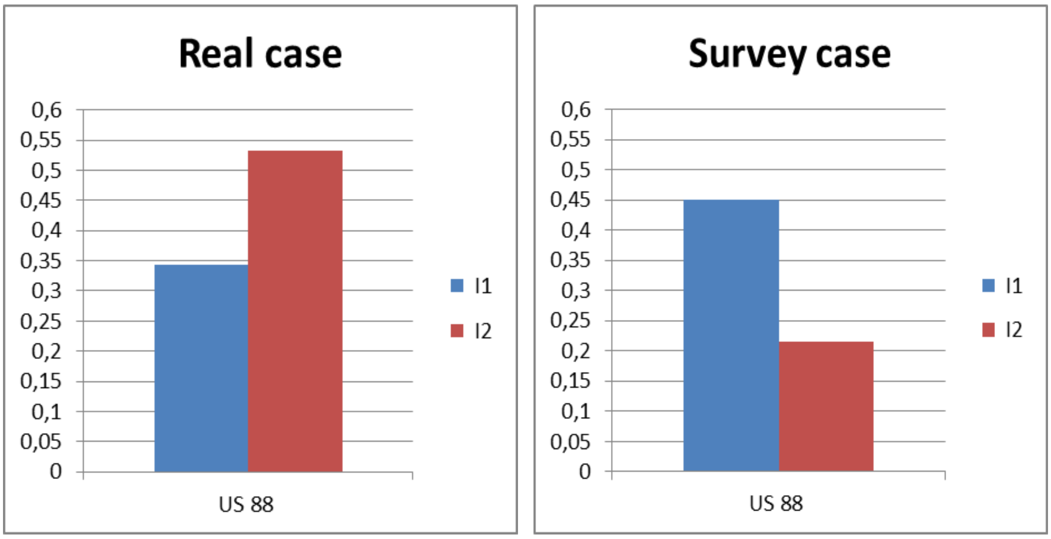
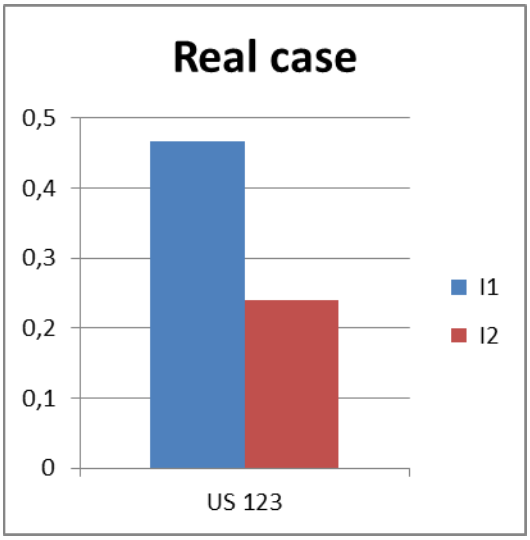


Fig. 4.3.9: Indices I1 and I2 for US 88

For US 123 both cases have the I1 index higher than the I2 index: a lower I2 indicates an higher vulnerability to out of plane mechanisms. The value of I1 is almost the same for the two cases, while the value of I2 increase of 0.119 for the survey case. The higher value of I2 in the survey case is justified by the representation of internal walls that do not exist anymore in the real case. The presence of these walls, perpendicular to the wall object of the mechanism, decrease the length of the overturning wall and consequently they increase its resistance.

Block	US	I1	I2	I1/I2	Eq. specific density [kg/m^3]	Volume [m^3]	Weight [kg]
C22	123 REAL	0,467	0,24	1,945	2201	3912	1938
	123 SURVEY	0,429	0,359	1,194	2222	3911	1896
	DIFFERENCE	-0,038	+0,119	-0,751	+21	-1	-42

Tab. 4.3.2: Data for the statistical analysis-US 123



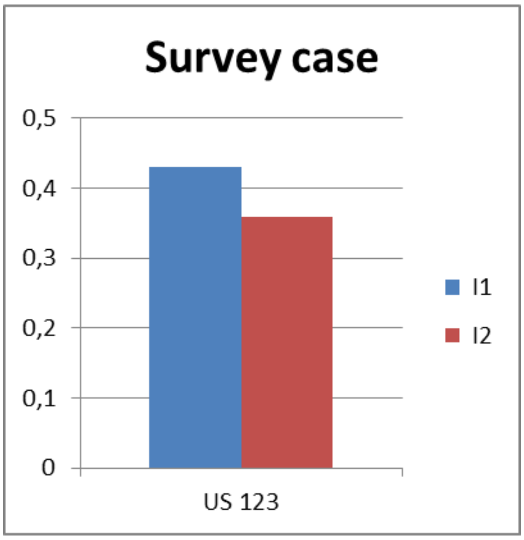


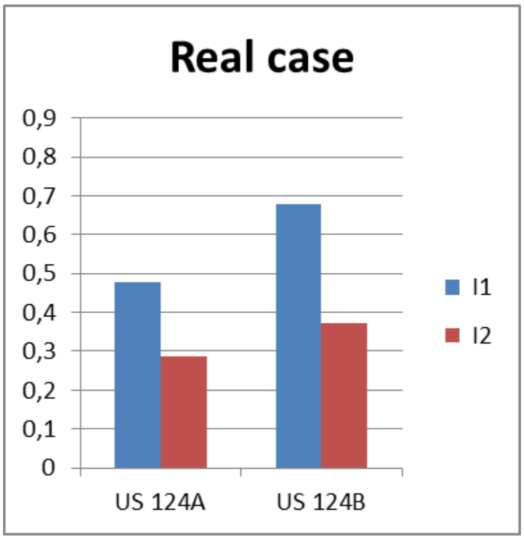
Fig. 4.3.10: Indices I1 and I2 for US 123

US 124 has been divided in two structural units because the front part on the main street is three stories high (US 124A) while the back part on the internal courtyard is two stories high (US 124B). In fact a structural unit must have the same stories number in all its part, in case of different stories number or different height, the building must be subdivided in more structural units. The I1 index for US 124A is the same in both cases while it decreases of 0.151 in the survey case for US 124B. The I2 index increases sensibly in the survey case (0.524 for US 124A and of 0.183 for US 124B). The anomalous difference of I2 index in US 124A is justified by the presence in the real case of a wall, with numerous and wide openings, on which a low barrel vault insists. This situation increases sensibly the vulnerability to out of plane mechanisms due to high value of the horizontal thrust which favors the mechanism activation.

Block	US	I1	I2	I1/I2	Eq. specific density [kg/m^3]	Volum e [m^3]	Weight [kg]
C22	124A REAL	0,478	0,286	1,668	2012	1380	1206
	124A SURVEY	0,455	0,81	0,562	2061	1404	1111
	DIFFERENCE	-0,023	+0,524	-1,106	+49	+24	-95
C22	124B REAL	0,679	0,372	1,822	2042	929	759

	124B SURVEY	0,528	0,555	0,95	2100	728	503
	DIFFERENCE	-0,151	+0,183	-0,872	+58	-201	-256

Tab. 4.3.3: Data for the statistical analysis-US 124



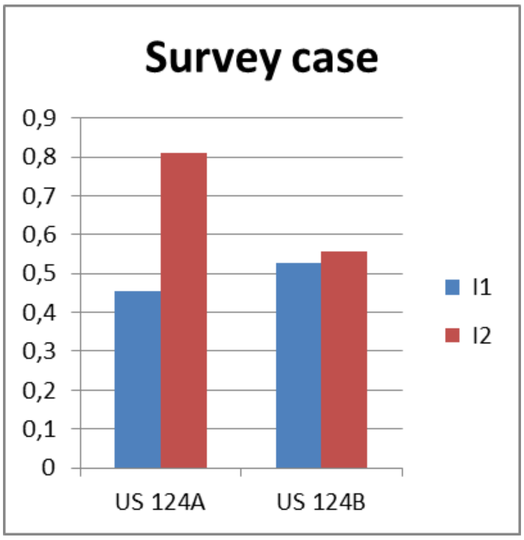


Fig. 4.3.11: Indices I1 and I2 for US 124

The US 130 has been divided in two structural units because the walls number exceeded 21, number accepted by Vulnus program. The front side towards the street is US 130A while the back side towards the internal courtyard is US 130B. The indices related to US 130A remain the same in both the analyzed cases. While the index I2 of US 130B remain the same, the index I1 increases in the survey case due to the higher thickness of walls and the lower dimensions of the openigns.

Block	US	I1	I2	I1/I2	Eq. specific density [kg/m^3]	Volume [m^3]	Weight [kg]
C23	130A REAL	0,324	0,156	2,075	2026	2847	1974
	130A SURVEY	0,351	0,176	1,992	2007	2847	2408
	DIFFERENCE	+0,027	+0,02	-0,083	-19	=	+434
C23	130B REAL	0,224	0,258	0,869	2093	2892	1763

	130B SURVEY	0,317	0,236	1,342	1949	2892	3334
	DIFFERENCE	+0,093	+0,022	+1,255	-144	=	+1571

Tab. 4.3.4: Data for the statistical analysis-US 130

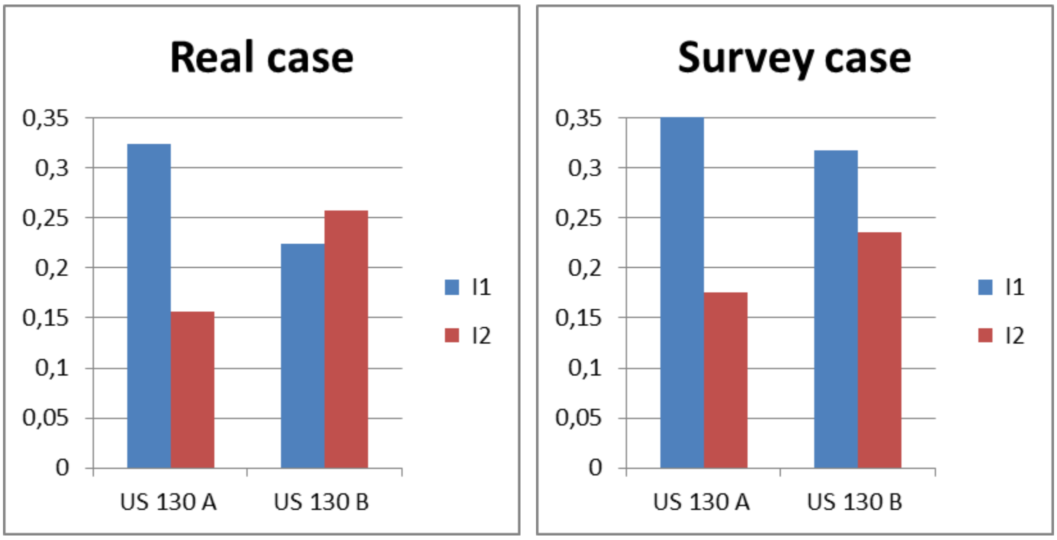


Fig. 4.3.12: Indices I1 and I2 for US 130

The probabilistic analysis shown in the following tables (Tab. 4.3.5, Tab. 4.3.6, Tab. 4.3.7, Tab 4.3.8) is helpful to define the probability of survival in percentage and to define the prevailing type of failure referring to I1 and I2 values. For both the cases (the real one and the survey one) US 88, 123 and 124 have the 100% probability of survival while US 130 has a probability of survival of 50% with collapse for out of plane mechanisms.

US 88	Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
REAL	a/g = 0,02	100%	0%	0%	0%
SURVEY	a/g = 0,02	100%	0%	0%	0%

Tab. 4.3.5: Probabilistic analysis for US 88

US 123	Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
REAL	a/g = 0,02	100%	0%	0%	0%
SURVEY	a/g = 0,02	100%	0%	0%	0%

Tab. 4.3.6: Probabilistic analysis for US 123

US 124	Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
REAL	a/g = 0,02	100%	0%	0%	0%
SURVEY	a/g = 0,02	100%	0%	0%	0%

Tab. 4.3.7: Probabilistic analysis for US 124

US 130	Probability of	Survival I1>a/g; I2 >a/g	Collapse for I1 I1<a/g; I2 >a/g	Collapse for I2 I1>a/g; I2 <a/g	Collapse for I1, I2 I1<a/g; I2 <a/g
REAL	a/g = 0,02	50%	0%	50%	0%
SURVEY	a/g = 0,02	50%	0%	50%	0%

Tab. 4.3.8: Probabilistic analysis for US 130

4.3.3 Vulnerability analysis

The results of the II level GNDT form are shown in Tab. 4.3.9. Classes and info quality are necessary to determinate index I3 in the vulnerability analysis. The parameters class usually does not change between the real case and the survey case, but the info quality often does. The info quality of the parameters planimetric configuration, D_{MAX}[12] walls and roof usually decreases of one level of quality from the real case to the survey case.

Values of I3 index, I GNDT and I GNDT norm are given in Tab. 4.3.10.

[12] It is the maximum walls distance and it is evaluated as a ratio between the orthogonal walls and the thickness of the analyzed wall.

CASE	RESISTANT SYSTEM TYPE AND ORGANIZATION		RESISTANT SYSTEM QUALITY		CONVENTIONAL RESISTANCE		BUILDING AND FOUNDATION POSITION		HORIZONTAL STRUCTURES		PLANIMETRIC CONFIGURATION		ALTIMETRIC CONFIGURATION		D_{MAX} WALLS		ROOF		NON-STRUCTURAL ELEMENTS		STATUS QUO	
	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality	Class	Info Quality
US 88																						
real	D	M	C	M	D	E	C	M	D	M	D	E	A	E	A	E	B	M	C	E	C	M
survey	D	B	C	B	D	M	C	M	D	B	D	M	A	M	B	B	B	B	C	E	C	M
US 123																						
real	D	B	C	B	C	B	C	M	D	B	B	M	A	B	D	M	B	B	C	B	C	B
survey	D	B	C	B	C	B	C	M	D	B	B	B	A	B	D	B	B	B	C	B	C	B
US 124A																						
real	D	B	C	B	B	B	C	M	D	B	D	M	A	B	A	M	B	B	C	B	C	B
survey	D	B	C	B	B	B	C	M	D	B	C	B	A	B	A	B	B	B	C	B	C	B
US 124B																						
real	D	B	C	B	B	B	C	M	D	B	D	M	A	B	A	M	B	B	C	B	C	B
survey	D	B	C	B	B	B	C	M	D	B	D	B	A	B	A	B	B	B	C	B	C	B
US 130A																						
real	D	E	C	M	C	E	C	M	D	E	D	E	A	E	B	E	B	E	C	E	D	E
survey	D	E	C	M	D	E	C	M	D	E	D	E	A	E	D	E	B	E	C	E	D	E
US 130B																						
real	D	B	C	B	C	M	C	M	D	M	D	M	A	M	B	B	B	M	C	E	D	M
survey	D	B	C	B	C	M	C	M	D	M	D	M	A	M	D	B	B	M	C	E	D	M

Tab. 4.3.9: Results of the II level GNDT form

US	Case	I3	I GNDT	I GNDT Norm
88	real	0,560847	262,5	0,686275
	survey	0,560847	266,25	0,696078
123	real	0,560847	236,25	0,617647
	survey	0,560847	236,25	0,617647
124A	real	0,560847	217,5	0,568627
	survey	0,560847	210	0,54902
124B	real	0,560847	217,5	0,568627
	survey	0,560847	217,5	0,568627
130A	real	0,613757	258,75	0,676470578
	survey	0,613757	258,75	0,676471
130B	real	0,613757	288,75	0,754901946
	survey	0,613757	266,25	0,696078

Tab. 4.3.10: I3, I GNDT and I GNDT norm indices

4.3.4 Building vulnerability

The vulnerability assessment has been defined as shown in Tab. 4.3.11. The qualitative scale used by Vulnus to define the vulnerability evaluation is the same as chapter 4.2.4. For structural units 123, 124B, 130A and 130B the evaluation is the same for both cases, while for US 88 and 124A it changes. The US 88 has a “small” vulnerability in the real case and a “medium” vulnerability in the survey case. The survey case is therefore in favor of safety. The US 124A has a “medium” vulnerability in the real case and a “very small” vulnerability in the survey case. This change reflects the situation already observed in chapter 4.3.2 about the same structural unit, in which the index I2 is sensibly increased in the survey case.

US	Case	a/g = 0.02
88	real	SMALL
	survey	MEDIUM

123	real	MEDIUM
	survey	MEDIUM
124A	real	MEDIUM
	survey	VERY SMALL
124B	real	SMALL
	survey	SMALL
130A	real	MEDIUM
	survey	MEDIUM
130B	real	MEDIUM
	survey	MEDIUM

Tab. 4.3.11: Vulnerability analysis for each building

4.3.5 Group vulnerability

Tables below show the probability that buildings have to belong to a certain value of the vulnerability evaluation scale, referring to buildings or to volumes (Tab. 4.3.12, Tab. 4.3.13, Tab.4.3.14, Tab.4.3.15). The vulnerability level referred to volumes is the same to the one referred to buildings for all the cases. Only for US 130 the program provides the same "medium" assessment for both cases, while for all the others units it changes. US 88 has a "very small" assessment for the real case and a "small" one for the survey case while for US 123 and 124 the judgments are the same but opposite.

US 88													
Vulnerability identify level referred to buildings													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	10 0	
0,02	real	1	0	0	0	0	0	0	0	0	0	0	VERY SMALL
	survey	0,899	1	0,899	0,8	0,1	0	0	0	0	0	0	SMALL
Vulnerability identify level referred to volumes													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	1	0	0	0	0	0	0	0	0	0	0	VERY SMALL

	survey	0,899	1	0,899	0,8	0,1	0	0	0	0	0	0	SMALL

Tab. 4.2.12: Group vulnerability for US 88

US 123													
Vulnerability identify level referred to buildings													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	1	0,899	0,5	0,1	0	0	0	0	0	0	0	SMALL
	survey	1	0,1	0	0	0	0	0	0	0	0	0	VERY SMALL
Vulnerability identify level referred to volumes													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	1	0,899	0,5	0,1	0	0	0	0	0	0	0	SMALL
	survey	1	0,1	0	0	0	0	0	0	0	0	0	VERY SMALL

Tab. 4.2.13: Group vulnerability for US 123

US 124													
Vulnerability identify level referred to buildings													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	1	0,899	0,5	0,1	0	0	0	0	0	0	0	SMALL
	survey	1	0,1	0	0	0	0	0	0	0	0	0	VERY SMALL
Vulnerability identify level referred to volumes													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	1	0,899	0,5	0,1	0	0	0	0	0	0	0	SMALL
	survey	1	0,1	0	0	0	0	0	0	0	0	0	VERY SMALL

Tab. 4.2.14: Group vulnerability for US 124

US 130													
Vulnerability identify level referred to buildings													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	0	0	0,11	0,55	0,88	1	0,88	0	0	0	0	MEDIUM
	survey	0,11	0,55	0,88	1	1	0,88	0,22	0	0	0	0	MEDIUM
Vulnerability identify level referred to volumes													
a/g	Case	Vg											Class
		0	10	20	30	40	50	60	70	80	90	100	
0,02	real	0	0	0,11	0,55	0,88	1	0,88	0	0	0	0	MEDIUM
	survey	0,11	0,55	0,88	1	1	0,88	0,22	0	0	0	0	MEDIUM

Tab. 4.2.15: Group vulnerability for US 130

4.3.6 Expected damage frequencies

The expected values of serious damage E[Vg] are defined in function of the different values of PGA/g (Tab. 4.3.16, Tab. 4.3.17, Tab. 4.3.18) and Vulnus represents this relation through fragility curves graphs (Fig. 4.3.6, Fig. 4.3.7, Tab. 4.3.8). Due to the proximity of US 123 and 124, these two units have been evaluated together and they belong to the same fragility curve.

For a PGA/g value of 0.2, the average expectation of damage for US 88 is 19%, with a range between 7% and 26%, in the real case and 46%, with a range between 7% and 78%, in the survey case. The range defined in the survey case is higher of the one defined in the real case for the PGA/g value of 0.2 due to the greater uncertainty of the information, and consequently to the lower info quality, in the GNDT for of the survey case.

US 88		PGA/g					
		0	0,1	0,2	0,3	0,4	0,5
real	E[Vg] Low	0	0	0,072	0,136	0,572	0,99
	E[Vg] White	0,02	0,02	0,193	0,486	0,796	1
	E[Vg] Up	0,049	0,049	0,263	0,789	0,986	1
survey	E[Vg] Low	0	0	0,072	0,499	0,99	0,99
	E[Vg] White	0,02	0,02	0,457	0,778	1	1
	E[Vg] Up	0,049	0,049	0,784	1	1	1

Tab. 4.3.16: Expectation values of damage for block US 88

For a PGA/g value of 0.2, the average expectation of damage for US 123 and 124 is 31%, with a range between 4% and 53%, in the real case and 19%, with a range between 3% and 27%, in the survey case.

US 123 US 124		PGA/g				
		0	0,2	0,4	0,6	0,8
real	E[Vg] Low	0	0,036	0,545	0,899	0,99
	E[Vg] White	0,02	0,315	0,781	0,963	1
	E[Vg] Up	0,049	0,532	0,973	1	1
survey	E[Vg] Low	0	0,027	0,254	0,672	0,99
	E[Vg] White	0,02	0,187	0,599	0,853	1
	E[Vg] Up	0,049	0,275	0,893	1	1

Tab. 4.3.17: Expectation values of damage for block US 123 and 124

For a PGA/g value of 0.2, the average expectation of damage for US 130 is 14%, with a range between 7% and 15%, in the real case and 18%, with a range between 5% and 25%, in the survey case. As seen in US 88, the range defined in the survey case is higher of the one defined in the real case for the same PGA/g value of 0.2 due to the lower info quality in the GNDT for of the survey case.

US 130		PGA/g					
		0	0,1	0,2	0,3	0,4	0,5
real	E[Vg] Low	0	0	0,072	0,109	0,481	0,945
	E[Vg] White	0,02	0,02	0,136	0,403	0,741	0,984
	E[Vg] Up	0,049	0,049	0,149	0,644	0,944	1
survey	E[Vg] Low	0	0	0,054	0,054	0,236	0,463
	E[Vg] White	0,02	0,02	0,179	0,363	0,599	0,757
	E[Vg] Up	0,049	0,049	0,247	0,616	0,901	0,986

Tab. 4.3.18: Expectation values of damage for block US 130

In the fragility curve graph (Fig. 4.3.13) for the **real case** of US 88 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: $0.01 \leq PGA/g \leq 0.14$
  E[Vg] White: $0.01 \leq PGA/g \leq 0.14$
  E[Vg] Up: $0.01 \leq PGA/g \leq 0.14$
- Second phase: the central curve increases with a constant and regular slope, while the upper limit grows with greater inclination compared to the central one. The lower limit has a first phase of constant 0,22 value and then it increases its inclination, with a slope similar to the upper curve.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: $PGA/g \geq 0.49$
  E[Vg] White: $PGA/g \geq 0.49$
  E[Vg] Up: $PGA/g \geq 0.42$

Therefore for the value of PGA/g of 0.2 the vulnerability is low-medium.

In the fragility curve graph (Fig. 4.3.14) for the **survey case** of US 88 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: 0.01 ≤ PGA/g ≤ 0.09
  E[Vg] White: 0.01 ≤ PGA/g ≤ 0.09
  E[Vg] Up: 0.01 ≤ PGA/g ≤ 0.09
- Second phase: the central curve, after a first phase of higher inclination, increases with a constant and regular slope, while the upper limit grows with a greater inclination compared to the central one. The lower limit has a first phase of constant 0,22 value and then it increases its inclination.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: PGA/g ≥ 0.41
  E[Vg] White: PGA/g ≥ 0.40
  E[Vg] Up: PGA/g ≥ 0.28

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.
The fragility curve of the survey case is shifted to the left, corresponding to lower values of PGA/g respect to the real case. Plus, the range between the lower and the upper limits is wider in the survey case.

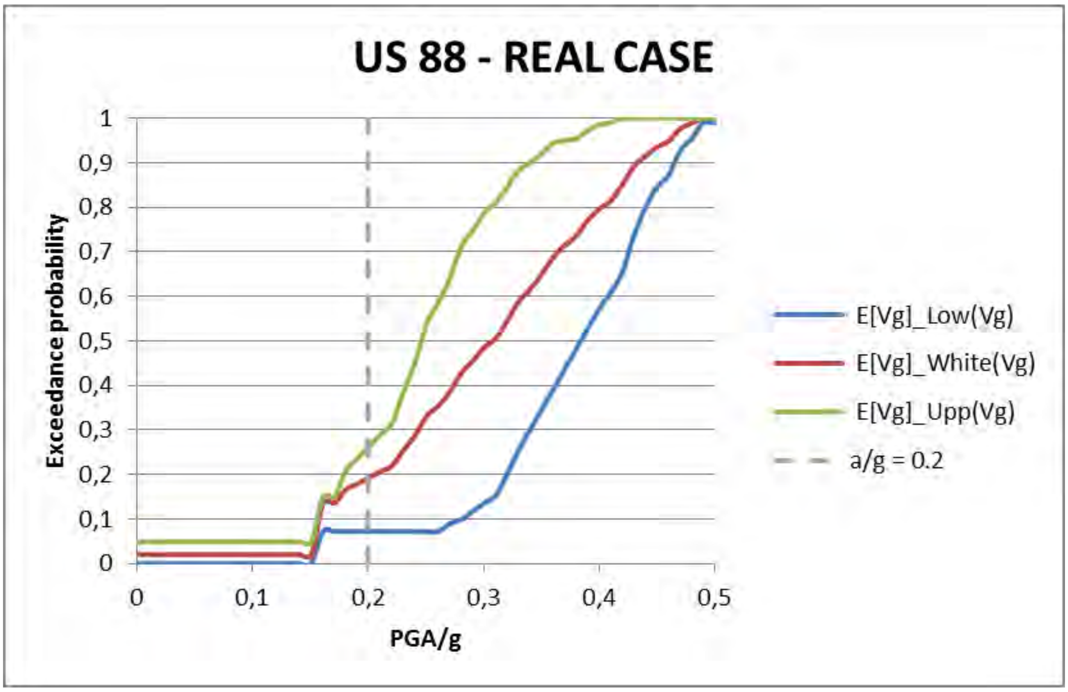


Fig. 4.3.13: Fragility curves for US 88- Real case

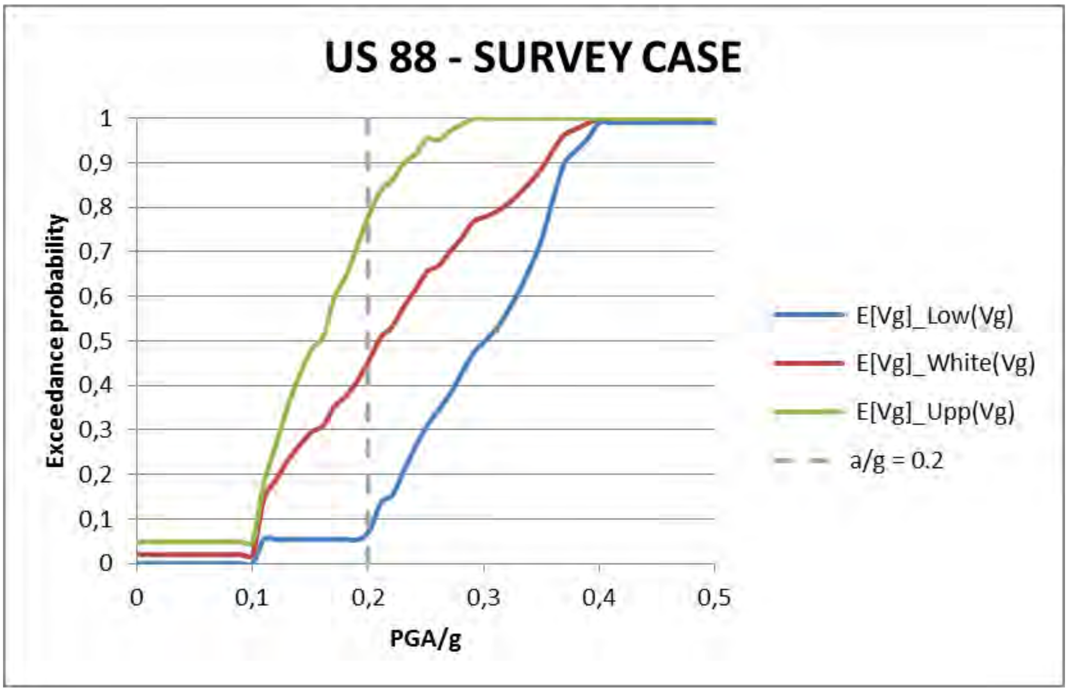


Fig. 4.3.14: Fragility curves for US 88- Survey case

In the fragility curve graph (Fig. 4.3.15) for the **real case** of US 123 and 124 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:

E[Vg] Low: 0.01 ≤ PGA/g ≤ 0.11

E[Vg] White: 0.01 ≤ PGA/g ≤ 0.11

E[Vg] Up: 0.01 ≤ PGA/g ≤ 0.11

- Second phase: the lower and the upper limits increase with similar and irregular slopes, while the central limit grows with lower inclination compared to the other two curves.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:

  E[Vg] Low: PGA/g ≥ 0.65

  E[Vg] White: PGA/g ≥ 0.63

  E[Vg] Up: PGA/g ≥ 0.47

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.

In the fragility curve graph (Fig. 4.3.16) for the **survey case** of US 123 and 124 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:

  E[Vg] Low: 0.01 ≤ PGA/g ≤ 0.13

  E[Vg] White: 0.01 ≤ PGA/g ≤ 0.13

  E[Vg] Up: 0.01 ≤ PGA/g ≤ 0.13
- Second phase: the central curve increases with a constant and regular slope, while the upper limit grows with greater inclination compared to the central one. The lower limit has a first phase low slope, until 0.21, and then it increases its inclination.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:

  E[Vg] Low: PGA/g ≥ 0.80

  E[Vg] White: PGA/g ≥ 0.79

  E[Vg] Up: PGA/g ≥ 0.60

Therefore for the value of PGA/g of 0.2 the vulnerability is low-medium.

The fragility curve of the survey case is shifted to the right, corresponding to higher values of PGA/g respect to the real case. Plus, the range between the lower and the upper limits is wider in the survey case, due to the approximations made in this case. The range of the curve is in fact related with the info quality of the GNDT form: if the quality is low, the range is wider, if the quality is elevated, the range if thinner.

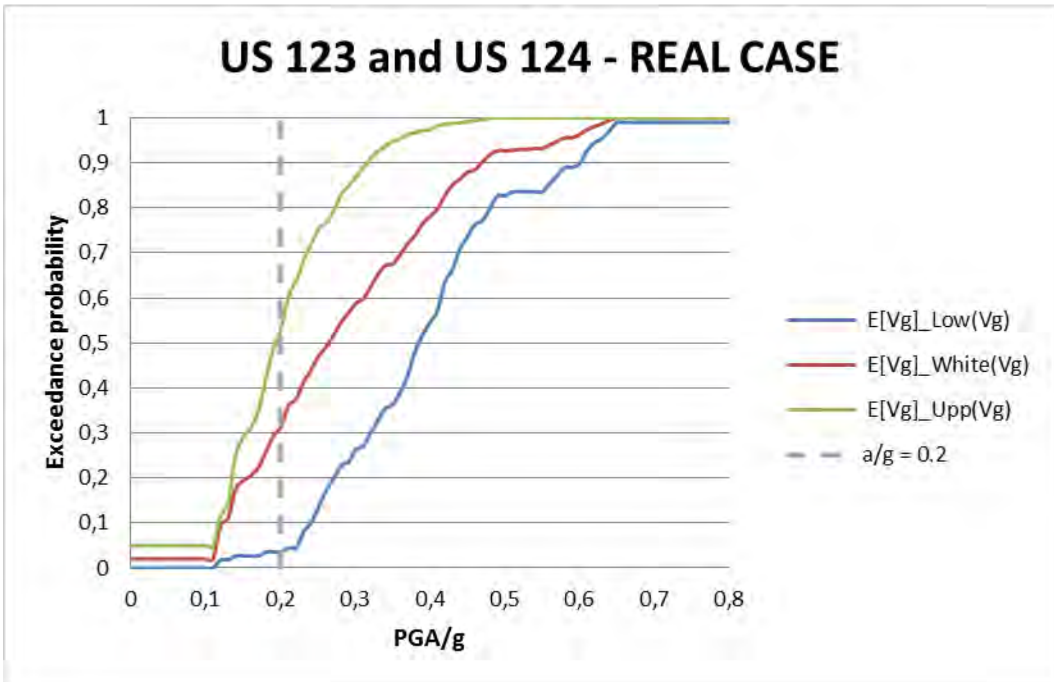


Fig. 4.3.15: Fragility curves for US 123 and 124- Real case

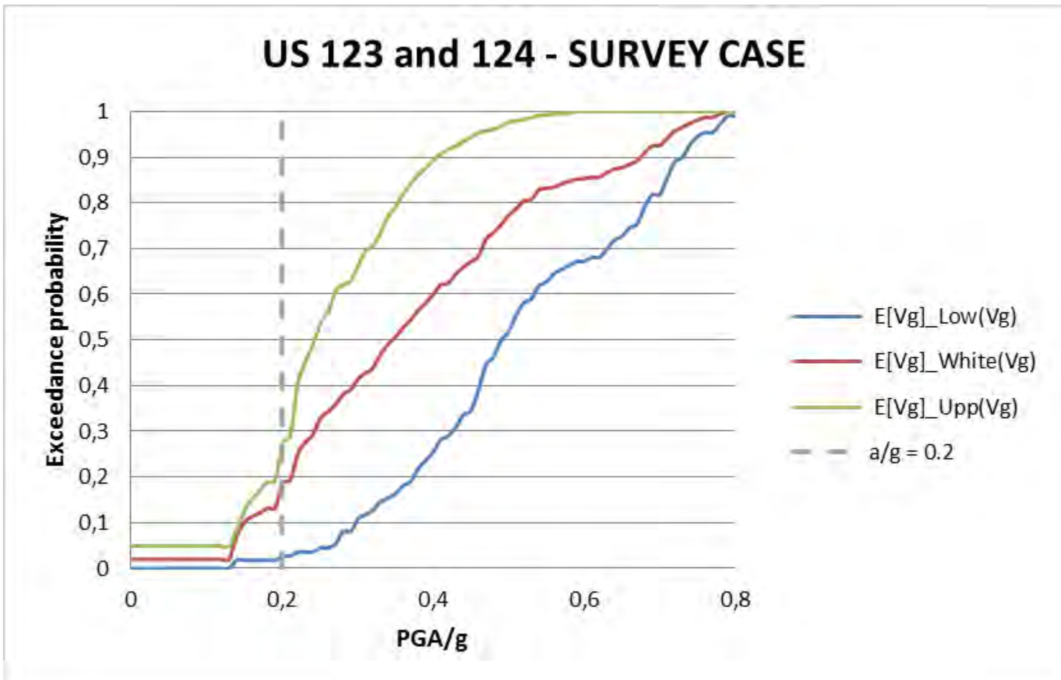


Fig. 4.3.16: Fragility curves for US 123 and 124- Survey case

In the fragility curve graph (Fig. 4.3.17) for the **real case** of US 130 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: $0.01 \leq PGA/g \leq 0.06$
  E[Vg] White: $0.01 \leq PGA/g \leq 0.06$
  E[Vg] Up: $0.01 \leq PGA/g \leq 0.06$
- Second phase: the central curve increases with a constant and regular slope, while the upper limit grows with greater inclination compared to the central one. The lower limit has a first phase low slope, until 0.13, and then it increases its inclination.
- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: $PGA/g \geq 0.28$
  E[Vg] White: $PGA/g \geq 0.26$
  E[Vg] Up: $PGA/g \geq 0.24$

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.

In the fragility curve graph (Fig. 4.3.18) for the **survey case** of US 130 three phases can be identified:

- First phase: there are constant values of low vulnerability in correspondence to low structural damage and in particular for each curve in the range:
  E[Vg] Low: $0.01 \leq PGA/g \leq 0.07$
  E[Vg] White: $0.01 \leq PGA/g \leq 0.07$
  E[Vg] Up: $0.01 \leq PGA/g \leq 0.07$
- Second phase: the central curve increases with a constant and regular slope, while the upper limit grows with greater inclination compared to the central one. The lower limit has a first phase low slope, until 0.15, and then it increases its inclination.

- Third phase: the three curves reach the maximum value of severe structural damage and consequent collapse of the structure after the following values:
  E[Vg] Low: PGA/g ≥ 0.37
  E[Vg] White: PGA/g ≥ 0.36
  E[Vg] Up: PGA/g ≥ 0.27

Therefore for the value of PGA/g of 0.2 the vulnerability is medium.

The fragility curve of the survey case is shifted to the right, corresponding to higher values of PGA/g respect to the real case. Plus, the range between the lower and the upper limits is sensibly wider in the survey case.

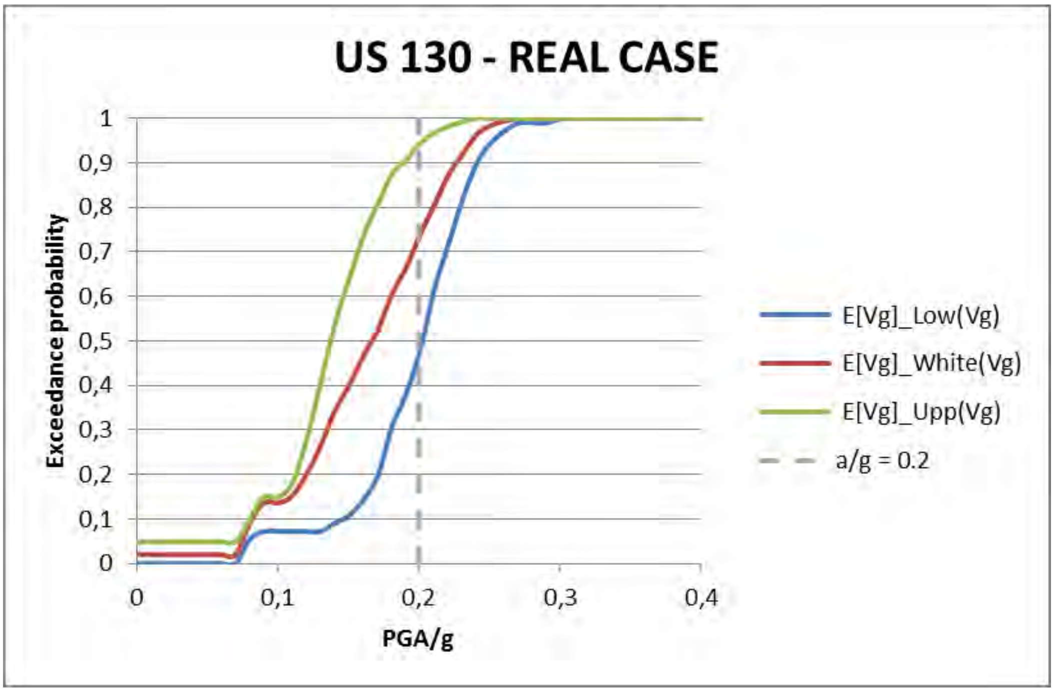


Fig. 4.3.17: Fragility curves for US 130- Real case

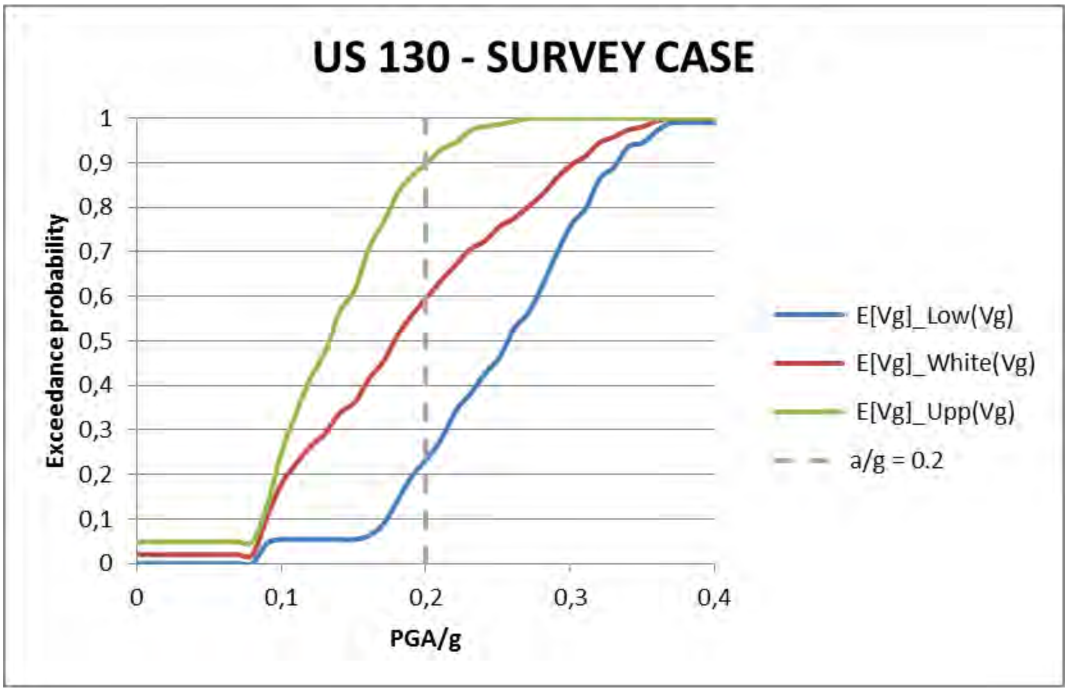


Fig. 4.3.18: Fragility curves for US 130- Survey case

The results obtained for US 88 show that the survey case is evaluated in general as more vulnerable than the real case: for the same value of a_g=0.2, the fragility curves define an higher probability of exceedance in the survey case respect the real case, the software gives a vulnerability evaluation of “small” in the real case and “medium” in the survey case and the I2 index, related with the out of plane mechanisms, is lower in the survey case. The assumptions and the simplifications made in the survey analysis are therefore in favor of the safer conduction.

The results of units 123 and 124 show how the peculiarity of the building characteristics can influence the analysis. In fact the fragility curve indicates, for the same value of a/g=0.2, lower values of the exceedance probability in the survey case, even though the range between the low and the upper curve is wider. The survey case represents then a situation that is less vulnerable than the real case, underestimating the risk. This is evident also in the comparison of the I2 index, which is higher in the survey case for both units, and the vulnerability judgement for US 124A, which is “medium” for the real case and “very small” for the survey case. The analysis of these units underline the fact that with a rapid survey is not possible to deeply evaluate each aspect of the buildings and the possible presence of adverse structure characteristics which increase the seismic vulnerability must be considered.

Also in the results of US 130 it is possible to note that the exceedance probability, correspondent to the same value of a/g=0.2, is lower in the survey case and the values of I1 and I2 are higher in the survey case than in the real case. Despite this, the vulnerability judgment of the software is "medium" for both cases. In this unit the approximations and the simplifications made in the survey case underestimate the vulnerability of the building, but the difference are not so incisive to change the global vulnerability assessment.

The comparison made between the real cases and the survey cases for these four structural units shows that in general the assumptions made for the survey analysis, and consequently the typological analysis, are valid. In fact the results overestimates the seismic vulnerability, in favor of security, in US 88 and they underestimates it in US 123, 124 and 130, but with small difference that does not change the global assessment. The only exception is made in case of peculiar building characteristics that are not evaluable with a rapid survey. More precisely, the assumptions made are valid as a part of a rapid and typological analysis that considers the main building characteristics.

5 LOCAL MECHANISMS OF COLLAPSE

For the vulnerability assessment of existing masonry buildings, two type of mechanisms of collapse are distinguished: local mechanisms of collapse and global mechanisms of collapse. The constructions safety must be evaluated considering both these type of mechanisms.[1]

Local mechanisms interest both single walls panels or entire parts of the construction and they are favored by the absence or the ineffectiveness of connections between walls and horizontal structures and/or between perpendicular walls. Global mechanisms interest the entire construction and they usually involve walls panel on their plan. In case of clustered buildings which are adjacent, in contact or interconnected with adjoining buildings, global methods are not appropriate. During the analysis of a clustered building, the possible interaction with adjacent buildings must be considered and the structural unit (US) on which the study is performed must be identified. The structural unit is usually delimitated by open spaces, structural joints or adjacent buildings with a different structural typology. In the same structural unit the flow of vertical loads must have continuity from sky to earth.[2]

Seismic events often cause the partial collapse of masonry buildings and in general it happens for the equilibrium loss of walls portions. The study of this mechanisms is possible only if the analyzed wall shows a monolithic behavior that prevents the disintegration of the masonry. The verification of local mechanisms of collapse, both in-plane and out-of-plane, can be performed using the limit equilibrium analysis, according to the kinematic approach. The identification of the collapse mechanism and the assessment of the horizontal action are the basis of this approach.[3]

The analyzed local mechanisms are: the simple overturning mechanism, the vertical bending mechanism and the in-plane mechanism. The analysis has been performed considering the typological assumptions made in chapter 2. The

[1] *§C8. Costruzioni esistenti* (Circolare esplicativa NTC 2008, p. 295)

[2] *§8.8.1 Costruzioni in muratura* (NTC 2008, pp. 331-332)

[3] *§C8A.4. Analisi dei meccanismi locali di collasso in edifici esistenti in muratura* (Circolare esplicativa NTC 2008, p. 409)

analysis aims to define the fragility curve for each building typology (see chapter 6). The study of the local mechanisms leads to define the capacity curves (chapters 5.4.4, 5.5.4, 5.6.4) for each analyzed typology and referring to each type of mechanism. The capacity curve is the basis to determinate the fragility curve and consequently to give a vulnerability assessment of the building typology, referring to the analyzed mechanism; once given the assessment to each typology it is possible to extend the vulnerability evaluation to all the historical center.

5.1 MECHANISMS OF DAMAGE

The building typological and structural characteristics influence the behavior that the construction has in response of the seismic action. The building response can be divided in two categories of mechanisms of damage: the first mode and the second mode mechanisms.

The first mode mechanisms correspond to out of plane mechanisms and they affect the walls that are perpendicular to the main direction of the seismic event. This mechanisms is the most common case in ordinary buildings and brings to the activation of simple overturning, composed overturning and vertical flection mechanisms.

The second mode mechanisms correspond to in plane mechanisms and they concern the walls that are parallel to the main direction of the seismic event. The damage caused by this type of mechanisms is typically due to shear and bending stresses and it usually occurs for higher values of the multiplier of the seismic masses than the ones obtained for out-of-plane mechanisms.

In the following chapters the analyzed mechanisms of collapse are explained in detail.

5.2 REGULATORY APPROACH TO THE ANALYS OF LOCAL MECHANISMS OF COLLAPSE

The kinematic approach[4] allow to define the trend of the horizontal action that the structure is progressively able to sustain during the mechanisms evolution. The curve is expressed through the multiplier α, that determinate the connection between the applied horizontal forces and the corresponding weight of the masses, represented in function of the displacement d_k of a reference point of the system. The curve must be defined until the annulment of the sustain horizontal actions ability (α=0). This curve can be transformed into the capacity curve of a one freedom degree equivalent system in which the ultimate displacement capacity of the local mechanism can be defined: the obtained value must be compared to the seismic displacement demand required by the seismic action.

For each significant local mechanisms of collapse, the kinematic method is developed following these steps:

- Transformation of a part of the construction in a labile system (kinematic chain), through the individuation of rigid bodies, which are defined by fracture planes hypothesized considering the low tensile strength of the masonry. Rigid bodies can rotate or slide between them (mechanisms of damage and collapse);
- Evaluation of the horizontal loads multiplier α_0, which implies the mechanism activation (limit state of damage)
- Evolution assessment of the horizontal load multiplier α_0 considering the increasing displacement d_k of a control point of the kinematic chain, usually defined in proximity of the masses gravity center, until the annulation of the horizontal seismic force;
- Transformation of the resulted curve in a capacity curve, ie in spectral acceleration (a*) and spectral displacement (d*), with the evaluation of the ultimate limit displacement obtained in case of collapse (ultimate limit state);
- Safety analysis, through the compatibility control of displacements and/or required resistance for the structure.

[4] *§C8A.4. Analisi dei meccanismi locali di collasso in edifici esistenti in muratura* (Circolare esplicativa NTC 2008, pp. 409-410)

For the application of the method the assumptions in general are:

- Null masonry tensile strength;
- Absence of sliding between the blocks;
- Unlimited masonry compressive strength

To have a more realistic simulation of the behavior it is appropriate to consider:

- Sliding between blocks, considering the friction;
- Connection between the masonry walls, even with limited resistance;
- The presence of tie rods;
- The limited compressive resistance of masonry, considering the hinge rearward from the section edge;
- The presence of walls with disconnected leafs.

5.2.1 Linear kinematic analysis[5]

In order to obtain the horizontal loads multiplier α_0 which induces the activation of the local mechanism of damage, the following forces must be applied to the rigid blocks that constitutes the kinematic chain: the weight force of the blocks, applied in their barycenter; the vertical loads that insist on the block; a system of horizontal loads proportional to the vertical loads; eventual external forces (like forces transmitted by tie rods) and eventual internal forces.

A virtual rotation θ_k is assigned to the generic block k and it is possible to define the applied forces displacements, as a function of θ_k and the building geometry. The multiplier α_0 is obtained from the application of the Principle of Virtual Work, in terms of displacements, equalizing the total work performed by external and internal forces applied to the system during the virtual motion act:

$$\alpha_0 \cdot \left(\sum_{i=1}^{n} Pi \cdot \delta_{x,i} + \sum_{j=n+1}^{n+m} Pj \cdot \delta_{x,j} \right) - \sum_{i=1}^{n} P_i \cdot \delta_{y,i} - \sum_{h=1}^{0} F_h \cdot \delta_h = L_{fi} \qquad (5.1)$$

where:

n is the number of the weight forces applied to the various blocks of the kinematic chain;

[5] *§C8A.4.1 Analisi cinematica lineare* (Circolare esplicativa NTC 2008, pp. 410-411)

m is the number of weight forces not directly imposed on the blocks, whose masses, under seismic action effect, generate horizontal forces on the elements of the kinematic chain;

o is the number of external forces, not associated with the masses, applied to different bocks;

P_i is the generic weight force applied (weight of the block, applies in its center of gravity);

P_j is the generic weight force, not directly applied on the blocks, whose mass, under seismic action effects, generates a horizontal force on the elements of the kinematic chain;

$\delta_{x,i}$ is the virtual horizontal displacement of the application point of the i-th weight P_i. The verse associated with the direction of the seismic action that activates the mechanisms is assumed positive;

$\delta_{x,j}$ is the virtual horizontal displacement of the application point of the j-th weight P_j. It is assumed positive the verse associated with the direction of the seismic action that activates the mechanisms;

$\delta_{y,i}$ is the virtual vertical displacement of the application point of the i-th weight P_i. It is assumed positive upward;

F_h is the generic external force (in absolute value) applied to a block;

δ_h is the virtual displacement of the application point of the h-th external force in the same direction, positive if with conflicting verse;

L_{fi} is the work of eventual internal forces.

5.2.2 Non-linear kinematic analysis[6]

In order to define the displacement capacity of the structure until the collapse for each considered mechanism, the horizontal loads multiplier α can be evaluated not only on the starting configuration, but also on the varied configuration of the kinematic chain, representative of the mechanism evolution and described by the displacement d_k of a control point of the system. The analysis must be carried out until the achievement of the configuration which correspond to the annulment of the multiplier α, corresponding to the $d_{k,0}$ displacement.

[6] *§C8A.4.2 Analisi cinematica non lineare* (Circolare esplicativa NTC 2008, pp. 412-414)

In correspondence with each kinematic configuration of rigid blocks, the α multiplier value can ben estimated using the equation (5.1), referring to the modified configuration. The analysis can be carried out graphically, identifying the system geometry in different configurations until the collapse, or by analytical-numerical analysis, considering a sequence of virtual rotations and modifying progressively the system geometry.

If the various actions (weight loads, external or internal forces) are constant during the kinematism evolution, the obtained curve is almost linear. In this condition, only the $d_{k,0}$ displacement must be evaluated, which correspond to the annulment of the multiplier and the curve assumes the following expression:

$$\alpha = \alpha_0 \cdot \left(\frac{1 - d_k}{d_{k0}} \right) \tag{5.2}$$

This configuration can be obtained by expressing the geometry in a generic modified configuration, function of the finite rotation θ_{k0}, applying the Principle of Virtual Work using the equation (5.1), assuming α=0 and obtaining θ_{k0} through the equation, usually non-linear.

Starting from the trend of the horizontal loads multiplier α in function of the d_k displacement of the structure control point, the capacity curve of the equivalent oscillator must be defined, identifying the relation between the a^* acceleration and the d^* displacement.

The participant mass of the kinematism M^* can be evaluated considering the virtual displacement of the application point of the different loads, associated with the kinematism, as a modal form of vibration:

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \cdot \delta_{x,i} \right)^2}{g \cdot \sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}^2} \tag{5.3}$$

where:

n+m is the number of the weight forces P_i applied whose masses, due to the seismic action, generate horizontal forces on the elements of the kinematic chain;

$\delta_{x,i}$ is the horizontal virtual displacement of the application point of the i-th weight load P_i;

The seismic spectral acceleration a^* is obtained multiplying the gravity acceleration and the multiplier α, divided by the fraction of participant mass of the kinematism. The spectral acceleration for the activation of the mechanism is:

$$a_0^* = \alpha_0 \cdot \frac{\sum_{i=1}^{n+m} P_i}{M^* \cdot FC} = \frac{\alpha_0 \cdot g}{e^* \cdot FC} \tag{5.4}$$

where:

g is the gravity acceleration;

e* is the fraction of participant mass of the structure and it is calculated with:

$$e^* = \frac{g \cdot M^*}{\sum_{i=1}^{n+m} P_i}; \tag{5.5}$$

FC is the confidence factor. In the case in which the compressive strength of the masonry has not be taken into account during the evaluation of the multiplier α, the confidence factor is the one of the knowledge level LC1.

The spectral displacement d^* of the equivalent oscillator can be considered as the average displacement of the different points in which the weight P_i are applied. It is possible to define the equivalent spectral displacement from the d_k displacement of the control point, considering the virtual displacements evaluated from the starting configuration:

$$d^* = d_k \cdot \frac{\sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}^2}{\delta_{x,k} \cdot \sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}} \tag{5.6}$$

where:

n, m, P_i, $\delta_{x,i}$ are already been defined above;

$\delta_{x,k}$ is the virtual horizontal displacement of the k point, assumed as a reference for the determination of the displacement d_k;

If the different actions are constant, the curve has a linear trend and the capacity curve assumes the following expression:

$$a^* = a_0^* \cdot \left(\frac{1 - d^*}{d_0^*} \right) \tag{5.7}$$

where:

d_0* is the equivalent spectral displacement corresponding to $d_{k,0}$.

When the external forces have different entity, the curve is assumed at linear intervals. The strength and the displacement at the limit state of damage (LSD) and at the ultimate limit state (ULS) are evaluated considering the capacity curve in correspondence of the spectral displacement d_u* for the ultimate limit state. The spectral displacement d_u* is defined as the 40% of the displacement that annul the spectral acceleration *a**.

5.2.3 Safety analysis at the ultimate limit state[7]

The verification at the ultimate limit state of local mechanisms can be applied using the following criteria.

<u>Linear kinematic analysis</u>

If the analysis is referred to an isolated element or a portion of the construction from the top to the ground, the ultimate limit state analysis is satisfied if the spectral acceleration a_0*, which activates the mechanism, satisfies the inequality:

$$a_0^* \geq \frac{a_g(P_{VR}) \cdot S}{q} \tag{5.8}$$

where:

a_g is the function of the probability of exceedance the chosen limit state and the reference life as defined in chapter 3.2 of NTC 2008 code;

S is the soil factor and its value is defined in chapter 3.2.2;

q is the behavior factor, that can be considered equal to 2.

If the local mechanism considers the upper part of the building, the absolute acceleration of the portion is amplified compared to the ground. An acceptable approximation consists in verifying both the inequality (5.8) and the following one:

$$a_0^* \geq \frac{S_e(T_1) \cdot \psi_{(z)} \cdot \gamma}{q} \tag{5.9}$$

[7] *§C8A.4.2.3 Verifiche di sicurezza* (Circolare esplicativa NTC 2008, pp. 415-417)

where:

$S_e(T_1)$ is the elastic response spectrum as defined in chapter 3.2.2, in function of the exceedance probability of the analyzed limit state (in this case 10%) and of the reference period V_R calculated for the period T_1;

T_1 is the first vibration period of the entire structure in the considered direction;

$\psi_{(z)}$ is the first vibration mode in the considered direction, standardized to 1 at the top of the building. It is assumed:

$$\psi_{(z)} = Z/H \tag{5.10}$$

where H is the height above the foundation;

Z is the height above the foundations of the barycenter of the constraint line between the blocks affected by the mechanism and the rest of the structure.

<u>Non-linear kinematic analysis</u>

The safety analysis for the ultimate limit state for local mechanisms considers the comparison between the ultimate displacement capacity d_u* of the local mechanism and the displacement demand obtained from the displacements spectrum in correspondence of the secant period T_s of the response spectrum.

The displacement is defined as:

$$d_s = 0.4\, d_u^* \tag{5.11}$$

and the acceleration a_s* relative to the displacement d_s* is identified on the capacity curve. The secant period of the response spectrum T_s is therefore calculated as:

$$T_s = 2\pi\sqrt{\frac{d_s^*}{a_s^*}} \tag{5.12}$$

The displacement demand $\Delta_d(T_s)$ is so obtained:

- If the analysis is referred to an isolated element or a portion of the building from the ground to the top, the safety verification for the ultimate limit state is satisfied if:

  $$d_u^* \geq S_{De}\,(T_s) \tag{5.13}$$

  where S_{De} is the elastic design response spectrum defined in §3.2.3.2.2 of the NTC code;

- If the local mechanism is referred to the upper part of the building, the ultimate limit state analysis is satisfied if:

$$d_u^* \geq S_{De}(T_1) \cdot \psi_{(z)} \cdot \gamma \cdot \frac{\left(\frac{T_s}{T_1}\right)^2}{\sqrt{\left(1-\frac{T_s}{T_1}\right)^2 + 0.02\frac{T_s}{T_1}}} \tag{5.14}$$

5.3 ANALYSIS OF THE MECHANISMS[8]

First, the regulatory approach to the calculation of kinematic mechanisms is explained and then the analyzed mechanisms are illustrated in detail.

The symbols used in the following paragraphs are:

α is the horizontal load multiplier;

n is the stories number which the mechanism affects;

W_i is the wall weight at the i-th level or of the i-th macroelement;

W_{0i} is the weight of the portion of the separated wedge at the i-th floor in shear walls (including eventual loads transmitted by arches or vaults);

F_{Vi} is the vertical component of arches or vaults thrust on the i-th level wall;

F_{Hi} is the horizontal component of arches or vaults thrust on the i-th level wall;

P_{Si} is the weight of the floor acting on the i-th level;

P_{S0i} is the weight of the floor acting on the wedge portion on shear walls at the i-th level;

P_{Vij} is the i-th vertical load transmitted on top of the j-th macroelement;

P is the transmitted load from the ridge beam or from the rafter of the hip-roof. For the in-plane mechanism is the weight of the triangular portion of the walls;

N is the generic vertical load acting on top of the macroelement. For the in plane mechanism it is the transmitted load of the last floor and the roof;

H is the maximum value of the reaction stainable by the shear wall or by the tie rod to the thrust of the horizontal arch effect. For the in-plane mechanism it is the height of the wall portion affected by the mechanism;

P_H is the static thrust acting on top of the macroelement due to the roof;

P_{Hij} is the i-th component of the static thrust acting on top of the j-ht body due to the roof;

T_i is the action of the eventual tie rods located on top of the wall at the i-th level;

s_i is the wall thickness at the i-th level;

h_i is the vertical arm of the action due to the floor or tie rod acting on thte wall at the i-th level or it is the height of the i-th macroelement;

[8] *Schede illustrative dei principali meccanismi di collasso locali negli edifici esistenti in muratura e dei relative modelli cinematici di analisi*. (Milano et al., 2009)

h_{Pi} is the vertical arm of the action due to the floor acting on the wall at the i-th level;

L_i is the length of the i-th macroelement;

x_{Gi} is the horizontal arm of the weight of the i-th element weight;

y_{Gi} is the vertical arm of the weight of the i-th element weight;

x_{G0i} is the horizontal arm of the wedge weight at the i-th level on shear walls;

y_{G0i} is the vertical arm of the wedge weight at the i-th level on shear walls;

d is the horizontal arm of the vertical load acting on top of the macroelement;

d_i is the horizontal arm of the load due to the floor acting on the wall at the i-th level;

d_{ij} is the horizontal arm of the i-th vertical load transmitted at the top of the j-th floor

d_{0i} is the horizontal arm of the load transmitted by the slab to the wedge on shear walls;

a_i is the horizontal arm of the load transmitted by the slab on the wall at the i-th level;

h_{Vi} is the vertical arm of the arches and vaults thrust at the i-th level;

d_{Vi} is the horizontal arm of the arches and vaults actions at the i-th level.

The arms of the different forces that act on the microelements are referred to the hinges respect to which the rotation occurs.
The arm of force N from the rotation pole during the in-plane analysis is assumed, for safety purpose, as 0.75L.

5.3.1 Simple overturning mechanism[9]

The mechanism is activated from the rigid rotation of whole facades or walls portions. The rotation occurs usually around horizontal axes at the base of the rotation element and these axes run through the masonry structure subjected to out of plane actions. (Fig.5.3.1)
The mechanism interests the monolithic external walls of the building that are perpendicular to the seismic action. It is influenced by the geometries and the

[9] *Schede illustrative dei principali meccanismi di collasso locali negli edifici esistenti in muratura e dei relative modelli cinematici di analisi*. (Milano et al., 2009, p. 4)

dimension of the analyzed wall and by the connection quality between horizontal structures and walls at the various levels of the structure. It can involve one or more building stories.
The main conditioning factors of the simple overturning mechanism are the following.

Constraint condition of the affected wall:

- No constraint on the top;
- No connection between perpendicular walls.

Deficiencies and vulnerability associated with the mechanism:

- Absence of ring beams and tie rods;
- Deformable and poorly connected horizontal diaphragms;
- Bad connections between walls;
- Presence of thrusting elements;
- Two leafs walls or poorly connected vertical leafs.

Elements that shows the mechanism activation:

- Vertical cracks at the walls intersections;
- Out of plane overturning of the wall;
- Beams slippage of the horizontal elements.

Different variants of the mechanism:

- The overturning may involve one or more floors, in relation to the connections of the different horizontal elements;
- It may involve the entire wall thickness or only the external leaf, in relation to the wall characteristics;
- The mechanism may interests different geometries of the wall, in relation to the discontinuity or openings presence.

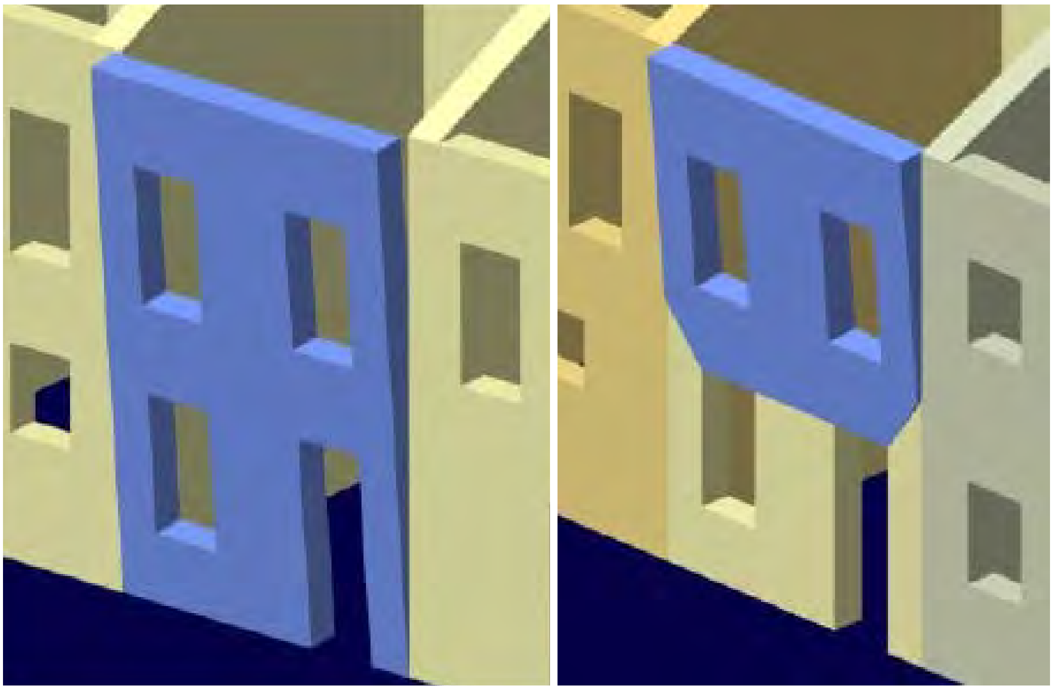


Fig. 5.3.1: Schema of the simple overturning – whole façade and upper wall portion

REFERENCE: Schede illustrative dei principali meccanismi di collasso locali
(Milano et al., 2009, p. 4)

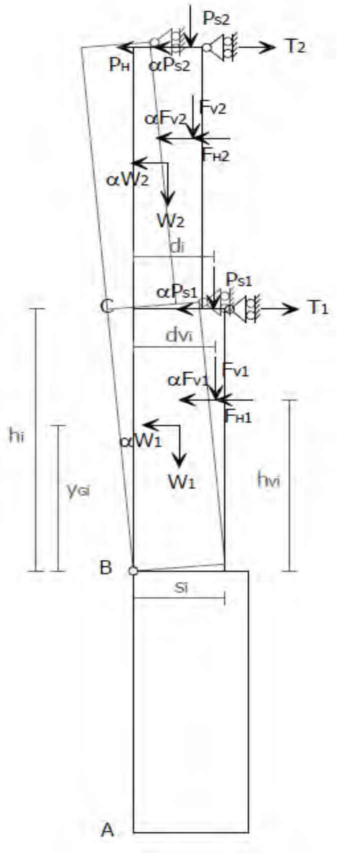


Fig. 5.3.2: Schema of the mechanism

REFERENCE: Schede illustrative dei principali meccanismi di collasso locali
(Milano et al., 2009, p. 4)

Calculation of the stabilizing moment:

$$M_S = \sum_{i=1}^{n} W_i \cdot \frac{s_i}{2} + \sum_{i=1}^{n} F_{Vi} \cdot d_{Vi} + \sum_{i=1}^{n} P_{Si} \cdot d_i + \sum_{i=1}^{n} T_i \cdot h_i \tag{5.15}$$

Calculation of the overturning moment:

$$M_R = \alpha \cdot \left(\sum_{i=1}^{n} W_i \cdot y_{Gi} + \sum_{i=1}^{n} F_{Vi} \cdot h_{Vi} + \sum_{i=1}^{n} P_{Si} \cdot h_i \right) + \sum_{i=1}^{n} F_{Hi} \cdot h_{Vi} + \sum_{i=1}^{n} P_H \cdot h_i$$

(5.16)

Calculation of the seismic masses multiplier:

$$\alpha = \frac{\sum_{i=1}^{n} W_i \cdot \frac{s_i}{2} + \sum_{i=1}^{n} F_{Vi} \cdot d_{Vi} + \sum_{i=1}^{n} P_{Si} \cdot d_i + \sum_{i=1}^{n} T_i \cdot h_i - \sum_{i=1}^{n} F_{Hi} \cdot h_i - \sum_{i=1}^{n} P_H \cdot h_i}{\sum_{i=1}^{n} W_i \cdot y_{Gi} + \sum_{i=1}^{n} F_{Vi} \cdot h_{Vi} + \sum_{i=1}^{n} P_{Si} \cdot h_i} \tag{5.17}$$

5.3.2 Vertical bending mechanism[10]

The mechanism is activated by the formation of a horizontal cylindrical hinge that divides the wall in two blocks and it is described by the mutual rotation of these blocks around the horizontal axis. (Fig. 5.3.3)

In case of clustered buildings, the wall structure resist to the bending stresses induced by orthogonal actions to its plane only if the result of the normal stresses is internal to the cross section. Otherwise the horizontal hinge arranges at that point and the bending mechanism activates.

The main influencing factors of the simple overturning mechanism are the following.

<u>Constraint condition of the affected wall:</u>

- Effective constraints on the top;
- No connection between perpendicular walls.

<u>Deficiencies and vulnerability associated with the mechanism</u>:

- Excessive wall slenderness;
- Poorly connected horizontal diaphragms;
- Presence of thrusting elements like arches or vaults;
- Two leafs walls or poorly connected vertical leafs.

[10] *Schede illustrative dei principali meccanismi di collasso locali negli edifici esistenti in muratura e dei relative modelli cinematici di analisi*. (Milano et al., 2009, p.9)

Elements that shows the mechanism activation:

- Vertical and horizontal cracks;
- Bulging and out of plumb walls;
- Beams slippage of the horizontal elements.

Different variants of the mechanism:

- The mechanism may involve one or more floors, in relation to the connections of the horizontal elements;
- It may involve the entire wall thickness or only the external leaf, in relation to the wall characteristics;
- The mechanism may interest different geometries of the wall, in relation to the discontinuity of the wall or the openings presence.

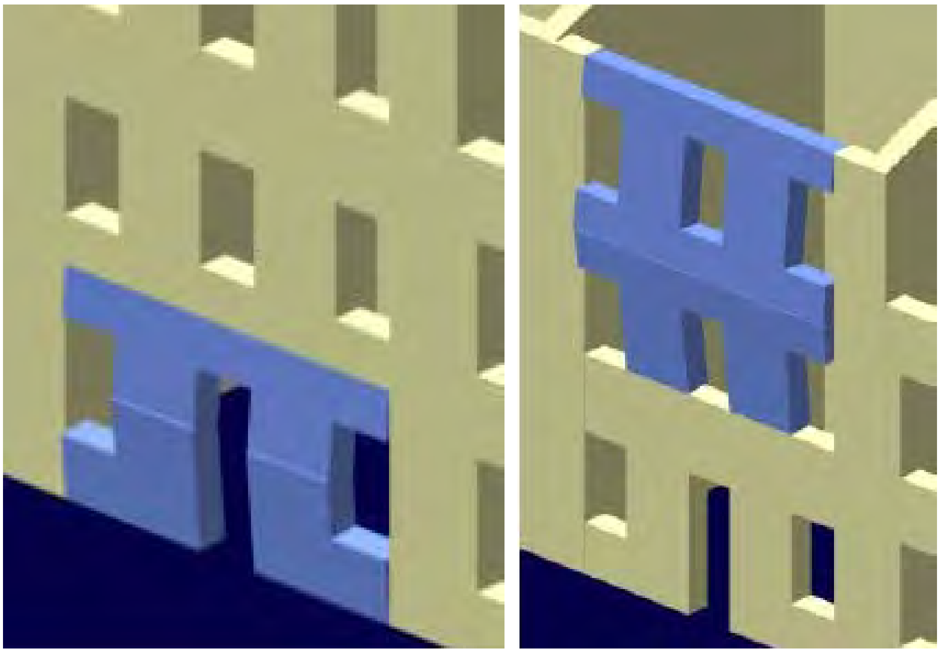


Fig. 5.3.3: Schema of the vertical bending mechanism – ground floor and upper wall portion

REFERENCE: Schede illustrative dei principali meccanismi di collasso locali
(Milano et al., 2009, p. 9)

The vertical bending mechanism analyzed in this thesis is the bending of one storey. This mechanism is activated with the formation of a horizontal cylindrical hinge that divides the wall included between two subsequent horizontal structures and it is described from the mutual rotation of these blocks around the horizontal axis due to out of plane actions (Fig. 5.3.4).

The formation of the horizontal hinge is hypothesized at the springing height of the vaults at the ground floor.

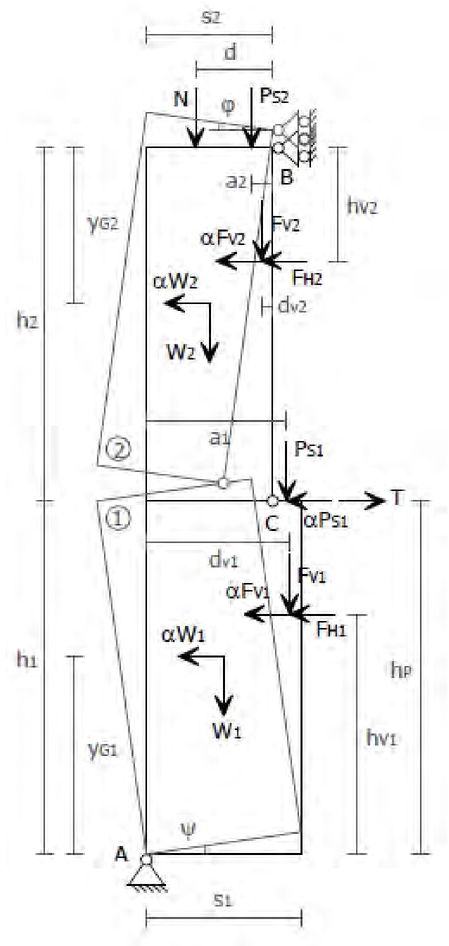


Fig. 5.3.4: Schema of the vertical bending mechanism

REFERENCE: Schede illustrative dei principali meccanismi di collasso locali

(Milano et al., 2009, p. 9)

Principle of Virtual Work:

$$\alpha\left[W_1 \cdot \delta_{1x} + W_2 \cdot \delta_{2x} + F_V \cdot \delta_{Vx}\right] + F_H \cdot \delta_{Vx} - W_1 \cdot \delta_{1y} - W_2 \cdot \delta_{2y} - N \cdot \delta_{Ny} - P_S \cdot \delta_{Py} - F_V \cdot \delta_{Vy} = 0 \quad (5.18)$$

Calculation of the seismic masses multiplier:

$$\alpha = \frac{E}{W_1 \cdot y_{G1} + F_{V1} \cdot h_{V1} + P_{S1} \cdot h_P + \left(W_2 \cdot y_{G2} + F_{V2} \cdot h_{V2}\right)\dfrac{h_1}{h_2}} \quad (5.19)$$

Where E represent the following expression:

$$
\begin{aligned}
E = {} & \frac{W_2}{2} s_1 + F_{V1} \cdot d_{V1} + \left(W_2 + P_{S2} + N + F_{V2}\right) \cdot s_2 + \\
& + \frac{h_1}{h_2}\left(\frac{W_2}{2} s_2 + P_{S2} \cdot a_2 + N \cdot d + F_{V2} \cdot d_{V2} - F_{H2} \cdot h_{V2}\right) + \\
& + P_{S1} \cdot a_1 - F_{H1} \cdot h_{V1} + T \cdot h_P
\end{aligned}
\tag{5.20}
$$

5.4 SIMPLE OVERTURNING MECHANISM

The analysis of the simple overturning mechanism is applied for the most widespread typologies of the historical center. Typologies that contain one structural unit or only buildings in Iosefin area are not considered for this analysis. The analysis is carried out considering the possible formation of the hinge at every floor, for example for a three stories building there are three possible hinge levels: at the ground level (level 0), at the level of the first floor (level 1) and at the level of the second floor (level 2). The wall portion above the hinge is considered rotating over the horizontal cylindrical axis at its base.

The a_g value used in the calculation is the value indicated in the Romanian Code and corresponds to a/g=0.2. As already defined in chapter 3.3, the material properties of the masonry are defined referring to the Italian normative and calculated with a knowledge level LC1, which correspond to a confidence factor of 1.35.[11]

The horizontal structures defined in the typological classification are divided into light and moderately light structures and it is indicated the presence of eventual vaults at the ground floor. For light structures, that includes both adjoining type timber structures and iron beams and little brick vaults structures, the permanent load is considered of 3 kN/m^2 while for moderately light structures, that correspond to concrete slab structures, the permanent load is considered of 4.5 kN/m^2. These values are taken from the permanent load values defined in Vulnus program in reference of these type of horizontal structures.[12] The vertical and horizontal vaults thrusts are evaluated using Arco program, as defined in chapter 3.3. The accidental loads follow the Italian regulation and are defined in chapter 3.3 as well.

The snow load has been calculated following the NTC code and it values 1.04 kN/m^2 but due to the low height above the sea level of the city of Timisoara, the snow load is annulled by its coefficient ψ on the seismic combination.

The retraction of the hinge is considered and, due to the bad quality of the connection between walls and horizontal structures, a friction coefficient of 0.05 is adopted to evaluate the friction forces.

[11] *§C8A.1.A.4 Costruzioni in muratura: livelli di conoscenza* (Circolare NTC 2008, p. 391)

[12] *§2.1 La scheda di rilievo Vulnus* (Manuale d’uso del programma Vulnus 4.0, 2009, p. 14)

5.4.1 Parameters description

One of the issues of a rapid survey on a large scale is the uncertainty of some kind of information that are hardly observable in many cases. During the in situ activities, only for 28 buildings a complete inspection has been possible and it corresponds to the 13% of the total. The characteristics that are more difficult to survey are interstorey height of upper and underground floors, wall thickness, type of horizontal structures and connection between constructive elements. Therefore the analysis of the local mechanisms has been applied considering a reference case, which is different for each micro-typology, by varying one parameter at time to consider the biggest number of possible cases. The obtained results refer both to the singular parameters and to the combination of them. As seen in Tab. 2.4.2 in chapter 2.4.1, every building typology can be subdivided in more micro-typology, referring to the ground floor thickness. Each of these micro-typologies has then 12 different cases: the reference case and 11 other cases corresponding to the 11 analyzed parameters, which are described below. For each case, a number of analysis equal to the typology stories number is made, one for each hinge level.

The reference case considers the wall thickness of the micro-typology for the ground floor and a thickness decreased of 15 cm after the first floor. This consideration has been made considering local architects indications about the most representative distribution of the wall thickness. The interstorey height is the average value of the structural units belonging to the analyzed typology and it is different for ground floor, upper floors and eventual basement. If the typology has vaults at the ground floor, the reference case considers a cross vault with a rise of 1m, which is the most common vaults type in the city center. The sept length and the considered area are referred to the most common plan module, defined in chapter 2.4.3: module 3. This module measures 5.8 x 4.10 m, for a total area of 23.6 m^2, and it usually collocated with its short side parallel to the façade. Therefore, the considered area depends from the horizontal structure type and it ranges from 5.9 m^2 in case of cross vaults (1/4 of the total area) to 11.8 m^2 (1/2 of the total area) in case of timber strictures, iron beams and little brick vaults or barrel vaults. The module most exposed side to out of plane mechanisms is the shorter one, so the sept length assumes the value of 4.1m. Initially the reference module was a varying parameter but a deeper analysis shows that even big differences of area and sept dimensions bring to almost irrelevant differences into the results, so it has been decided to consider it as a constant.

Finally, the analyzed parameters are:

1- Constant wall thickness along the entire building height: the ground floor thickness is considered constant in all the stories, from the ground to the top of the building.
2- Wall thickness that decreases on each floor: the wall thickness does not decrease of 15cm after the first floor and the it stays constant, but it decreases of 15cm on each floor until a minimum of 30cm.
3- Minimum interstorey height for upper floors
4- Maximum interstorey height for upper floors
5- Minimum interstorey height for the ground floor
6- Maximum interstorey height for the ground floor
7- Barrel vault: instead of a cross vault, a barrel vault with rise of 1m is considered.
8- Free standing wall: no loads from horizontal structures are considered in this case and the sept is considered free and disconnected along the entire building height.
9- Absence of vaults at the ground floor: a light and deformable horizontal structure is considered at the ground floor instead of the cross vault.
10- Vault rise of 1.5m: the cross vault is considered to have a rise of 1.5m instead of 1m. The new thrusts are calculated with Arco program.
11- Vault rise of 1.5m: the cross vault is considered to have a rise of 0.5m instead of 1m. The new thrusts are calculated with Arco program.

Parameters 3, 4, 5 and 6 represent the minimum and the maximum values of the interstorey high for the ground floor and the upper floors. These values has been calculated considering not the absolute minimum and the absolute maximum between the interstorey height of the considered buildings, but choosing a range of ±0.5 m or of ±1m from the average value. This consideration has been made to find a value that better represents the building majority and not the individual extreme case. In parameters 10 and 11 the variation of the vault rise causes the variation of the hinge height and consequently of the application point of the horizontal thrust, which is related to the hinge formation.

In macro-typology G, characterized by moderately heavy horizontal structure and concrete and timber trusses roof, the parameters related to the vault are not considered, so the parameters 7, 9, 10 and 11 are missing.

Not all the typologies have been directly analyzed because they can be included in the analysis of similar typologies, in correspondence of specific parameters. In particular:

- Typologies 12 and 13, characterized by metal reticular columns at the ground floor, can be included in typology 3 and 4 respectively, considering the simple overturning mechanism at the upper floors;
- Typology 22 is characterized by an inner RC septum which does not influence the façade overturning, so it can be included in typology 20;
- Typologies 27 and 28 can be included in typologies 24 and 26 because they differ from each other just for the presence of the basement and a lower interstorey height.

The Tab. 5.4.1 represents the analyzed typologies (green), the typologies that can be include in the analysis of similar typologies (orange) and the typologies that have been omitted because constituted of singular US or because they include only structural units located in Iosefin (red). The micro-typologies written in grey have been added subsequently to consider all the possible wall thickness, even if no structural units have been surveyed with these measures. The typology one is the only case that has been analyzed but the results result to be all negative and consequently it was not possible to carry on the analysis.

In typologies 2 and 7, characterized by two stories and two stories with basement, only the values of the mechanism at the upper floors for micro-typologies A and B have been adopted because the values of the mechanism at the ground floor result negative. In Tab. 5.4.1 they are colored in light green.

MACRO-TYPOLOGY	TYPOLOGY	MICRO-TYPOLOGY	STORIES NUMBER	U.S.	GROUND FLOOR THICKNESS	AVERAGE INTERSTOREY HEIGHT- GROUND FLOOR	INTERSTOREY VARIATION - GROUND FLOOR	AVERAGE INTERSTOREY HEIGHT- UPPER FLOORS	INTERSTOREY VARIATION - UPPER FLOORS	AVERAGE INTERSTOREY HEIGHT- BASEMENT
A	1	1A	1	36, 37	45	3,8	± 0,5	-	-	-
		1B		152, 241	60					
	2	2A	2	7, 44, 74, 75, 98	45	4	± 0,5	4,3	± 0,5	-
		2B		8, 18, 19, 20, 29, 30, 31, 55, 58, 59, 73, 77, 84, 97, 115, 120, 121, 151, 175, 193 202, 228, 301	60					
		2C		71, 80, 117, 145, 154	75					
		2D		94, 95, 167, 185	90					
	3	3A	3	49, 50, 51	45	4,3	± 1	4,2	± 0,5	-
		3B		4, 5, 53, 76, 96, 103, 104,105, 116, 118, 123, 124, 140, 174, 178, 192	60					
		3C		81, 109, 130, 134	75					
		3D		79, 86, 87, 88, 137, 138, 141, 166, 169B, 182, 183, 188, 189, 190	90					
	4	4A	4	25, 56, 133	60	4,2	± 0,5	4	± 0,5	-
		4B		129	75					
		4C		90, 92, 93, 128, 135, 155, 156, 187	90					
	5		5	157	90	4,8	-	3,7	-	-
	6		6	195	90	3,5	-	3,3	-	-
	7	7 A	2+B	_	45	4	± 0,5	5	± 0,5	1,3
		7B		35, 38, 41, 61, 62, 64, 149, 150, 172, 211, 212, 213, 229, 230, 231, 233, 236, 237	60					

		7C		68	75					
		7D		28, 100, 114, 217	90					
	8	8A	3+B	46	45	4,2	± 0,5	4,4	± 0,5	1,1
		8B		9, 24, 26, 43, 47, 54, 57, 60, 65, 168, 173, 184, 200, 201, 203, 207, 221, 234, 235	60					
		8C		69, 70, 143, 153	75					
		8D		101, 170	90					
	9		4+B	169A	90	4,7	-	4,6	-	-
A Iosef in	10	10A	1+B	224, 238, 239, 242	45	4,7	± 0,5	-	-	1,5
		10B		208, 209, 210, 215, 216, 218, 219, 220, 232	60					
	11		3+B	204	60	3,4	-	-	-	-
B	12		3	89, 127	45	4,9	-	5	-	-
	13		4	132, 144, 158	60	4,8	± 0,5	4,2	-	-
C	14		2	139	60	4	-	3,9	-	-
D	15		3	72	60	4,7	-	4,9	-	-
	16		3+B	181	105	4,4	-	2,8	-	1,2
E	17	17A	2	82	45	4,2	± 0,5	4,7	± 1	-
		17B		83, 85, 111, 186	60					
		17C		-	75					
		17D		-	90					
	18	18 A	3	-	45	4,3	± 0,5	4,2	± 0,5	-
		18B		52, 105,125, 131	60					
		18C		-	75					
		18D		-	90					
	19		1+B	32	60	4,45	-	-	-	0,7
	20	20 A	3+B	-	45	4	± 0,5	4,2	-	1,4
		20B		2	60					
		20C		148	75					
		20D		102, 159	90					
E Iosef in	21		1+B	225	45	4,5	-	-	-	0,5
F	22		3+B	3	60	4,1	-	3,35	-	1,9
G	23		1	22	60	3,8	-	-	-	-
	24	24 A		-	45	4	± 0,5	4,5	± 1	-

		24B	2	17, 21, 23, 63, 119	60					
		24C		146	75					
		24D		-	90					
	25	25A	3	107	45	3,6	± 0,5	3,3	± 0,5	-
		25B		108, 110, 164, 179	60					
		25C		-	75					
		25D		-	90					
	26	26B	4	91	60	3,7	-	4	± 0,5	-
		26C		-	75					
		26D		180	90					
	27		2+B	48	45	3,7	-	4,8	-	0,8
	28		4+B	99	60	3,5	-	3,5	-	1,5
G Iosef in	29		1	223	45	4,2	-	-	-	-
	30	30A	1+B	226,227,243	45	3,9	± 0,5	-	-	0,8
		30B		222,245	60					
H	31		2+B	34,206	60	4	± 0,5	4,8	± 0,5	1,1
	32		3+B	106	45	3,6	-	3,6	-	1,1
H Iosef in	33		3+B	205	60	3,8	-	3,8	-	1,9

Tab. 5.4.1: Analyzed typologies and interstorey height variations

Not all the parameters have been analyzed in each typology because in few cases they result to be unnecessary or incongruous with the typology. The cases in which a parameter is not considered are the following.

- Typologies characterized by a wall thickness at the ground floor of 45cm and 30cm at the upper floor do not have the Parameter 2 (wall thickness that decreases on each floor) because it has already reached the minimum thickness;
- Typologies without brick vaults at the ground floor do not consider parameters 7, 10 and 11 because they are related to the vault presence;
- If all the structural units of the same typology have the same interstorey height, the parameters 3 and 4 of 5 and 6 are not considered, because they refer to the interstorey variation.
- In typologies with four stories the wall thickness of 45cm at the ground floor is not considered because it results incongruous with the great building height.

5.4.2 Verification of the mechanism

For each analyzed case, a linear and a non-linear kinematic analysis have been applied, following the method shown in chapter 5.2. The values of the horizontal load multiplier α_0 has been reported and compared to each other. In the linear kinematic analysis, the value of the equivalent spectral acceleration a_0* is compared with the inequality (5.8), in case of the hinge at level 0, and with the maximum value between (5.8) and (5.9) inequalities in case of the hinge at upper levels. In the non-linear kinematic analysis, the value of the ultimate displacement capacity d_u* is compared with the inequality (5.13), in case of the hinge at level 0, and with the maximum value between (5.13) and (5.14) inequalities in case of the hinge at upper levels.

The verification values for typology 3 are shown in the following tables (Tab. 5.4.2, Tab. 5.4.3, Tab. 5.4.4, Tab. 5.4.5) as example, while the values for the other typologies are shown in annex D1. The typology 3 is characterized by three stories high buildings, with an average interstorey height of 4.3m at the ground floor and 4.2m at the upper floors. The interstorey variation is of ±1m for the ground floor and ±0.5m at upper floors. It is subdivided in four micro-typologies:

- Micro-typology 3A has a wall thickness of 45cm and includes 3 structural units;
- Micro-typology 3B has a wall thickness of 60cm and includes 16 structural units;
- Micro-typology 3C has a wall thickness of 75cm and includes 4 structural units;
- Micro-typology 3D has a wall thickness of 90cm and includes 14 structural units.

3A										
PARAMETER	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0	0,0167	0,1645	1,4715	0,1118	NO	0,0592	0,0560	1,0577	YES
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0540	3,0337	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES
1	0	0,0294	0,2767	1,4715	0,1880	NO	0,1043	0,0560	1,8629	YES
	1	0,0730	0,6553	1,4715	0,4453	NO	0,1787	0,0539	3,3179	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
2	0	-	-	-	-	-	-	-	-	-
	1	-	-	-	-	-	-	-	-	-
	2	-	-	-	-	-	-	-	-	-
3	0	0,0130	0,1267	1,4715	0,0861	NO	0,0426	0,0560	0,7600	NO
	1	0,0691	0,6115	1,4715	0,4156	NO	0,1562	0,0563	2,7739	YES
	2	0,0768	0,6254	2,5561	0,2447	NO	0,0934	0,0844	1,1065	YES
4	0	0,0197	0,1950	1,4715	0,1325	NO	0,0750	0,0560	1,3398	YES
	1	0,0608	0,5419	1,4715	0,3683	NO	0,1710	0,0536	3,1926	YES
	2	0,0642	0,5246	2,1817	0,2404	NO	0,0956	0,0900	1,0624	YES
5	0	0,0284	0,2845	1,4715	0,1934	NO	0,0937	0,0560	1,6737	YES
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0560	2,9254	YES
	2	0,0699	0,5705	2,3964	0,2381	NO	0,0947	0,0825	1,1482	YES
6	0	0,0079	0,0764	1,4715	0,0519	NO	0,0298	0,0560	0,5328	NO
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0634	2,5859	YES
	2	0,0699	0,5705	2,3029	0,2477	NO	0,0947	0,0917	1,0329	YES
7	0	-0,0343	-0,3433	1,4715	-0,2333	NO	-0,1174	0,0560	-2,0958	NO
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0540	3,0337	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES
8	0	0,0634	0,6344	1,4715	0,4311	NO	0,1998	0,0560	3,5678	YES
	1	0,0526	0,4779	1,4715	0,3248	NO	0,1102	0,0560	1,9683	YES
	2	0,0633	0,4602	2,3528	0,1956	NO	0,0531	0,0763	0,6960	NO
9	0	0,0741	0,7112	1,4715	0,4833	NO	0,2674	0,0560	4,7750	YES
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0540	3,0337	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES
10	0	0,0366	0,3656	1,4715	0,2484	NO	0,1293	0,0560	2,3096	YES
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0540	3,0337	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES
11	0	-0,0390	-0,3781	1,4715	-0,2570	NO	-0,1391	0,0560	-2,4833	NO
	1	0,0645	0,5735	1,4715	0,3898	NO	0,1638	0,0540	3,0337	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES

Tab. 5.4.2: Verification of simple overturning mechanism – Micro-typology 3A

3B										
PARAMETER	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0	0,0333	0,3238	1,4715	0,2200	NO	0,1158	0,0560	2,0678	YES
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
1	0	0,0428	0,4034	1,4715	0,2741	NO	0,1498	0,0560	2,6747	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
2	0	0,0264	0,2621	1,4715	0,1781	NO	0,0891	0,0560	1,5912	YES
	1	0,0703	0,6517	1,4715	0,4429	NO	0,1709	0,0560	3,0512	YES
	2	0,0699	0,5705	2,3528	0,2425	NO	0,0947	0,0873	1,0848	YES
3	0	0,0314	0,3036	1,4715	0,2064	NO	0,1011	0,0560	1,8048	YES
	1	0,0795	0,7120	1,4715	0,4839	NO	0,1733	0,5770	0,3003	NO
	2	0,1026	0,8385	2,5561	0,3280	NO	0,1159	0,0819	1,4140	YES
4	0	0,0346	0,3382	1,4715	0,2298	NO	0,1294	0,0560	2,3113	YES
	1	0,0677	0,6088	1,4715	0,4137	NO	0,1838	0,0560	3,2815	YES
	2	0,0835	0,6814	2,1817	0,3123	NO	0,1153	0,0875	1,3181	YES
5	0	0,0435	0,4297	1,4715	0,2920	NO	0,1407	0,0560	2,5120	YES
	1	0,0730	0,6553	1,4715	0,4453	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3964	0,3140	NO	0,1157	0,0800	1,4458	YES
6	0	0,0253	0,2434	1,4715	0,1654	NO	0,0943	0,0560	1,6844	YES
	1	0,0730	0,6553	1,4715	0,4453	NO	0,1787	0,0622	2,8747	YES
	2	0,0921	0,7524	2,3029	0,3267	NO	0,1157	0,0891	1,2989	YES
7	0	-0,0081	-0,0803	1,4715	-0,0546	NO	-0,0277	0,0560	-0,4944	NO
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
8	0	0,0715	0,7024	1,4715	0,4774	NO	0,2280	0,0560	4,0709	YES
	1	0,0664	0,6033	1,4715	0,4100	NO	0,1390	0,0560	2,4815	YES
	2	0,0950	0,6903	2,3528	0,2934	NO	0,0794	0,0762	1,0423	YES
9	0	0,0781	0,7472	1,4715	0,5078	NO	0,2766	0,0560	4,9396	YES
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
10	0	0,0489	0,4823	1,4715	0,3278	NO	0,1700	0,0560	3,0351	YES
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
11	0	-0,0104	-0,1004	1,4715	-0,0682	NO	-0,0366	0,0560	-0,6540	NO
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES

Tab. 5.4.3: Verification of simple overturning mechanism – Micro-typology 3B

3C										
PARAMETER	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0	0,0467	0,4516	1,4715	0,3069	NO	0,1607	0,0560	2,8690	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
1	0	0,0545	0,5136	1,4715	0,3491	NO	0,1887	0,0560	3,3688	YES
	1	0,0947	0,8563	1,4715	0,5819	NO	0,2214	0,0569	3,8909	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES
2	0	0,0416	0,4090	1,4715	0,2779	NO	0,1396	0,0560	2,4935	YES
	1	0,0799	0,7397	1,4715	0,5027	NO	0,1890	0,0560	3,3749	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES
3	0	0,0465	0,4474	1,4715	0,3041	NO	0,1479	0,0560	2,6409	YES
	1	0,0919	0,8275	1,4715	0,5624	NO	0,1949	0,0560	3,4796	YES
	2	0,1314	1,0716	2,5561	0,4192	NO	0,1402	0,0803	1,7466	YES
4	0	0,0465	0,4519	1,4715	0,3071	NO	0,1723	0,0560	3,0761	YES
	1	0,0764	0,6899	1,4715	0,4689	NO	0,2023	0,0560	3,6121	YES
	2	0,1055	0,8557	2,1817	0,3922	NO	0,1378	0,0859	1,6045	YES
5	0	0,0563	0,5517	1,4715	0,3749	NO	0,1798	0,0560	3,2109	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3964	0,3976	NO	0,1391	0,0784	1,7731	YES
6	0	0,0390	0,3733	1,4715	0,2537	NO	0,1438	0,0560	2,5683	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3029	0,4137	NO	0,1391	0,0874	1,5913	YES
7	0	0,0124	0,1226	1,4715	0,0833	NO	0,0403	0,0560	0,7200	NO
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
8	0	0,0794	0,7727	1,4715	0,5251	NO	0,2546	0,0560	4,5469	YES
	1	0,0802	0,7287	1,4715	0,4952	NO	0,1676	0,0560	2,9927	YES
	2	0,1267	0,9204	2,3528	0,3912	NO	0,1055	0,0761	1,3866	YES
9	0	0,0834	0,7964	1,4715	0,5412	NO	0,2917	0,0560	5,2088	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
10	0	0,0596	0,5827	1,4715	0,3960	NO	0,2047	0,0560	3,6562	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
11	0	0,0063	0,0586	1,4715	0,0398	NO	0,0220	0,0560	0,3923	NO
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES

Tab. 5.4.4: Verification of simple overturning mechanism – Micro-typology 3C

3D										
PARAMETER	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0	0,0584	0,5626	1,4715	0,3823	NO	0,1993	0,0560	3,5588	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES
1	0	0,0651	0,6140	1,4715	0,4173	NO	0,2236	0,0560	3,9934	YES
	1	0,1066	0,9662	1,4717	0,6565	NO	0,2455	0,0560	4,3847	YES
	2	0,1718	1,3803	2,3528	0,5867	NO	0,1880	0,0810	2,3205	YES
2	0	0,0543	0,5306	1,4715	0,3606	NO	0,1816	0,0560	3,2429	YES
	1	0,0909	0,8402	1,4715	0,5710	NO	0,2108	0,0560	3,7635	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES
3	0	0,0597	0,5728	1,4715	0,3893	NO	0,1883	0,0560	3,3631	YES
	1	0,1054	0,9520	1,4715	0,6469	NO	0,2188	0,0560	3,9071	YES
	2	0,1621	1,3160	2,5561	0,5148	NO	0,1652	0,0790	2,0901	YES
4	0	0,0569	0,5502	1,4715	0,3739	NO	0,2090	0,0560	3,7329	YES
	1	0,0860	0,7791	1,4715	0,5294	NO	0,2237	0,0560	3,9945	YES
	2	0,1290	1,0397	2,1817	0,4765	NO	0,1614	0,0847	1,9052	YES
5	0	0,0679	0,6614	1,4715	0,4495	NO	0,2148	0,0560	3,8352	YES
	1	0,0947	0,8563	1,4715	0,5819	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,0811	0,5591	NO	0,1634	0,0560	2,9176	YES
6	0	0,0506	0,4833	1,4715	0,3284	NO	0,1853	0,0560	3,3094	YES
	1	0,0947	0,8563	1,4715	0,5819	NO	0,2214	0,0608	3,6395	YES
	2	0,1439	1,1635	2,3029	0,5052	NO	0,1634	0,0862	1,8959	YES
7	0	0,0287	0,2799	1,4715	0,1902	NO	0,0964	0,0560	1,7212	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES
8	0	0,0873	0,8441	1,4715	0,5736	NO	0,2806	0,0560	5,0105	YES
	1	0,0940	0,8542	1,4717	0,5804	NO	0,1961	0,0560	3,5016	YES
	2	0,1583	1,1505	2,3528	0,4890	NO	0,1314	0,0760	1,7283	YES
9	0	0,0895	0,8529	1,4715	0,5796	NO	0,3098	0,0560	5,5329	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES
10	0	0,0694	0,6746	1,4715	0,4585	NO	0,2364	0,0560	4,2221	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES
11	0	0,0291	0,2788	1,4715	0,1895	NO	0,0999	0,0560	1,7844	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES

Tab. 5.4.5: Verification of simple overturning mechanism – Micro-typology 3D

In micro-typology 3A the not verified cases are:

- Parameter 3 (minimum interstorey height for upper floors) with the hinge at level 0;
- Parameter 6 (maximum interstorey height for the ground floor) with the hinge at level 0;
- Parameter 7 (barrel vault) with the hinge at level 0;
- Parameter 8 (free standing wall) with the hinge at level 2;
- Parameter 11(vault rise at 0.5m) with the hinge at level 0.

In micro-typology 3B the not verified cases are:

- Parameter 3 (minimum interstorey height for upper floors) with the hinge at level 1;
- Parameter 7 (barrel vault) with the hinge at level 0;
- Parameter 11(vault rise at 0.5m) with the hinge at level 0.

In micro-typology 3C the not verified cases are:

- Parameter 7 (barrel vault) with the hinge at level 0;
- Parameter 11(vault rise at 0.5m) with the hinge at level 0.

Parameters 7 and 11 with the hinge at level 0 result not verified for all the three cases, because the horizontal thrust is considerably increased and promotes the out of plane mechanism. Micro-typologies 3A and 3B, which have a low wall thickness at the upper floors, are more vulnerable to interstorey height variations. The only collapse at level 2 occurs in micro-typology 3A (upper floors thickness of 30cm) in correspondence of the parameter "free standing wall".

All cases result not verified with the linear kinematic analysis, but the majority of them results verified with the non-linear kinematic analysis. It is possible to observe in the graphics below (Fig. 5.4.1 and Fig. 5.4.2) that the percentage of verified cases increases with the increment of the wall thickness, reaching 100% in micro-typology 3D.

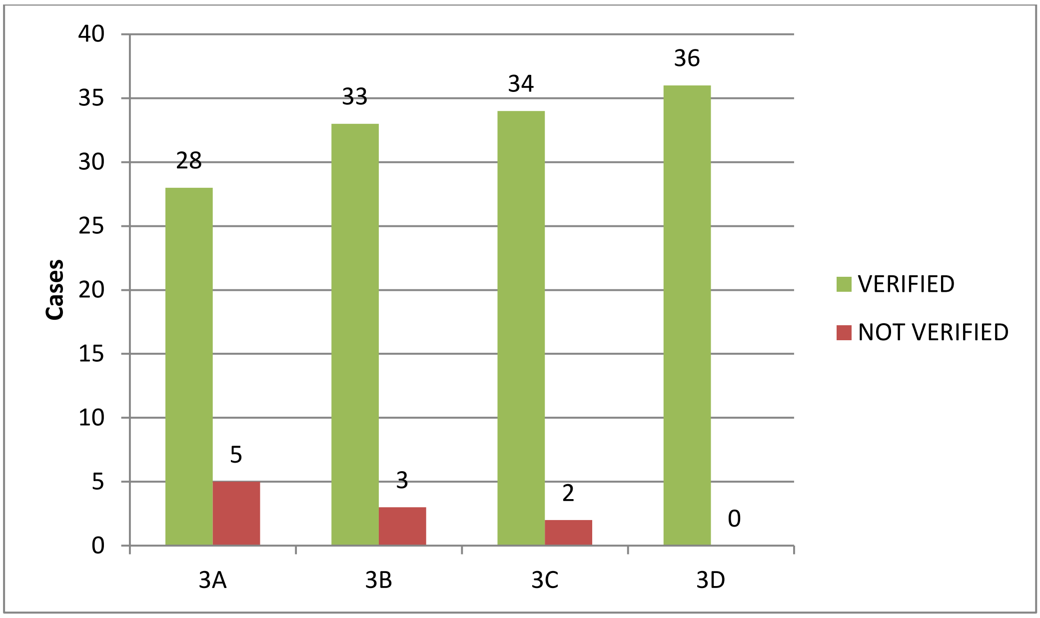


Fig. 5.4.1: Verification of simple overturning mechanism – Cases number

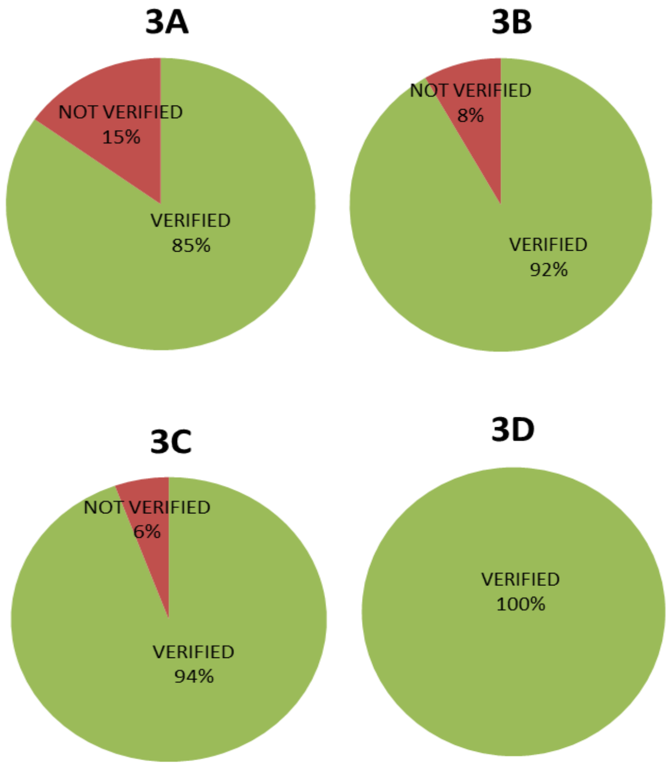


Fig. 5.4.2: Verification of simple overturning mechanism – Percentage

In typologies 2, 7 and 17 , characterized by two stories and brick vaults at the ground floor, in correspondence of micro-typologies A and B (wall thickness of 45 and 60cm), the simple overturning mechanism is usually verified at the upper level but is not verified at the ground floor. The only exceptions are the

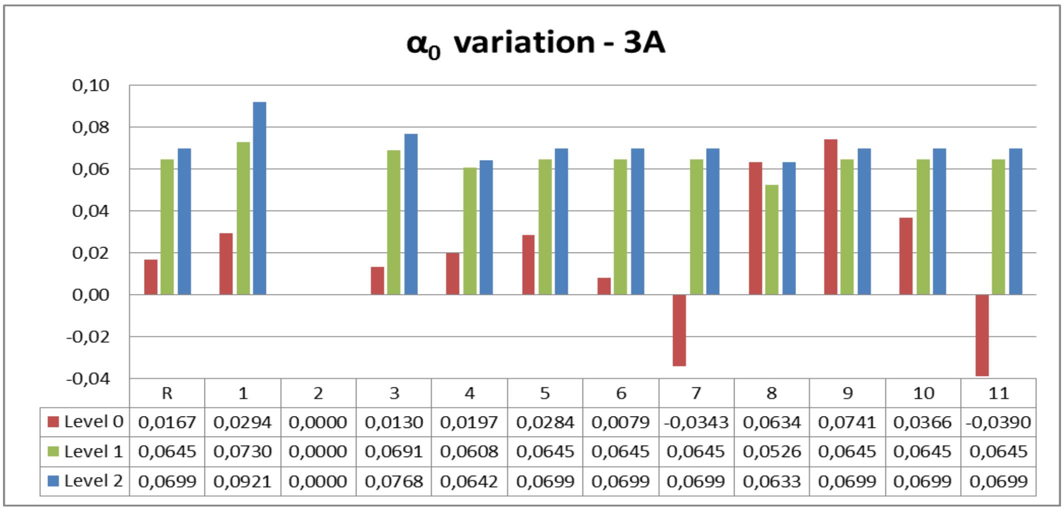
parameters 8 (unstressed wall) and parameter 9 (absence of vaults at the ground floor) which do not consider the vault thrust. In micro-typology C (wall thickness of 75cm) the mechanism is not verified in correspondence of parameter 11 (vault rise at 0.5m) at the ground floor, due to the considerable horizontal thrust.
The addition of the basement in a typology characterized by three stories, such as typology 8, increases the force at the last floor and consequently parameters 3 (minimum interstorey height for upper floors), 6 (maximum interstorey height for the ground floor) and 8 (unstressed wall) result not verified for the last floor but with a safety coefficient very close to the verification. In the same typology the parameter 11 (vault rise at 0.5m) results verified only for the wall thickness of 90cm.
The typology 4 is the only one characterized by four stories and it does not consider the micro-typology A, because a wall thickness of 45cm is not valuable for a four stories building. all the analysis are verified, except for the parameter 2 (wall thickness that decreases on each floor) in all the micro-typologies and the parameter 8 (unstressed wall) for a wall thickness of 60cm.
all the macro-typology G is verified, except for the parameter 8 (unstressed wall) which is not verified at the last floor for the ground floor thickness of 45cm.

5.4.3 Parameters analysis

Into the same micro-typology, a comparison between the values of α_0, a_0*, d_0*and d_u* of each parameter has been made. In this way it is possible to observe the influence of each parameter in the mechanism behavior. The histograms (Fig. 5.4.3, Fig. 5.4.4, Fig. 5.4.5, Fig. 5.4.6) show the value variations of the multiplier α_0 in each micro-typology of the building typology 3.
It is possible to observe that the value of α_0:

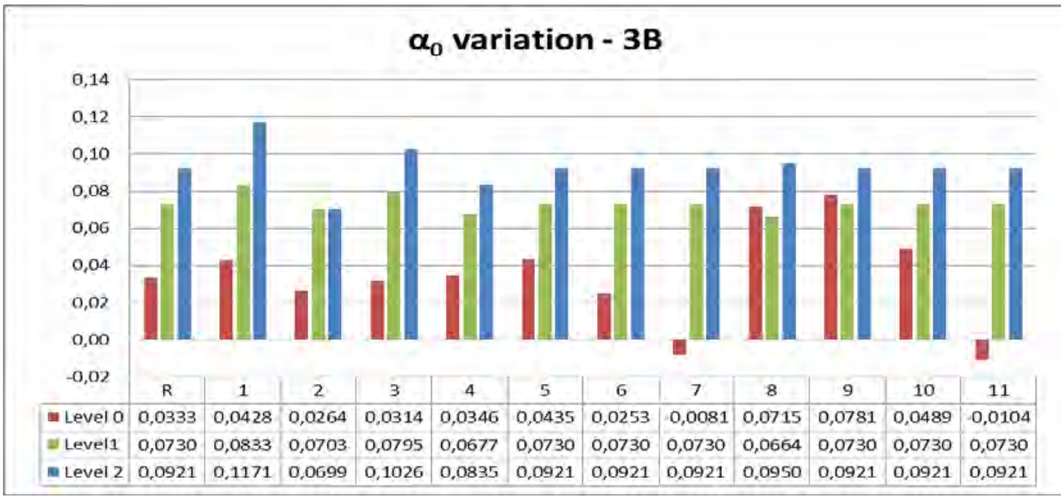
- Is usually higher in correspondence of the hinge at level 2 and lower in correspondence of level 0. This distribution shows that the most vulnerable situation is the one with the hinge formation at the ground level. The parameters that do not have the same behavior are parameters 8 (free standing wall) and parameter 9 (absence of vaults at the ground floor). In this two cases the α_0 value at the level 0 is higher of both level 1 and 2 for micro-typology 3A, is higher than level 1 for micro-typology 3B, is equal to level 1 value for micro-typology 3C and is lower for micro-typology 3D.

- Reaches the minimum for parameters 7 (barrel vault) and parameter 11 (vault rise of 1.5m) in case of hinge at level 0 because the horizontal thrust of these types of vaults is considerably higher than the reference case, and it encourages the out of plane mechanism.
- Reaches the maximum for parameter 1 (constant wall thickness along the entire building height) in case of hinge at level 2 because the increment of the wall thickness in the upper floors improve the stabilizing action of the mechanism.
- Can be negative if the overturning moment is higher than the stabilizing moment when the mechanism is not activated. This condition should bring to the collapse of the analyzed wall without the intervention of the seismic action and, as the building is in fact not collapsed, it is not an acceptable result. This means that possibly there are reinforcing elements that have not be surveyed. Negative values of α_0 appear more often in buildings with one or two stories and with low wall thickness, as can be seen in annex D1. These cases will be analyzed considering the local mechanism of vertical bending in chapter 5.5.



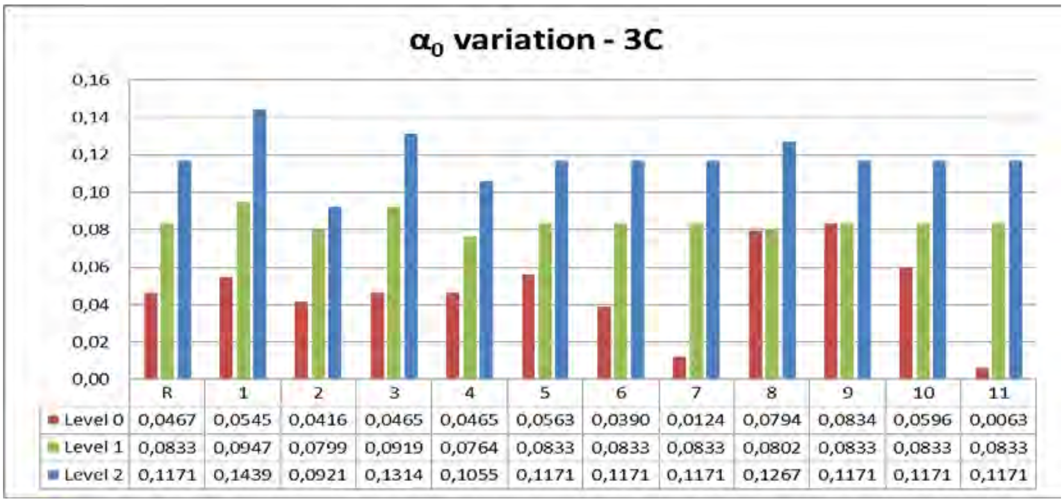
	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,0167	0,0294	0,0000	0,0130	0,0197	0,0284	0,0079	-0,0343	0,0634	0,0741	0,0366	-0,0390
Level 1	0,0645	0,0730	0,0000	0,0691	0,0608	0,0645	0,0645	0,0645	0,0526	0,0645	0,0645	0,0645
Level 2	0,0699	0,0921	0,0000	0,0768	0,0642	0,0699	0,0699	0,0699	0,0633	0,0699	0,0699	0,0699

Fig. 5.4.3: Variation of α_0 values – micro-typology 3A



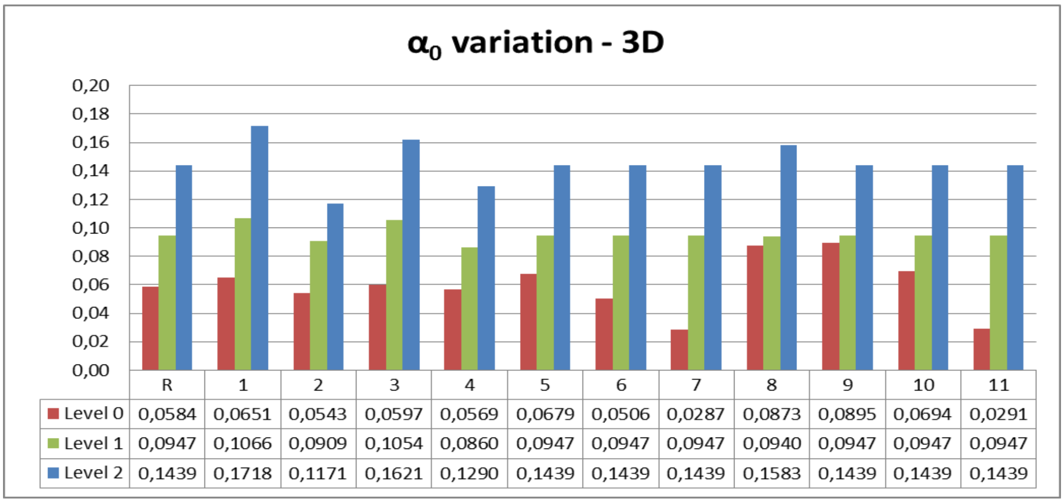
	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,0333	0,0428	0,0264	0,0314	0,0346	0,0435	0,0253	-0,0081	0,0715	0,0781	0,0489	-0,0104
Level1	0,0730	0,0833	0,0703	0,0795	0,0677	0,0730	0,0730	0,0730	0,0664	0,0730	0,0730	0,0730
Level 2	0,0921	0,1171	0,0699	0,1026	0,0835	0,0921	0,0921	0,0921	0,0950	0,0921	0,0921	0,0921

Fig. 5.4.4: Variation of α_0 *values – micro-typology 3B*



	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,0467	0,0545	0,0416	0,0465	0,0465	0,0563	0,0390	0,0124	0,0794	0,0834	0,0596	0,0063
Level 1	0,0833	0,0947	0,0799	0,0919	0,0764	0,0833	0,0833	0,0833	0,0802	0,0833	0,0833	0,0833
Level 2	0,1171	0,1439	0,0921	0,1314	0,1055	0,1171	0,1171	0,1171	0,1267	0,1171	0,1171	0,1171

Fig. 5.4.5: Variation of α_0 *values – micro-typology 3C*



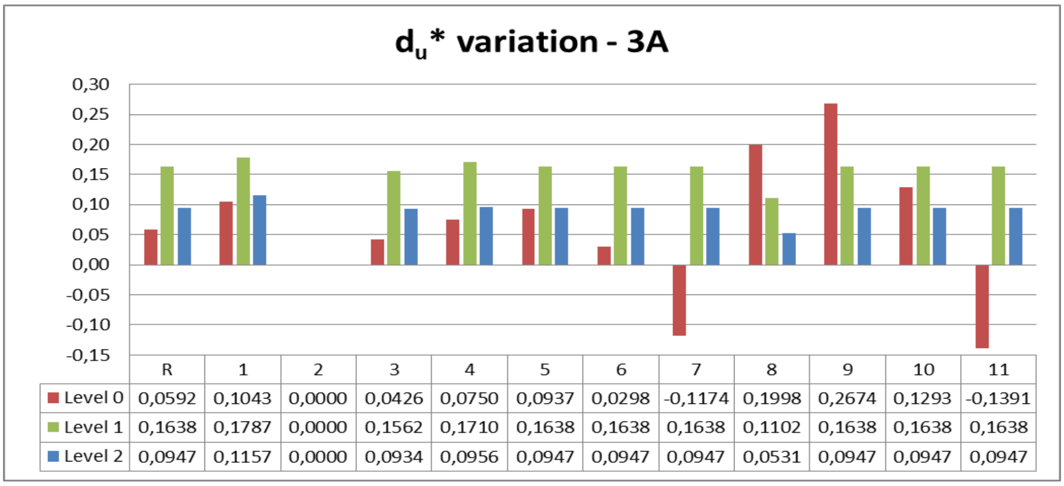
	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,0584	0,0651	0,0543	0,0597	0,0569	0,0679	0,0506	0,0287	0,0873	0,0895	0,0694	0,0291
Level 1	0,0947	0,1066	0,0909	0,1054	0,0860	0,0947	0,0947	0,0947	0,0940	0,0947	0,0947	0,0947
Level 2	0,1439	0,1718	0,1171	0,1621	0,1290	0,1439	0,1439	0,1439	0,1583	0,1439	0,1439	0,1439

Fig. 5.4.6: Variation of α_0 *values – micro-typology 3D*

The same comparison can be made for the ultimate displacement capacity d_u* . The histograms below (Fig. 5.4.7, Fig. 5.4.8, Fig. 5.4.9, Fig. 5.4.10) show the variations of value d_u* in each micro-typology of the building typology 3.

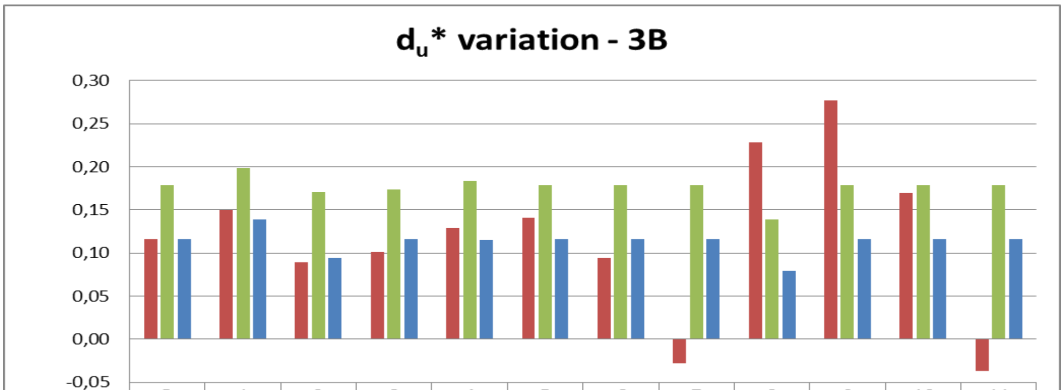
In the micro-typology 3A, the lower value of d_u* usually corresponds to the hinge at level 0, while the higher value to the hinge at level 1. Exceptions are made for parameters 8 (free standing wall), 9 (absence of vaults at the ground floor) and 10 (vault rise at 1.5m). In these cases the higher value corresponds to the hinge at level 0, for parameters 8 and 9, and for parameter 10 the lower value becomes the one at level 2. The micro-typology 3B shows more or less the same behavior, while in micro-typologies 3C and 3D the lower value corresponds to the hinge at level 2 in almost all the parameters.

A comparison has been made also for a_0*and d_0* values as it has been made for α_0 and d_u*. The histograms have the same trend of d_u* ones, and the same considerations made above can be extended to a_0*and d_0*comparison.



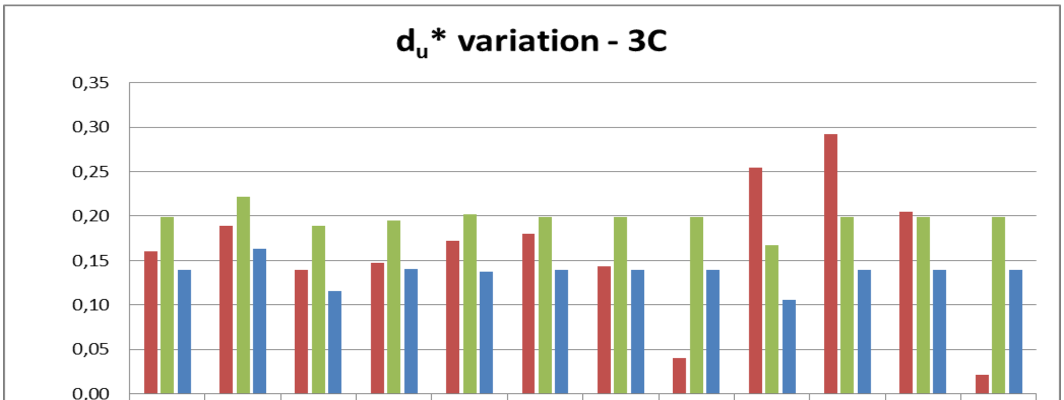
	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,0592	0,1043	0,0000	0,0426	0,0750	0,0937	0,0298	-0,1174	0,1998	0,2674	0,1293	-0,1391
Level 1	0,1638	0,1787	0,0000	0,1562	0,1710	0,1638	0,1638	0,1638	0,1102	0,1638	0,1638	0,1638
Level 2	0,0947	0,1157	0,0000	0,0934	0,0956	0,0947	0,0947	0,0947	0,0531	0,0947	0,0947	0,0947

Fig. 5.4.7: Variation of d_u values – micro-typology 3A*



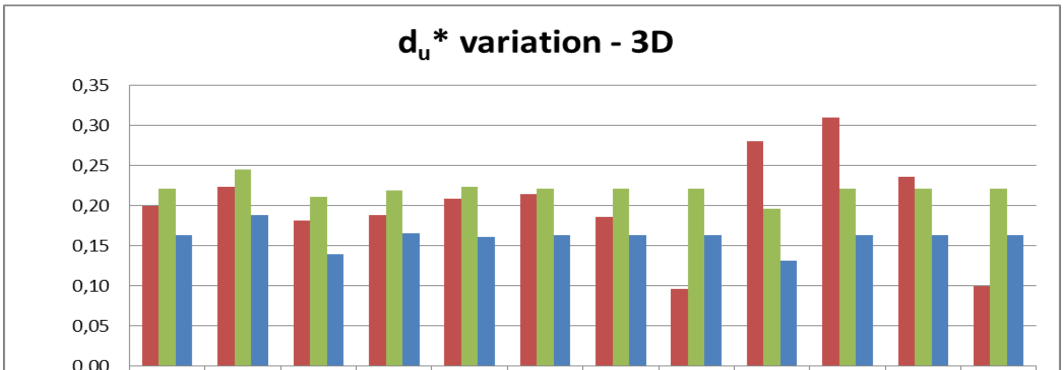
	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,1158	0,1498	0,0891	0,1011	0,1294	0,1407	0,0943	-0,0277	0,2280	0,2766	0,1700	-0,0366
Level 1	0,1787	0,1987	0,1709	0,1733	0,1838	0,1787	0,1787	0,1787	0,1390	0,1787	0,1787	0,1787
Level 2	0,1157	0,1391	0,0947	0,1159	0,1153	0,1157	0,1157	0,1157	0,0794	0,1157	0,1157	0,1157

Fig. 5.4.8: Variation of d_u values – micro-typology 3B*



	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,1607	0,1887	0,1396	0,1479	0,1723	0,1798	0,1438	0,0403	0,2546	0,2917	0,2047	0,0220
Level 1	0,1987	0,2214	0,1890	0,1949	0,2023	0,1987	0,1987	0,1987	0,1676	0,1987	0,1987	0,1987
Level 2	0,1391	0,1634	0,1157	0,1402	0,1378	0,1391	0,1391	0,1391	0,1055	0,1391	0,1391	0,1391

Fig. 5.4.9: Variation of d_u values – micro-typology 3C*

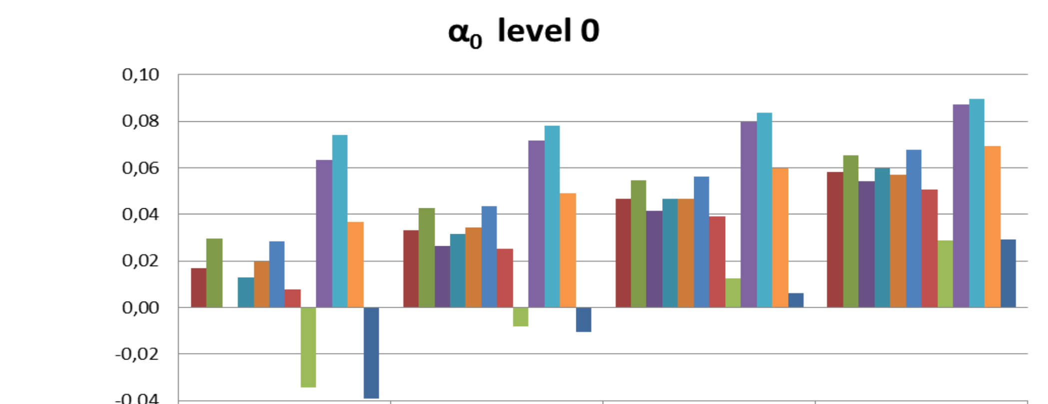


	R	1	2	3	4	5	6	7	8	9	10	11
Level 0	0,1993	0,2236	0,1816	0,1883	0,2090	0,2148	0,1853	0,0964	0,2806	0,3098	0,2364	0,0999
Level 1	0,2214	0,2455	0,2108	0,2188	0,2237	0,2214	0,2214	0,2214	0,1961	0,2214	0,2214	0,2214
Level 2	0,1634	0,1880	0,1391	0,1652	0,1614	0,1634	0,1634	0,1634	0,1314	0,1634	0,1634	0,1634

Fig. 5.4.10: Variation of d_u values – micro-typology 3D*

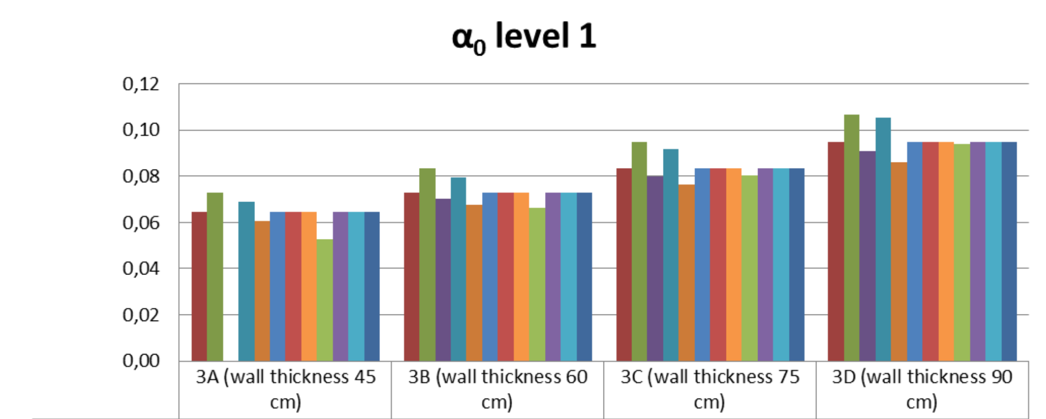
Comparing values in the same micro-typology is helpful to study the parameters influence but does not take into account the influence of the wall thickness in the local mechanism behavior, so a new kind of comparison has been made. In the histograms below (Fig. 5.4.11, Fig. 5.4.12, Fig. 5.4.13) the value of α_0 of the parameters set of each micro-typology has been compared, referring to the same hinge level.

In general, it is possible to observe that the wall thickness affects more the values of α_0 when the hinge is collocated at level 0 or at level 2 than when it is collocated at level 1. For example, the value of α_0 in the reference case has a percentage increment from micro-typology 3A (wall thickness 45cm) to micro-typology 3D (wall thickness 90 cm) of 251% at level 0, 47% at level 1 and 106% at level 2. The level 0 is the most sensible to thickness variations and the only negative values of α_0 appear at this level: for a wall thickness of 45 and 60cm, corresponding to parameter 7 (barrel vault) and parameter 11 (rise of 1.5m).



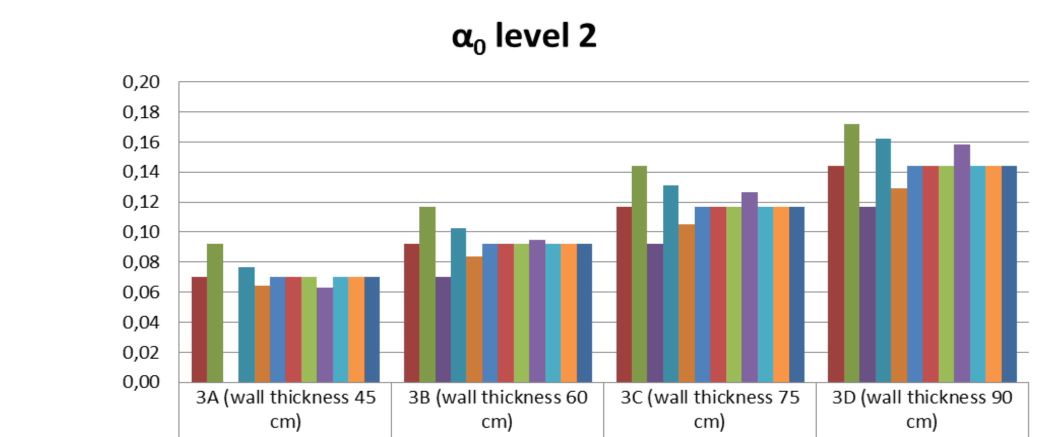
	3A (wall thickness 45 cm)	3B (wall thickness 60 cm)	3C (wall thickness 75 cm)	3D (wall thickness 90 cm)
REFERENCE	0,0167	0,0333	0,0467	0,0584
Parameter 1	0,0294	0,0428	0,0545	0,0651
Parameter 2	0,0000	0,0264	0,0416	0,0543
Parameter 3	0,0130	0,0314	0,0465	0,0597
Parameter 4	0,0197	0,0346	0,0465	0,0569
Parameter 5	0,0284	0,0435	0,0563	0,0679
Parameter 6	0,0079	0,0253	0,0390	0,0506
Parameter 7	-0,0343	-0,0081	0,0124	0,0287
Parameter 8	0,0634	0,0715	0,0794	0,0873
Parameter 9	0,0741	0,0781	0,0834	0,0895
Parameter 10	0,0366	0,0489	0,0596	0,0694
Parameter 11	-0,0390	-0,0104	0,0063	0,0291

Fig. 5.4.11: Variation of α_0 referring to level 0- Typology 3



	3A (wall thickness 45 cm)	3B (wall thickness 60 cm)	3C (wall thickness 75 cm)	3D (wall thickness 90 cm)
REFERENCE	0,0645	0,0730	0,0833	0,0947
Parameter 1	0,0730	0,0833	0,0947	0,1066
Parameter 2	0,0000	0,0703	0,0799	0,0909
Parameter 3	0,0691	0,0795	0,0919	0,1054
Parameter 4	0,0608	0,0677	0,0764	0,0860
Parameter 5	0,0645	0,0730	0,0833	0,0947
Parameter 6	0,0645	0,0730	0,0833	0,0947
Parameter 7	0,0645	0,0730	0,0833	0,0947
Parameter 8	0,0526	0,0664	0,0802	0,0940
Parameter 9	0,0645	0,0730	0,0833	0,0947
Parameter 10	0,0645	0,0730	0,0833	0,0947
Parameter 11	0,0645	0,0730	0,0833	0,0947

Fig. 5.4.12: Variation of α_0 referring to level 1- Typology 3



	3A (wall thickness 45 cm)	3B (wall thickness 60 cm)	3C (wall thickness 75 cm)	3D (wall thickness 90 cm)
REFERENCE	0,0699	0,0921	0,1171	0,1439
Parameter 1	0,0921	0,1171	0,1439	0,1718
Parameter 2	0,0000	0,0699	0,0921	0,1171
Parameter 3	0,0768	0,1026	0,1314	0,1621
Parameter 4	0,0642	0,0835	0,1055	0,1290
Parameter 5	0,0699	0,0921	0,1171	0,1439
Parameter 6	0,0699	0,0921	0,1171	0,1439
Parameter 7	0,0699	0,0921	0,1171	0,1439
Parameter 8	0,0633	0,0950	0,1267	0,1583
Parameter 9	0,0699	0,0921	0,1171	0,1439
Parameter 10	0,0699	0,0921	0,1171	0,1439
Parameter 11	0,0699	0,0921	0,1171	0,1439

Fig. 5.4.13: Variation of α_0 referring to level 2- Typology 3

After the analysis of the parameters values it is possible to define which ones have more influence in the building behavior and which ones can be overlooked. For example, the influence area was initially considered a varying parameter but due to its small influence in the mechanism behavior it has been overlooked and considered as a constant. Other parameters, on the opposite, prove to be very influential on the mechanism. For example, the parameters regarding the characteristics of the horizontal structures, such as parameters 7, 8, 9, 10 and 11, where introduced initially to consider the uncertainty that characterizes the in situ survey. It is indeed not always possible to define the horizontal structure of all the building, and these parameters considers the most probable options that diverge from the typological reference. The presence of a cross vault with a rise of 0.5m instead of 1m (parameter 11) can cause the collapse of the wall, such as the presence of a light horizontal structure instead of a cross vault (parameter 9) can strongly improve the seismic behavior of the local mechanism.

5.4.4 Capacity curves[13]

The capacity curve of the equivalent oscillator is defined through the relation between the acceleration a^* and the displacement d^*. As exposed in chapter 5.2.2, the spectral acceleration of the equivalent oscillator a_0^* is evaluated with the equation 5.4 during the non-linear kinematic analysis. The equivalent spectral displacement d_0^*, corresponding to $d_{k,0}$, is given by the following expression:

$$d_0^* = d_{k0} \cdot \frac{\sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}^2}{\delta_{x,k} \cdot \sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}} \tag{5.23}$$

while the displacement of the control point (the barycenter of the seismic masses) $d_{k,0}$ is evaluated as follow:

$$d_{k0} = h_{bar} \cdot sen\theta_{k0} \tag{5.24}$$

where θ_{k0} is the finite rotation which annuls the stabilizing moment Ms.

[13] *§C8A.4.2.2 Valutazione della curva di capacità* (Circolare esplicativa NTC 2008, pp. 412-414)

During the non-linear analysis the secant period *Ts*, which intersects the capacity curve and defines the displacement demand, is evaluated as follow in equation 5.25, in relation of d_s* and a_s*.

$$Ts = 2\pi \cdot \sqrt{\frac{d_S^*}{a_S^*}} \tag{5.25}$$

where:

$$d_S^* = 0.4 d_u^* \tag{5.26}$$

$$a_S^* = a_0^* \cdot \left(\frac{1 - d_S^*}{d_0^*} \right) \tag{5.27}$$

$$d_u^* = 0.4 d_{k0}$$

The values of a_0*, d_0*, a_s*, d_s* and d_u* are calculated for each parameter and reported in the following tables (Tab.5.4.6, Tab.5.4.7, Tab.5.4.8, Tab.5.4.9). The same values have been calculated for each analyzed typologies.

PARAMETER	HINGE LEVEL	a_0*	d_0*	a_s*	d_s*	d_u*
R	0	0,1644	0,1481	0,1381	0,0237	0,0592
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
1	0	0,2767	0,2608	0,2324	0,0417	0,1043
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
2	0	-	-	-	-	-
	1	-	-	-	-	-
	2	-	-	-	-	-
3	0	0,1267	0,1064	0,1064	0,0170	0,0426
	1	0,6115	0,3904	0,5137	0,0625	0,1562
	2	0,6254	0,2335	0,5253	0,0374	0,0934
4	0	0,1950	0,1876	0,1638	0,0300	0,0750
	1	0,5419	0,4275	0,4552	0,0684	0,1710
	2	0,5246	0,2390	0,4406	0,0382	0,0956
5	0	0,2845	0,2343	0,2390	0,0375	0,0937

	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
6	0	0,0764	0,0746	0,0642	0,0119	0,0298
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
7	0	-0,3433	-0,2934	-0,2884	-0,0469	-0,1174
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
8	0	0,6344	0,4995	0,5329	0,0799	0,1998
	1	0,4779	0,2756	0,4014	0,0441	0,1102
	2	0,4602	0,1327	0,3866	0,0212	0,0531
9	0	0,7112	0,6685	0,5974	0,1070	0,2674
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
10	0	0,3656	0,3233	0,3071	0,0517	0,1293
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947
11	0	-0,3781	-0,3477	-0,3176	-0,0556	-0,1391
	1	0,5735	0,4096	0,4818	0,0655	0,1638
	2	0,5705	0,2367	0,4792	0,0379	0,0947

Tab. 5.4.6: Capacity curve values – Micro-typology 3A

PARAMETER	HINGE LEVEL	a_0*	d_0*	a_s*	d_s*	d_u*
R	0	0,3238	0,2895	0,2720	0,0463	0,1158
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
1	0	0,4034	0,3745	0,3388	0,0599	0,1498
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
2	0	0,2621	0,2228	0,2202	0,0356	0,0891
	1	0,6517	0,4272	0,5474	0,0683	0,1709
	2	0,5705	0,2367	0,4792	0,0379	0,0947
3	0	0,3036	0,2527	0,2551	0,0404	0,1011
	1	0,7120	0,4331	0,5981	0,0693	0,1733
	2	0,8385	0,2896	0,7044	0,0463	0,1159
4	0	0,3382	0,3236	0,2841	0,0518	0,1294
	1	0,6088	0,4594	0,5114	0,0735	0,1838
	2	0,6814	0,2883	0,5723	0,0461	0,1153
5	0	0,4297	0,3517	0,3609	0,0563	0,1407
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157

6	0	0,2434	0,2358	0,2045	0,0377	0,0943
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
7	0	-0,0803	-0,0692	-0,0674	-0,0111	-0,0277
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
8	0	0,7024	0,5699	0,5900	0,0912	0,2280
	1	0,6033	0,3474	0,5068	0,0556	0,1390
	2	0,6903	0,1986	0,5798	0,0318	0,0794
9	0	0,7472	0,6915	0,6276	0,1106	0,2766
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
10	0	0,4823	0,4249	0,4051	0,0680	0,1700
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157
11	0	-0,1004	-0,0916	-0,0843	-0,0146	-0,0366
	1	0,6553	0,4467	0,5504	0,0715	0,1787
	2	0,7524	0,2893	0,6320	0,0463	0,1157

Tab. 5.4.7: Capacity curve values – Micro-typology 3B

PARAMETER	HINGE LEVEL	a_0*	d_0*	a_s*	d_s*	d_u*
R	0	0,4516	0,4017	0,3794	0,0643	0,1607
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
1	0	0,5136	0,4716	0,4315	0,0755	0,1887
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
2	0	0,4090	0,3491	0,3435	0,0559	0,1396
	1	0,7397	0,4725	0,6213	0,0756	0,1890
	2	0,7524	0,2893	0,6320	0,0463	0,1157
3	0	0,4474	0,3697	0,3758	0,0592	0,1479
	1	0,8275	0,4871	0,6951	0,0779	0,1949
	2	1,0716	0,3505	0,9001	0,0561	0,1402
4	0	0,4519	0,4307	0,3796	0,0689	0,1723
	1	0,6899	0,5057	0,5795	0,0809	0,2023
	2	0,8557	0,3444	0,7188	0,0551	0,1378
5	0	0,5517	0,4495	0,4634	0,0719	0,1798
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
6	0	0,3733	0,3596	0,3136	0,0575	0,1438
	1	0,7516	0,4968	0,6313	0,0795	0,1987

	2	0,9527	0,3477	0,8003	0,0556	0,1391
7	0	0,1226	0,1008	0,1029	0,0161	0,0403
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
8	0	0,7727	0,6366	0,6491	0,1018	0,2546
	1	0,7287	0,4190	0,6121	0,0670	0,1676
	2	0,9204	0,2639	0,7731	0,0422	0,1055
9	0	0,7964	0,7292	0,6690	0,1167	0,2917
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
10	0	0,5827	0,5119	0,4895	0,0819	0,2047
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391
11	0	0,0586	0,0549	0,0492	0,0088	0,0220
	1	0,7516	0,4968	0,6313	0,0795	0,1987
	2	0,9527	0,3477	0,8003	0,0556	0,1391

Tab. 5.4.8: Capacity curve values – Micro-typology 3C

PARAMETER	HINGE LEVEL	a_0*	d_0*	a_s*	d_s*	d_u*
R	0	0,5626	0,4982	0,4726	0,0797	0,1993
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
1	0	0,6140	0,5591	0,5158	0,0895	0,2236
	1	0,9662	0,6139	0,8116	0,0982	0,2455
	2	1,3803	0,4699	1,1595	0,0752	0,1880
2	0	0,5306	0,4540	0,4457	0,0726	0,1816
	1	0,8402	0,5269	0,7058	0,0843	0,2108
	2	0,9527	0,3477	0,8003	0,0556	0,1391
3	0	0,5728	0,4708	0,4812	0,0753	0,1883
	1	0,9520	0,5470	0,7996	0,0875	0,2188
	2	1,3160	0,4129	1,1054	0,0661	0,1652
4	0	0,5502	0,5226	0,4621	0,0836	0,2090
	1	0,7791	0,5592	0,6544	0,0895	0,2237
	2	1,0397	0,4034	0,8733	0,0645	0,1614
5	0	0,6614	0,5369	0,5556	0,0859	0,2148
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
6	0	0,4833	0,4633	0,4059	0,0741	0,1853
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
7	0	0,2799	0,2410	0,2351	0,0386	0,0964

	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
8	0	0,8441	0,7015	0,7090	0,1122	0,2806
	1	0,8542	0,4902	0,7175	0,0784	0,1961
	2	1,1505	0,3284	0,9664	0,0525	0,1314
9	0	0,8529	0,7746	0,7165	0,1239	0,3098
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
10	0	0,6746	0,5911	0,5667	0,0946	0,2364
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634
11	0	0,2788	0,2498	0,2342	0,0400	0,0999
	1	0,8563	0,5535	0,7193	0,0886	0,2214
	2	1,1635	0,4085	0,9773	0,0654	0,1634

Tab. 5.4.9: Capacity curve values – Micro-typology 3D

A first comparison is made between the curves of the same level on the same micro-typology. As example, the capacity curves of the micro-typology 3A for each level are shown in Fig. 5.4.14, Fig. 5.4.15 and Fig. 5.4.16.

An higher position of the capacity curve, corresponding to higher values of a_0*and d_0*, defines a lower vulnerability to the local mechanism. At level 0, the curves are almost all parallel to each other and are arranged in three groups. The higher one corresponds to parameters 8 (free standing wall) and parameter 9 (absence of vaults at the ground floor) and, as already seen in chapter 5.4.3, these parameters correspond to the less vulnerable cases. The lower group includes the parameter 7 (barrel vault) and the parameter 11(vault rise at 0.5m) which, as seen in chapter 5.4.3, correspond to the most vulnerable cases and assume negative values. The middle group includes the reference case and the remaining parameters.

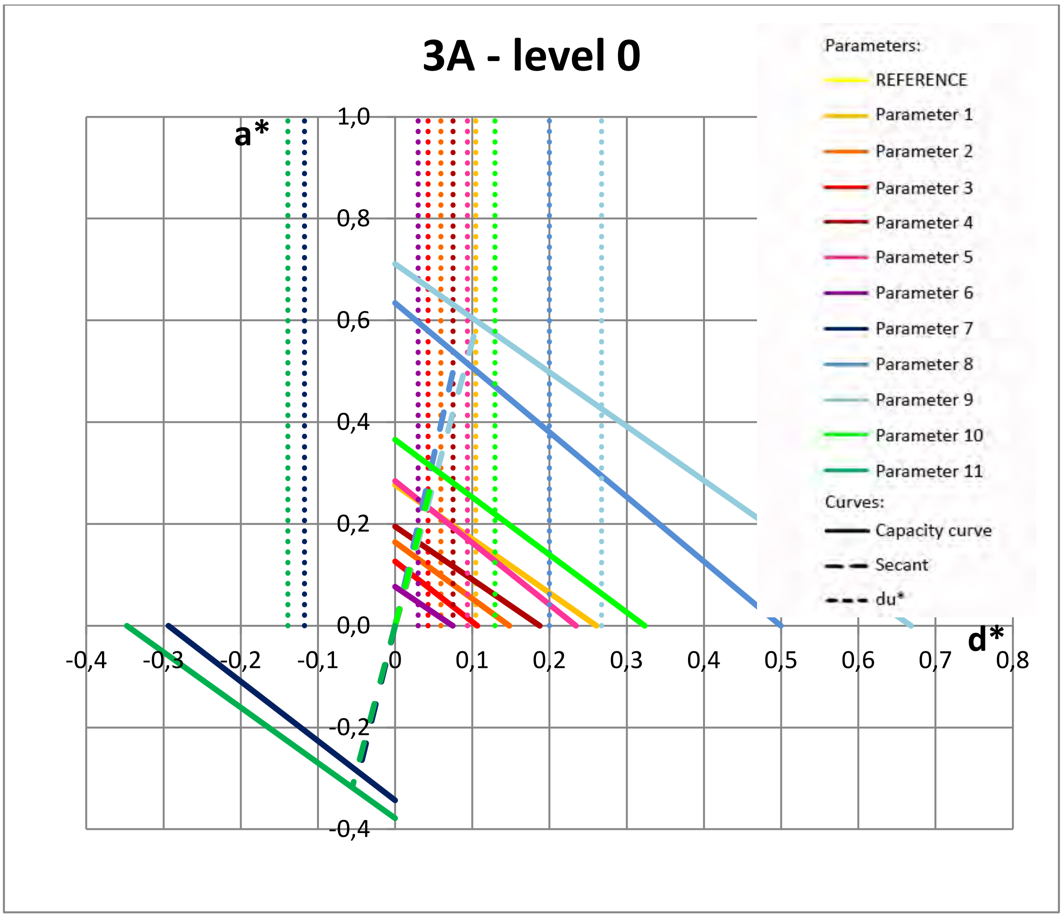


Fig.5.4.14: Capacity curve of micro-typology 3A-Level 0

At levels 1 and 2, the parameters 7, 9, 10 and 11 have the same value of the reference case, because they interest building characteristics of the ground floor that are no more considered if the hinge is formed in correspondence of upper floors. In these levels the trend of the curves is the same, except for the slope, that is higher at level 2. The higher curve corresponds to parameter 1 (constant thickness along the entire building height) while the lower to parameter 8 (free standing wall). The case of free standing wall is more vulnerable with the increase of the floor level and it is the only parameter that is not verified at level 2 (see Tab. 5.4.2).

The value of the secant is almost constant inside the same level and its slope increases with the level.

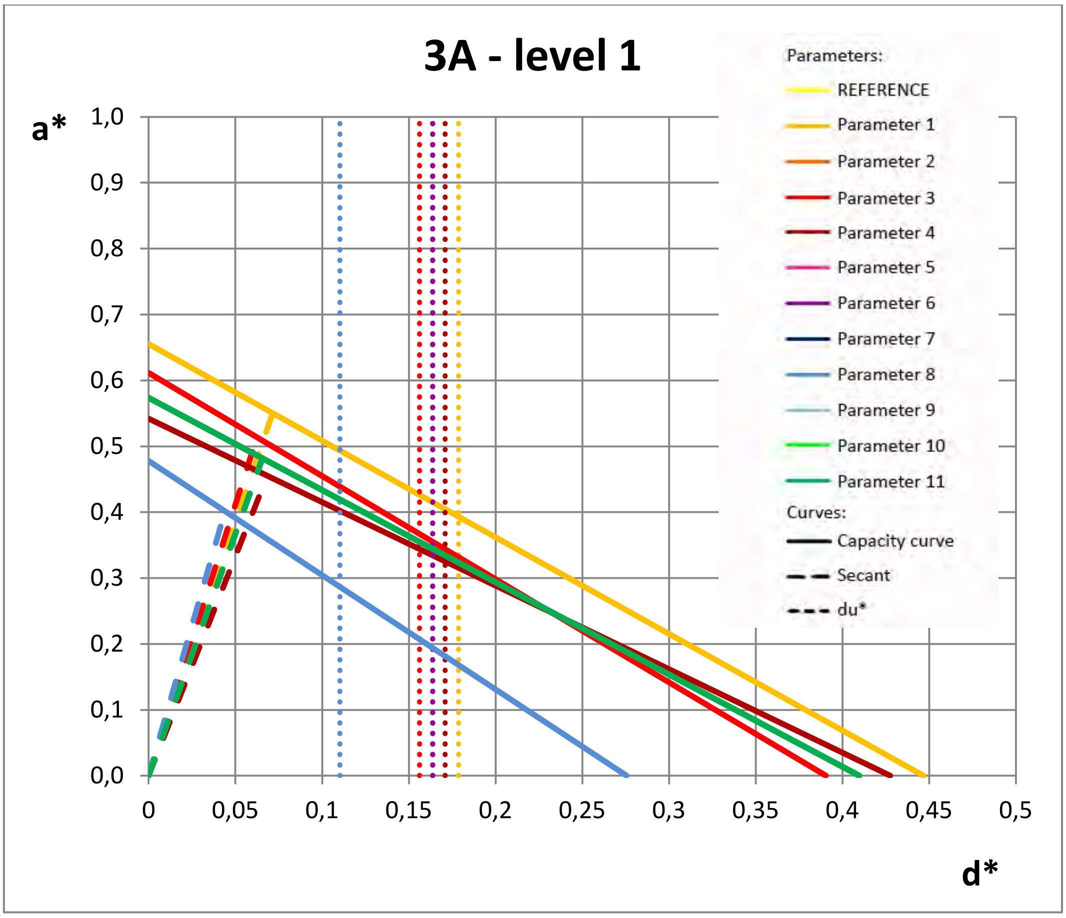


Fig.5.4.15: Capacity curve of micro-typology 3A-Level 1

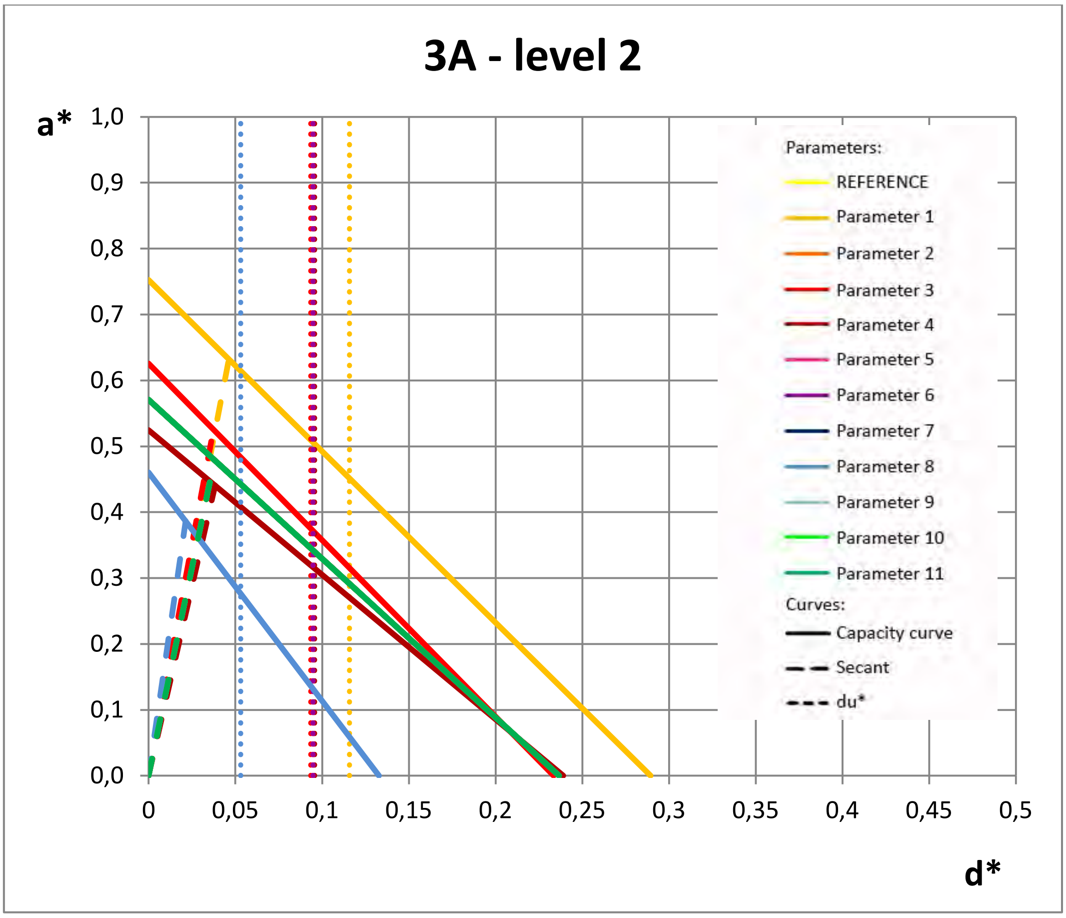


Fig.5.4.16: Capacity curve of micro-typology 3A-Level 2

The same considerations can be made for the others micro-typologies and their capacity curves can be found in annex D1.

A second comparison is made between the parameters of all the micro-typologies, unifying the curves of the same hinge level. The graphs below (Fig. 5.3.17, Fig. 5.3.18, Fig. 5.3.19) shows the capacity curves of all the parameters, distinguished by the curve color, corresponding of each micro-typology, distinguished by the line type.

In general it is possible to observe that the curves are almost parallel to each other in the same level, particularly at level 0, and their slope increases with the hinge level. The values of $a*$ become higher with the level height, while the values of d* become lower.

The global capacity curve at level 0 shows 4 negative curves: the same already seen in Figure 5.3.14 about parameters 7(barrel vaults) and parameter 11(vault rise at 0.5m) in micro-typology 3A, and the same parameters in micro-typology

3B. These two micro-typologies have the lower wall thickness(45 and 60cm). Other vulnerable curves, in the lower part of the first quadrant, are:

- Parameter 11 (vault rise at 0.5m) in micro-typology 3C (wall thickness of 75cm);
- Parameter 6 (maximum interstorey height for the ground floor) in micro-typology 3A (wall thickness of 45cm);
- Parameter 7 (barrel vaults) in micro-typology 3C (wall thickness of 75cm);
- Parameter 3 (minimum interstorey height for upper floors) in micro-typology 3A (wall thickness of 45cm);

The maximum interstorey height at the ground floor (parameter 6) causes the increment of the arm of the horizontal vault thrust and consequently it increases the overturning component of the mechanism. The minimum interstorey height for upper floors (parameter 3) reduces the stabilizing component of the wall weight at upper floors, promoting the activation of the local mechanism.

It is possible to observe that this group of curves includes the most vulnerable parameters of micro-typology 3C and medium parameters of micro-typology 3A. In particular, the curve corresponding to a barrel vaults that insist on a wall of 75cm of thickness (parameter 7 in 3C) and the one corresponding to the minimum interstorey height for upper floors in case of 45cm wall are almost coincident. This shows how the vulnerability is not related to a singular parameter or to a particular wall thickness, but it is defined by a combination of elements and aspects of the building.

The less vulnerable curves, in the upper part of the first quadrant, correspond to the parameters 8 (free standing wall) and 9 (absence of vaults at the ground floor) of all the micro-typologies.

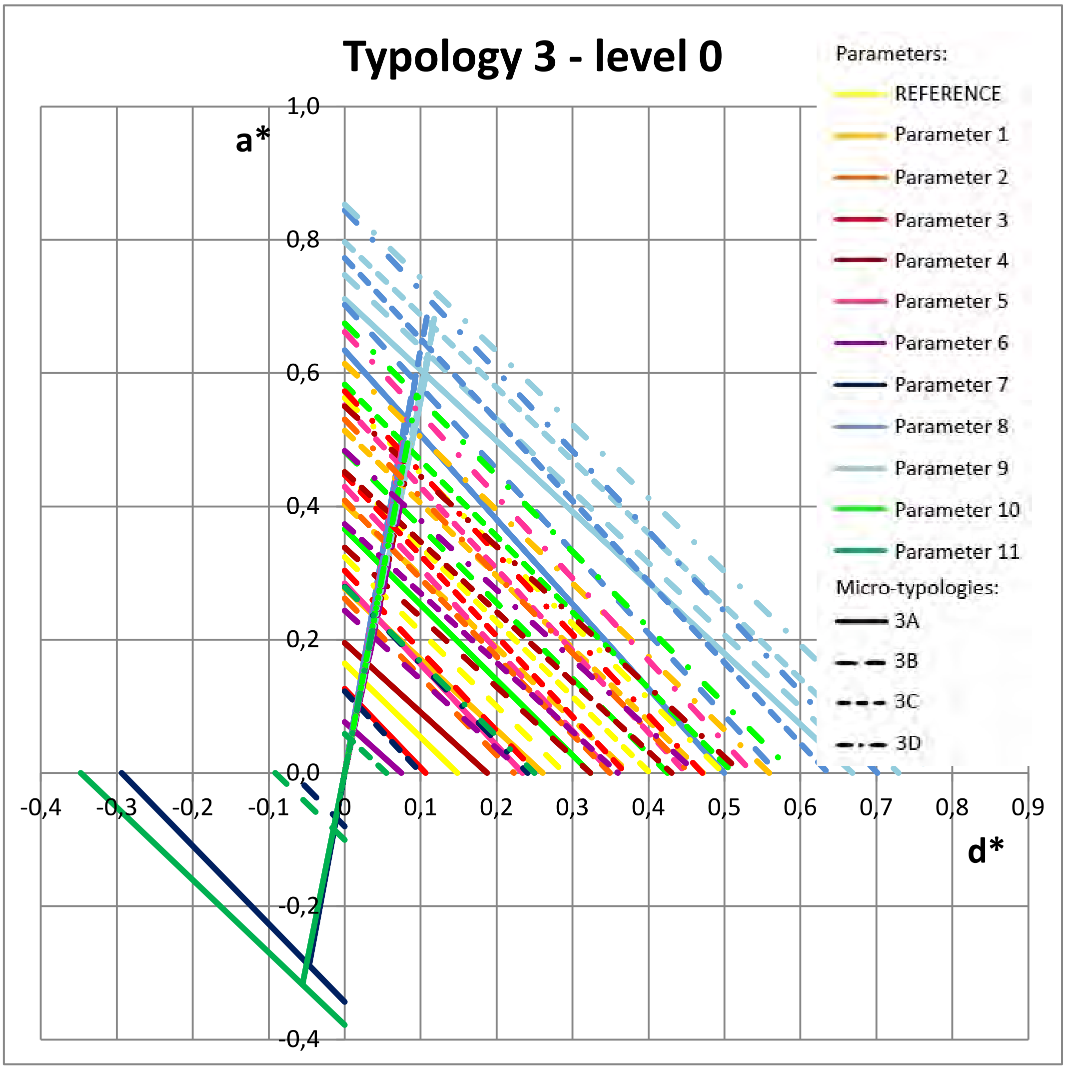


Fig.5.4.16: Capacity curve of typology 3-Level 0

As already seen, at levels 1 and 2 the parameters 7, 9, 10 and 11 have the same value of the reference case and they do not appear in the graphs. The trend of the curves is similar in the two levels: the lower curves correspond to parameter 8 (free standing wall) in micro-typologies 3A and the higher ones correspond to parameter 1 (constant wall thickness along the entire building height) in micro-typology 3D.

The capacity curves in relation with the level for each building typology can be found in annex D1.

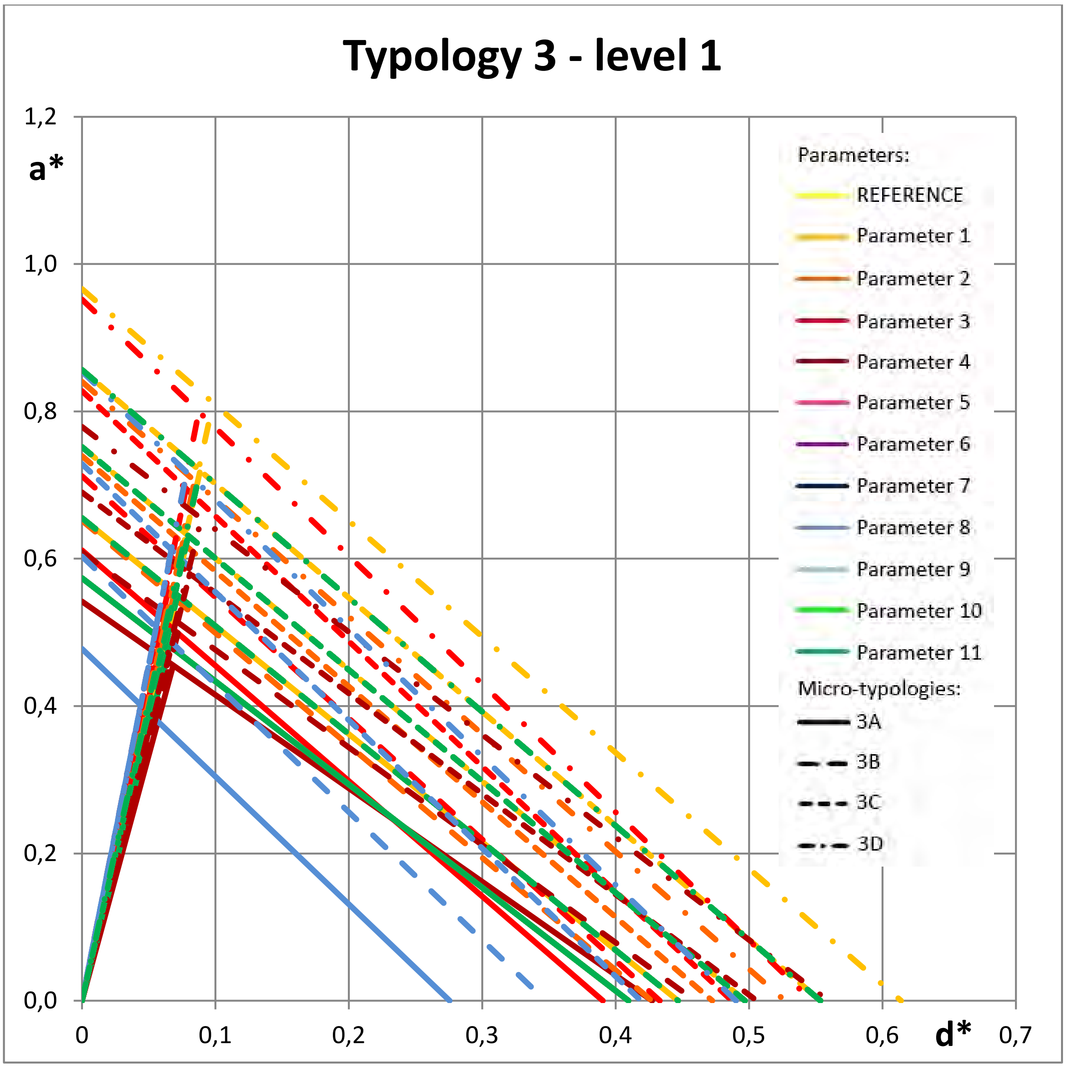


Fig.5.4.17: Capacity curve of typology 3-Level

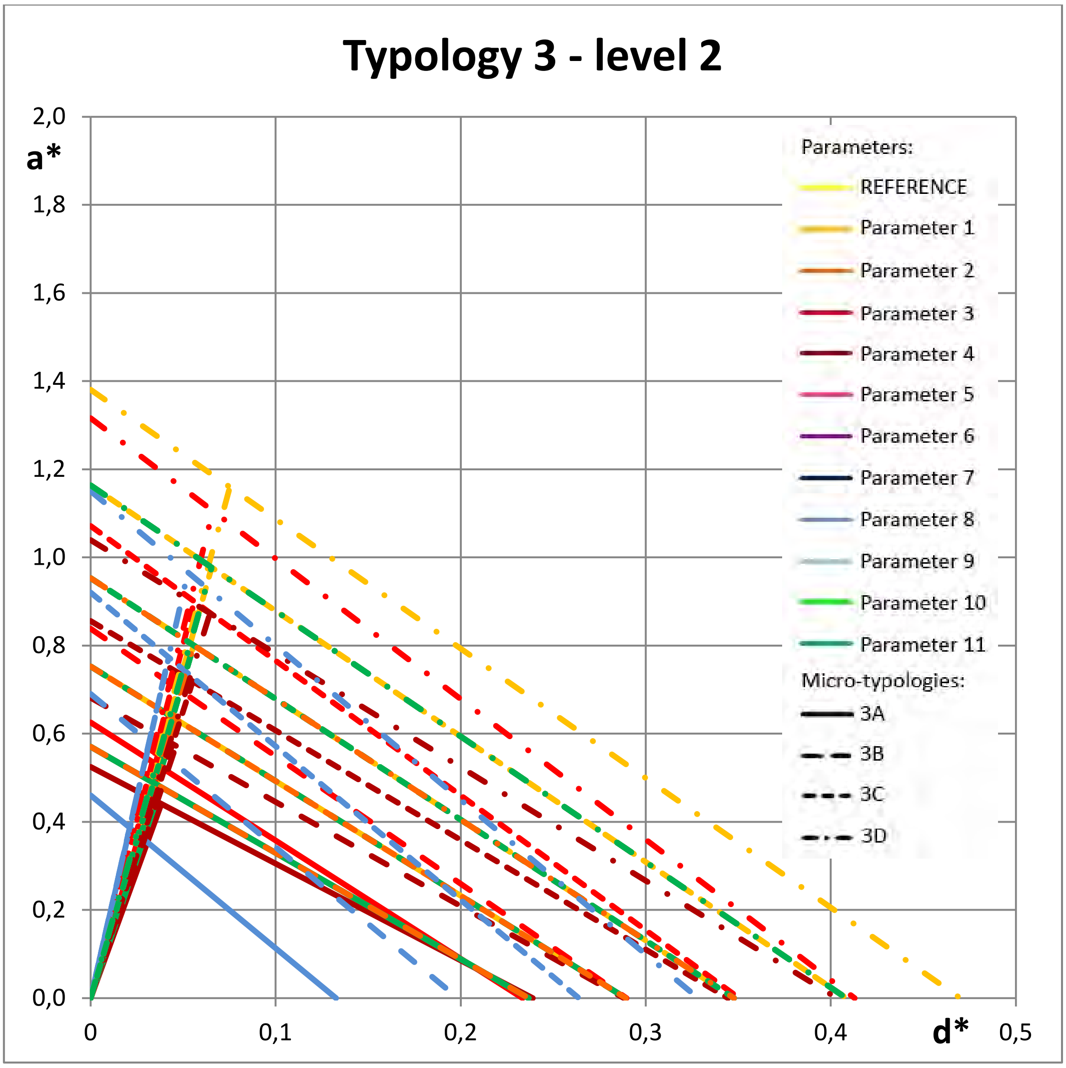


Fig.5.4.18: Capacity curve of typology 3-Level 2

5.4.5 Structural units comparison

The studies carried on in this thesis are based on data collected during a rapid survey of Timisoara and on the city center map that represents the ground floor plan around 1980. The information collected in situ are often incomplete, even about important building characteristics, and the city plan does not represent eventual interventions of the latest years. Farther, the typological analysis makes some approximations and simplifications of the structural characteristics that can influence the building behavior. For this reason it is important to verify that the adopted method does not deviate too much from the real case, comparing different cases with different levels of information. The same comparison has been made in chapter 4.3 with the application of Vulnus methodology for the real case and the survey case. In this chapter, the comparison is made between the values of α_0, a_0*,

d_0* and d_u* of the simple overturning mechanism of three different level of information:

- Real case: building dimensions and information about the structural typology and constructive details are obtained from plans and sections, usually in scale 1:50 or 1:100, of the building.[14]
- Survey case: information about structural typologies, wall thickness and interstorey high result from the in situ survey, while plan and openings dimension has been taken from the city plan of 1980 (see chapter 2.4.4)
- Typological case: building dimensions, structural typologies and plan dimensions are taken from the building typology in which the analyzed building is included. (see Tab. 2.4.2)

The analyzed units are US 88, US 123, US 124 and US 130. They all belong to typology 3, characterized by three stories, masonry vertical structures, deformable horizontal structures with brick vaults at the ground floor and timber roof. Despite that, they belong to three different micro-typologies: US 123 and US 124 belong to micro-typology 3B, characterized by a wall thickness of 60 cm, US 130 belongs to micro-typology 3C, characterized by a wall thickness of 75 cm, and US 88 belongs to micro-typology 3D, characterized by a wall thickness of 90 cm. All the units have an average typological interstorey high of 4,3 m for the ground floor and 4,2 m for upper floors.

A friction coefficient of 0.05 has been used, with a ground acceleration of a_g=0.2g. The main differences between the three cases are the wall thickness, the interstorey height and the horizontal structures typology. The analyzed wall is chosen considering the ones for which the simple overturning mechanism can be activated. The most vulnerable one is analyzed. Each structural unit is now analyzed.

US 88

The analyzed wall is underlined by a red shape in Fig. 5.4.19 and Fig. 5.4.20. It is collocated in the building façade and it is three stories high. In the real case the horizontal structure typology of the ground floor is recognizable as a cross vault

[14] The material has been provided by Arch. Bodgan Demetrescu

but its rise and springing height is not indicated. For this unit indeed there are no building sections and the vertical dimensions are taken from the survey data.

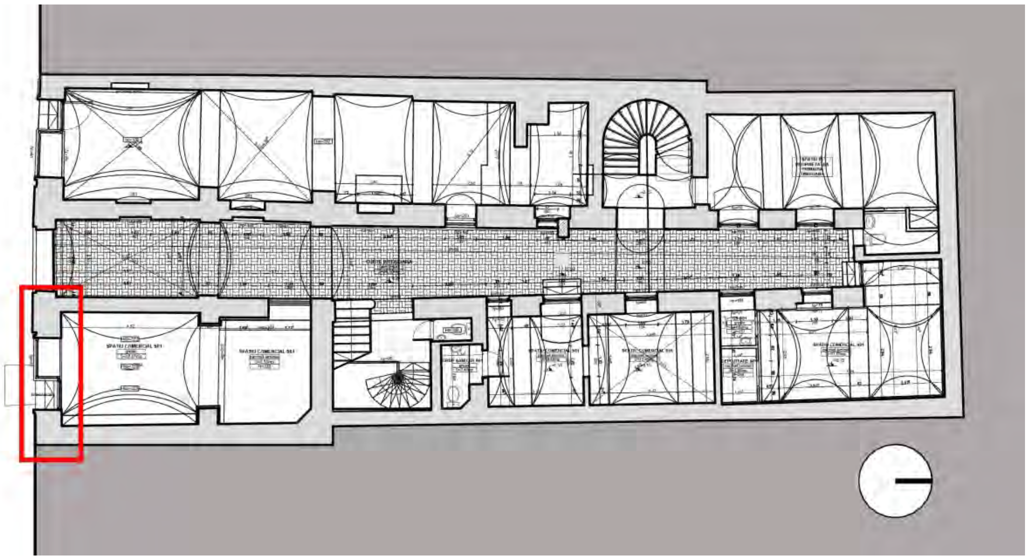


Fig.5.4.19: US 88 – Real case plan of the ground floor

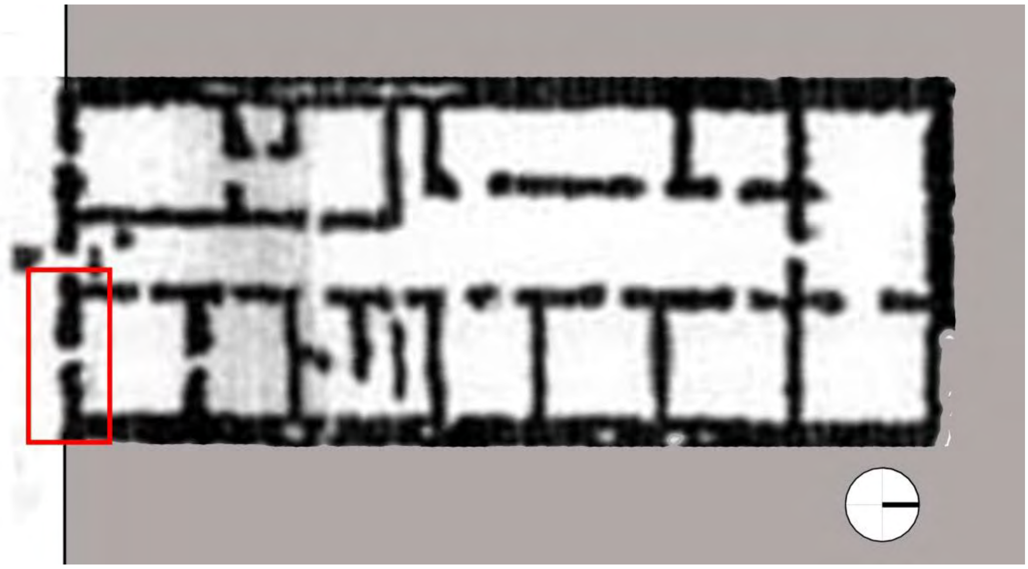


Fig.5.4.20: US 88 – Survey case plan the ground floor

A table to resume the cases characteristics has been made and it is shown in Tab. 5.4.10. It is possible to observe that:

- the wall thickness in the real case is different from the other two cases of some centimeters;
- The interstorey height at the ground floor is 50cm higher in the typological case because the typology interstorey height derives from the average value of all the analyzed building of the typology 3;

- The plan module, the area and the septs dimension are quite homogenous;
- The vault typology and its rise are the same in all the cases.

CASE	TYPOLOGY / US	STORIES NUMBER	HINGE LIVEL	WALL THICKNESS Ground floor [cm]	WALL THICKNESS Upper floors [cm]	INTERSTORY HEIGHT Ground floor [m]	INTERSTORY HEIGHT Upper floors [m]	PLAN MODULE [m]	AREA [m^2]	SEPT DIMENSION [m]	VAULT TYPOLOGY	VAULT RISE [m]
Real	US 88	3	0	86	76_73	3,8	4,2	4,21 x 5,06	5,3	4,21	Cross	1
		3	1	-	76_73	-	4,2	4,21 x 5,06	5,3	4,21	Cross	1
		3	2	-	73	-	4,2	4,21 x 5,06	5,3	4,21	Cross	1
Survey	US 88	3	0	90	75	3,8	4,2	4,34 x 4,05	4,3	4,34	Cross	1
		3	1	-	75	-	4,2	4,34 x 4,05	4,3	4,34	Cross	1
		3	2	-	75	-	4,2	4,34 x 4,05	4,3	4,34	Cross	1
Typological	3D	3	0	90	75	4,3	4,2	4,1 x 5,8	5,9	4,1	Cross	1
		3	1	-	75	-	4,2	4,1 x 5,8	5,9	4,1	Cross	1
		3	2	-	75	-	4,2	4,1 x 5,8	5,9	4,1	Cross	1

Tab. 5.4.10: Resume table of the characteristics of real, survey and typological cases- US 88

As made for the typological analysis, the mechanism has been verified with a linear kinematic analysis and a non-linear kinematic analysis for each case and the values of α_0, a_0*, d_0* and d_u* have been collected and compared. Tab. 5.4.11 shows that all the cases result not verified in the linear kinematic analysis but verified in the non-linear kinematic analysis.

Fig. 5.4.21 and Fig 5.4.22 shows the comparison of α_0 and d_u*values and it is possible to observe that the histograms underline very few differences between the three cases: maximum and minimum are constant and the values have a variation of 0.007 for α_0 and 0.03 for d_u*.

Thanks to the result comparison it can be stated that the building typology that includes the US 88 well represents the units behavior. The assumptions made during the typological analysis are so far validated.

CASE	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
Real	0	0,0571	0,5557	1,4715	0,3776	NO	0,1793	0,0560	3,2021	YES
	1	0,0945	0,8628	1,4715	0,5864	NO	0,2067	0,0560	3,6919	YES
	2	0,1476	1,1487	2,3756	0,4835	NO	0,1444	0,0765	1,8888	YES
Survey	0	0,0590	0,5783	1,4715	0,3930	NO	0,1846	0,0560	3,2961	YES
	1	0,0944	0,8586	1,4715	0,5835	NO	0,2054	0,0560	3,6683	YES
	2	0,1402	1,0716	2,3756	0,4511	NO	0,1384	0,0772	1,7919	YES
Typological	0	0,0584	0,5626	1,4715	0,3823	NO	0,1993	0,0560	3,5588	YES
	1	0,0947	0,8563	1,1417	0,7500	NO	0,2214	0,0560	3,9535	YES
	2	0,1439	1,1635	2,3528	0,4945	NO	0,1634	0,0819	1,9941	YES

Tab. 5.4.11: Verification of simple overturning mechanism – US 88

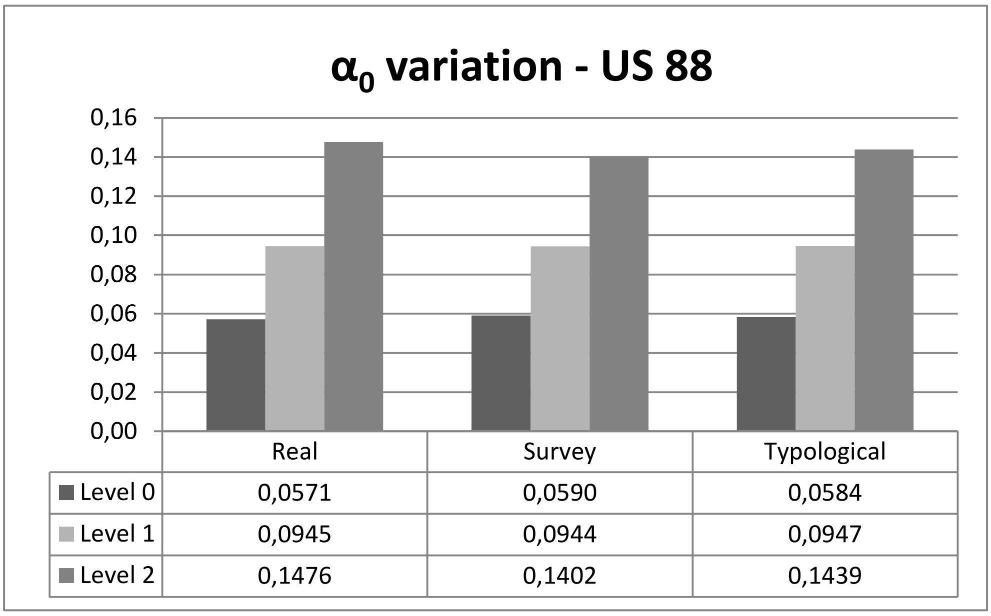
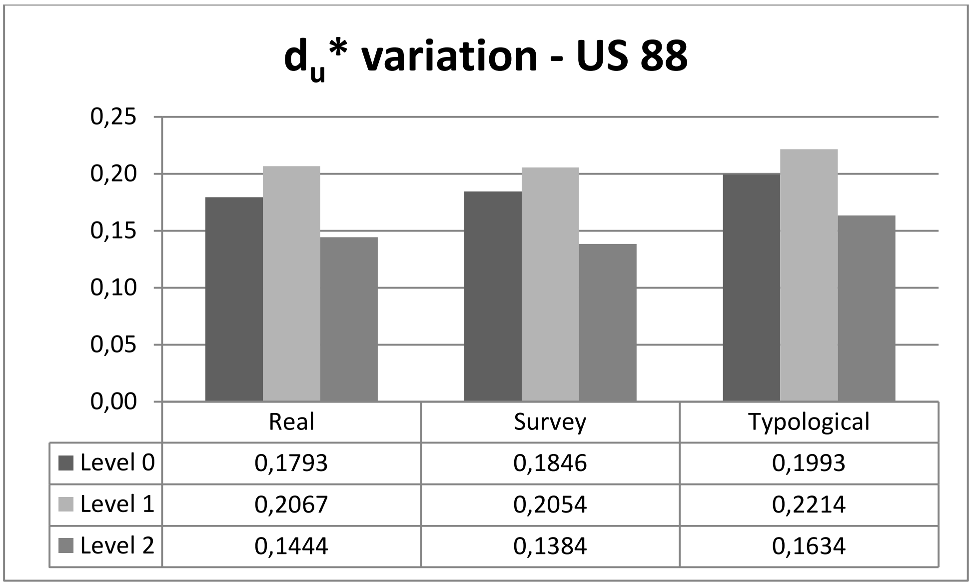


Fig.5.4.21:α_0 variation - US 88



	Real	Survey	Typological
Level 0	0,1793	0,1846	0,1993
Level 1	0,2067	0,2054	0,2214
Level 2	0,1444	0,1384	0,1634

Fig.5.4.22: d_u variation - US 88*

US 123

The analyzed sept is underlined by a red shape in Fig. 5.4.23 and Fig. 5.4.24. It is collocated in the internal courtyard and it is indicated by Vulnus as the most vulnerable sept to out of plane mechanisms. In the real case the sept is characterized by a thickness of 1.05m and by large openings over the full height. At the ground floor on a total length of 7.65m the openings occupy about 6m. In the survey case plan the sept appears to be thinner and with only one small opening.

The Table 5.4.12 resumes the characteristics of the three cases and it is possible to observe:

- The real case has a wall thickness of 1.05m at the ground floor that is considerably higher than the other two cases. The same observation can be made for upper floor walls thickness;
- The interstorey height is similar in the real case and in the survey case but it is higher in the typological case. The difference of 1m is due to the simplifications made during the typologies definition;
- The real case is characterized by a barrel vault with a rise of 0.5m that insists on the analyzed sept. The horizontal thrust of this type of vault is one of the highest of the entire analysis.

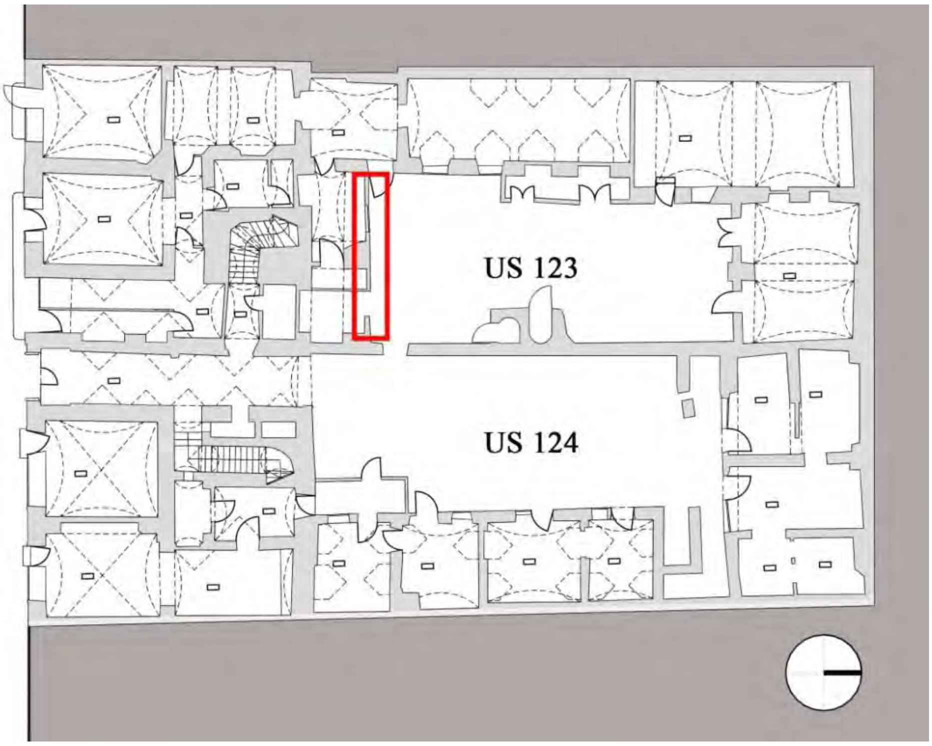


Fig.5.4.23: US 123– Real case plan the ground floor

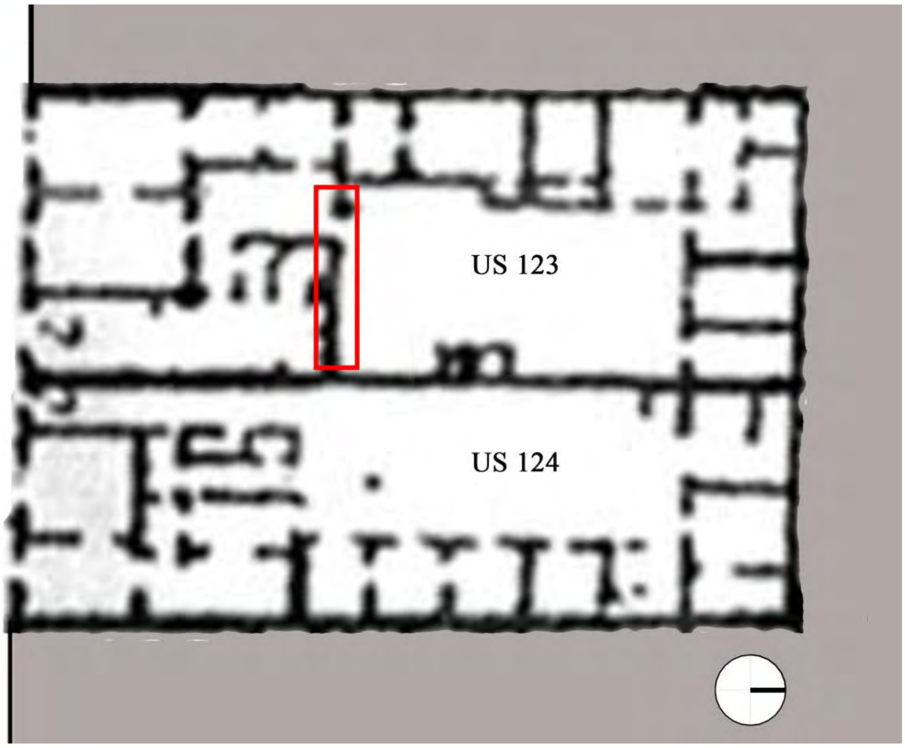


Fig.5.4.24: US 123– Survey case plan the ground floor

CASE	TYPOLOGY / US	STORIES NUMBER	HINGE LIVEL	WALL THICKNESS Ground floor [cm]	WALL THICKNESS Upper floors [cm]	INTERSTORY HEIGHT Ground floor [m]	INTERSTORY HEIGHT Upper floors [m]	PLAN MODULE [m]	AREA [m^2]	SEPT DIMENSION [m]	VAULT TYPOLOGY	VAULT RISE [m]
Real	US 123	3	0	105	88_67	3,11	3,64_2,65	1,89x7,65	7,25	7,65	Barrel	0,5
		3	1	-	88_67	-	3,64_2,65	1,89x7,65	7,25	7,65	Barrel	0,5
		3	2	-	88_67	-	3,64_2,65	1,89x7,65	7,25	7,65	Barrel	0,5
Survey	US 123	3	0	60	45	3,4	3,55	1,89x7,65	7,25	7,65	Cross	1
		3	1	-	45	-	3,55	1,89x7,65	7,25	7,65	Cross	1
		3	2	-	45	-	3,55	1,89x7,65	7,25	7,65	Cross	1
Typolo gical	3B	3	0	60	45	4,3	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	1	-	45	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	2	-	45	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1

Tab. 5.4.12: Resume table of the characteristics of real, survey and typological cases- US 123

The mechanism has been verified with a linear kinematic analysis and a non-linear kinematic analysis for each case and the values of α_0, a_0*, d_0* and d_u* have been collected and compared. Tab. 5.4.13 shows that all the cases result not verified in the linear kinematic analysis but almost all verified in the non-linear kinematic analysis, except for the real case with the hinge at the ground floor level. In the real case the horizontal thrust of the barrel vault brings to a negative value of α_0 and, consequently, to negative values of a_0*, d_0* and d_u*. In this situation there is probably a resistance device, which was not detected, that prevents the mechanism activation. This particularity is evident also in Fig. 5.4.25 and Fig.5.4.26 that underline the variations of α_0 and d_u*values. Except for level 0, α_0 is higher in the real case due to a greater wall thickness than the other two cases. The survey case and the typological case have similar values, even if the survey case has a lower α_0 at level 0 and higher values at levels 1 and 2. The d_u* comparison shows that level 1 has the highest value in all the cases, followed by level 2 and level 0. In the typological case the levels 0 and 2 have the same d_u* value.

Due to the difference of wall thickness and the presence of the pushing barrel vaults the results do not coincide as well as US 88 ones, but they still valid the typological analysis.

CASE	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
Real	0	-0,0539	-0,5297	1,4715	-0,3599	NO	-0,1255	0,0560	-2,2419	NO
	1	0,1336	1,2306	1,4715	0,8363	NO	0,2091	0,0560	3,7335	YES
	2	0,2184	1,7189	3,0222	0,5688	NO	0,1374	0,0560	2,4532	YES
Survey	0	0,0240	0,2413	1,4715	0,1640	NO	0,0639	0,0560	1,1406	YES
	1	0,0785	0,7133	1,4715	0,4848	NO	0,1472	0,0560	2,6278	YES
	2	0,1119	0,8766	2,6836	0,3267	NO	0,0948	0,0709	1,3375	YES
Typological	0	0,0333	0,3238	1,4715	0,2200	NO	0,1158	0,0560	2,0678	YES
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES

Tab. 5.4.13: Verification of simple overturning mechanism – US 123

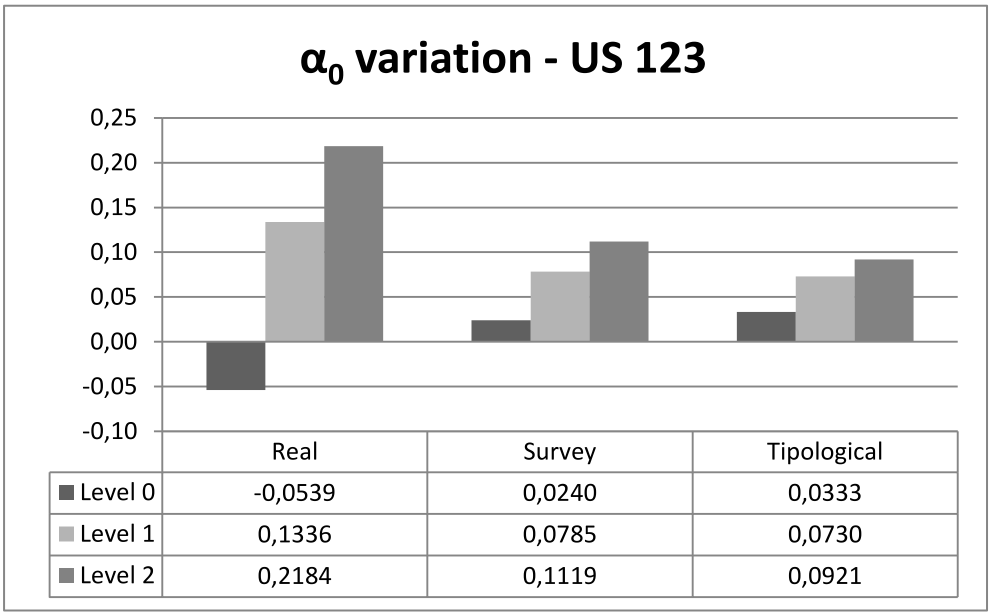


Fig.5.4.25:α_0 variation - US 123

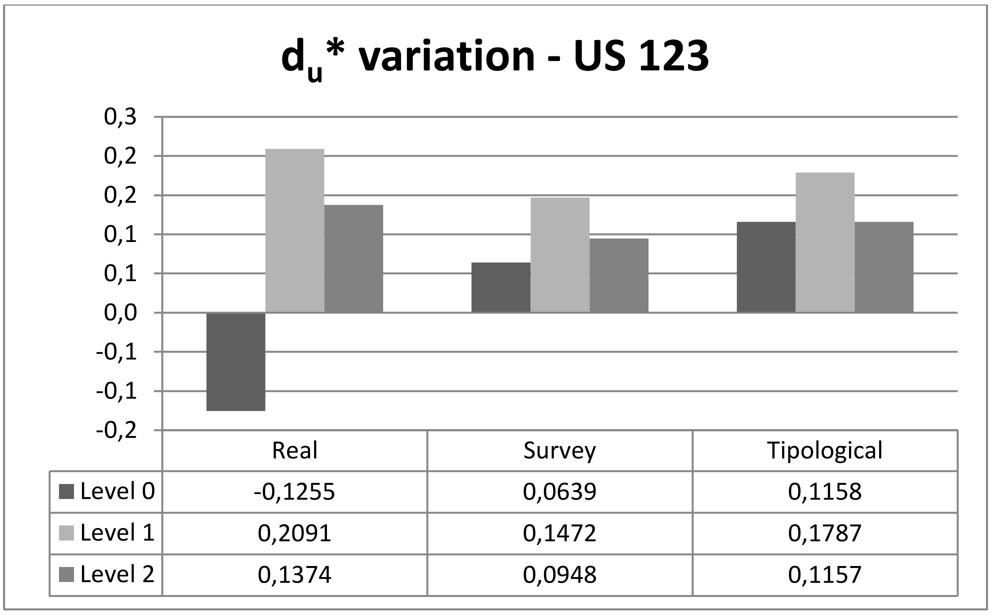


Fig.5.4.26:d_u variation - US 123*

US 124

The analyzed sept is underline by a red shape in Fig. 5.4.27 and Fig. 5.4.28. It is collocated on the internal courtyard and it is indicated by Vulnus as the most vulnerable sept to out of plane mechanisms. Both in the real case and in the survey case the sept is widely open. The Table 5.4.14 resumes the characteristics of the three cases and it is possible to observe that:

- The wall thickness is different for the three cases both at the ground floor and at the upper floors;
- The interstorey height at the ground floor is similar in the real case and in the survey case, while it is different in the three cases at upper floors;
- The area and the sept dimensions are different but comparable in terms of values and module shape;
- In the real case the analyzed sept at the ground floor is unstressed because the barrel vault of the room insists on walls perpendicular to the façade.

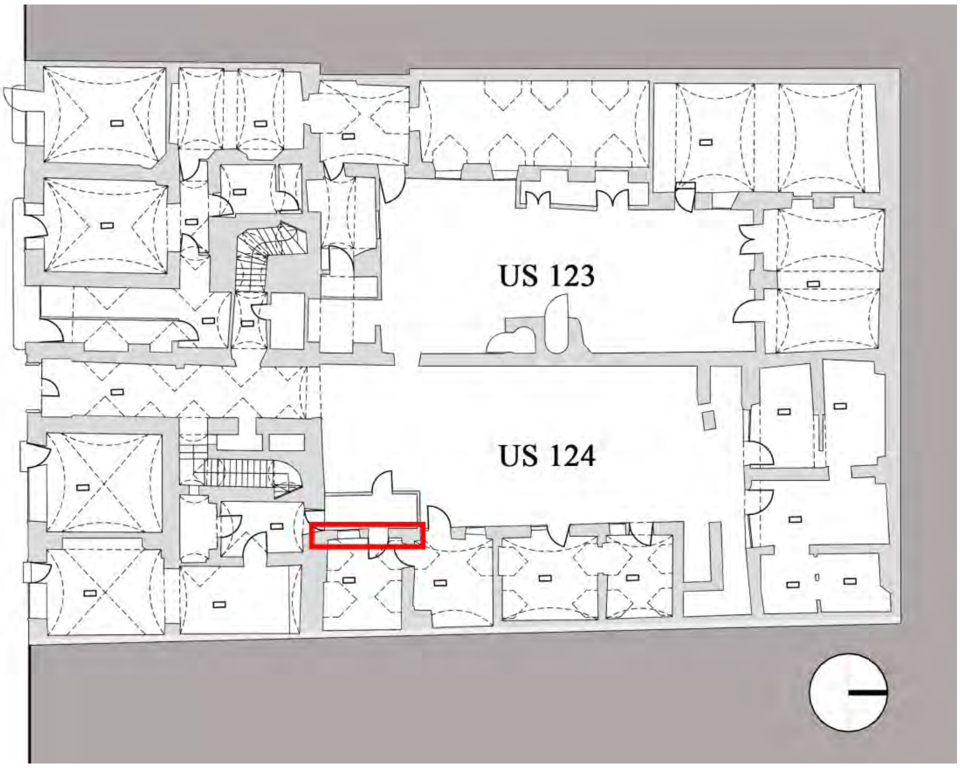


Fig.5.4.27: US 124– Real case plan the ground floor

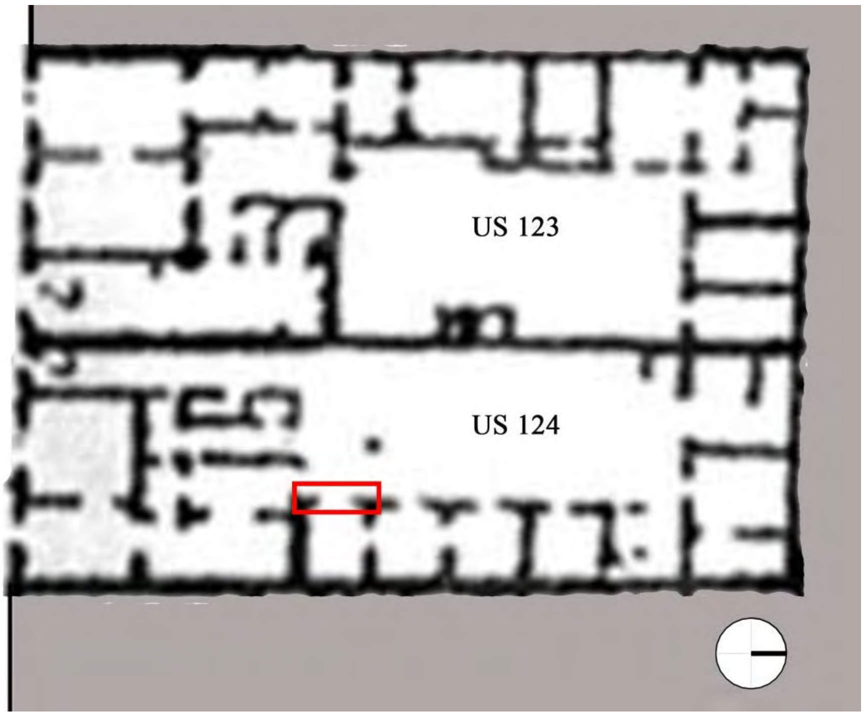


Fig.5.4.28: US 124 – Survey case plan the ground floor

CASE	TYPOLOGY / US	STORIES NUMBER	HINGE LIVEL	WALL THICKNESS Ground floor [cm]	WALL THICKNESS Upper floors [cm]	INTERSTORY HEIGHT Ground floor [m]	INTERSTORY HEIGHT Upper floors [m]	PLAN MODULE [m]	AREA [m^2]	SEPT DIMENSION [m]	VAULT TYPOLOGY	VAULT RISE [m]
Real	US 124	3	0	56	38_31	3,26	3,77_2,77	3,5x4,11	7,28	4,11	-	-
		3	1	-	38_31	-	3,77_2,77	3,5x4,11	7,28	4,11	-	-
		3	2	-	38_31	-	3,77_2,77	3,5x4,11	7,28	4,11	-	-
Survey	US 124	3	0	75	60	3,3	4	3,65x4,67	8,54	3,65	Cross	1
		3	1	-	60	-	4	3,65x4,67	8,54	3,65	Cross	1
		3	2	-	60	-	4	3,65x4,67	8,54	3,65	Cross	1
Typolo gical	3B	3	0	60	45	4,3	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	1	-	45	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	2	-	45	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1

Tab. 5.4.14: Resume table of the characteristics of real, survey and typological cases- US 124

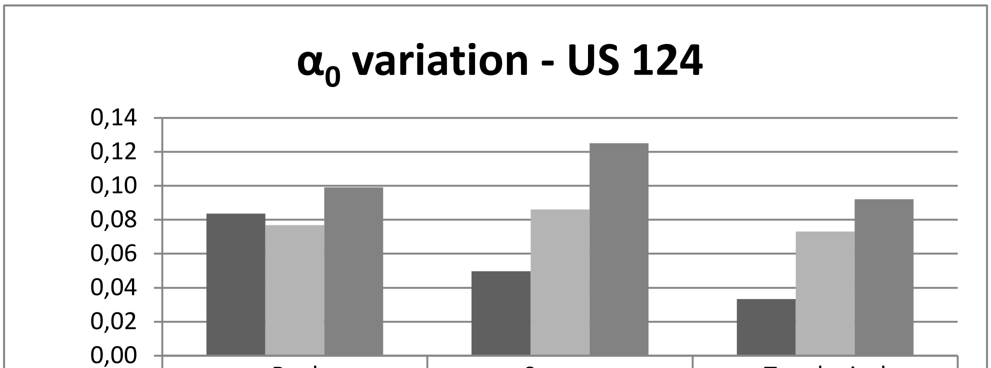
The mechanism has been verified with a linear kinematic analysis and a non-linear kinematic analysis for each case and the values of α_0, a_0*, d_0* and d_u* have been collected and compared. Tab. 5.4.15 shows that the cases result all not verified in the linear kinematic analysis but all verified in the non-linear kinematic analysis.

CASE	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
Real	0	0,0835	0,8230	1,4715	0,5593	NO	0,2241	0,0560	4,0027	YES
	1	0,0768	0,6961	1,4715	0,4731	NO	0,1436	0,0560	2,5637	YES
	2	0,0991	0,8101	3,0628	0,2645	NO	0,0848	0,0749	1,1321	YES
Survey	0	0,0497	0,4929	1,4715	0,3350	NO	0,1498	0,0560	2,6750	YES
	1	0,0861	0,7804	1,4715	0,5303	NO	0,1906	0,0560	3,4031	YES
	2	0,1250	1,0081	2,4789	0,4067	NO	0,1333	0,0757	1,7618	YES
Typologic al	0	0,0333	0,3238	1,4715	0,2200	NO	0,1158	0,0560	2,0678	YES
	1	0,0730	0,6553	1,1902	0,5505	NO	0,1787	0,0560	3,1910	YES
	2	0,0921	0,7524	2,3528	0,3198	NO	0,1157	0,0848	1,3650	YES

Tab. 5.4.15: Verification of simple overturning mechanism – US 124

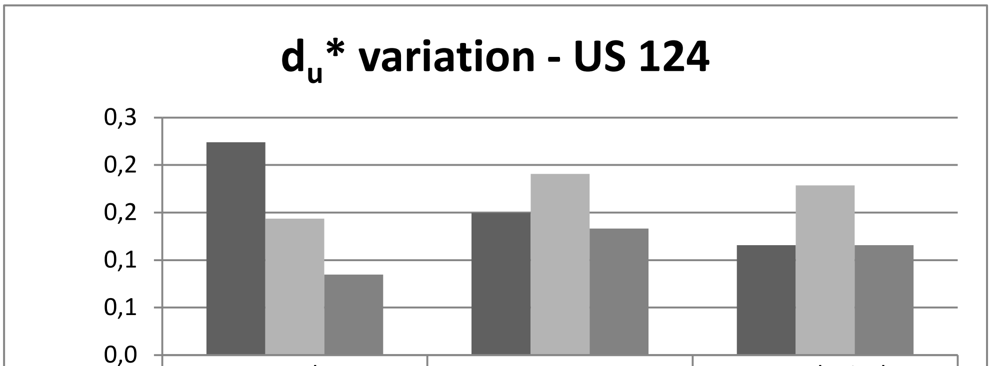
The histograms below (Fig.5.4.29 and Fig.5.4.30) show the comparison of α_0 and d_u* variations. Both values are higher in correspondence of level 0 in the real case, because the absence of the horizontal thrust disfavors the activation of the overturning mechanism. The values of the survey case are the highest, except for the case above, both for α_0 and d_u* due to a greater wall thickness combined with a lower interstorey height at the ground floor.

The results comparison is quite homogeneous, except for the particular case of the free standing wall at the ground floor. In general the building typology represents well the unit behavior.



	Real	Survey	Typological
Level 0	0,0835	0,0497	0,0333
Level 1	0,0768	0,0861	0,0730
Level 2	0,0991	0,1250	0,0921

Fig.5.4.29: α_0 *variation - US 124*



	Real	Survey	Typological
Level 0	0,2241	0,1498	0,1158
Level 1	0,1436	0,1906	0,1787
Level 2	0,0848	0,1333	0,1157

Fig.5.4.30: d_u* *variation - US 124*

US 130

The analyzed sept is underlined by a red shape in Fig. 5.4.31 and Fig. 5.4.32. It occupies all the south façade of the structural unit, between the corner and the adjacent building. It is indicates by Vulnus as the most vulnerable wall to out of plane mechanism. In the real case the horizontal structure is recognizable as a barrel vault which insists on the analyzed wall.

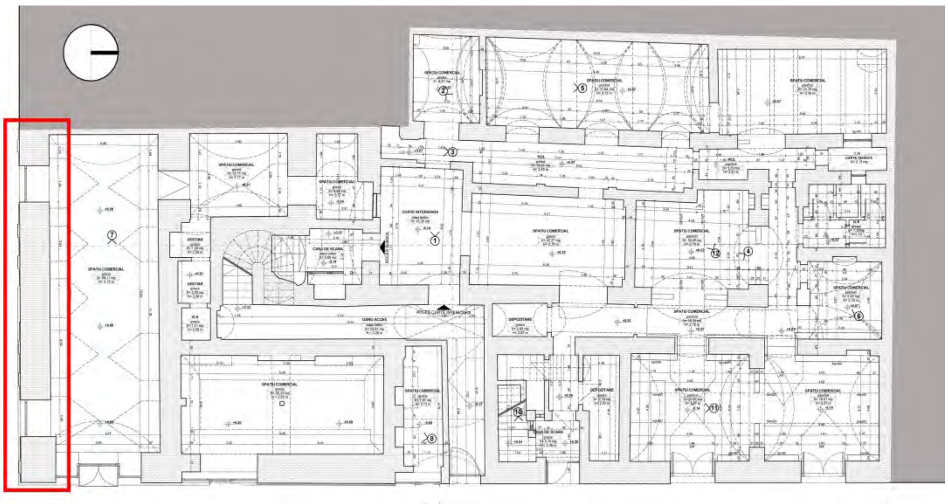


Fig.5.4.31: US 130– Real case plan

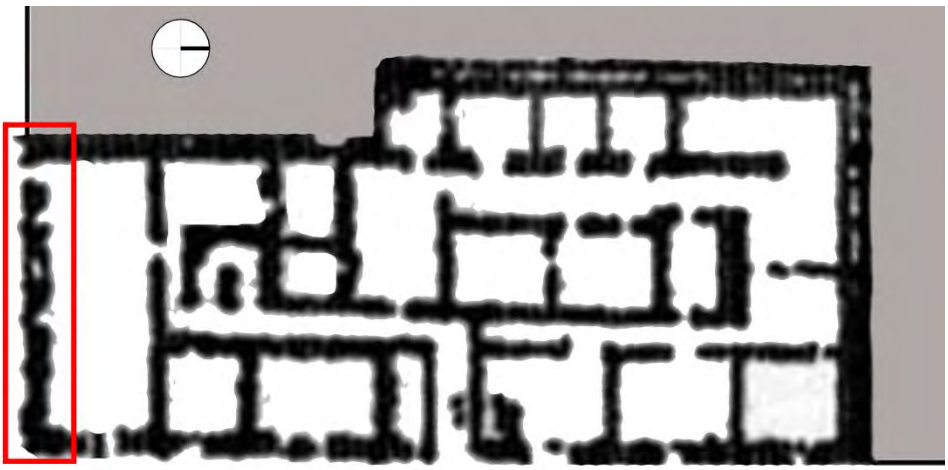


Fig.5.4.32: US 130 – Survey case plan

A table to resume the cases characteristics has been made and it is shown in Tab. 5.4.16. It is possible to observe that:

- The wall thickness of the ground floor is considerably higher in the real case, while it is homogeneous in the upper floors;

- The interstorey height is considerably higher in the typological case, while is quite homogenous in the upper floors;
- The area and the sept dimensions are considerably lower in the typological case, due to the unusual great dimension of the analyzed room.

CASE	TYPOLOGY / US	STORIES NUMBER	HINGE LIVEL	WALL THICKNESS Ground floor [cm]	WALL THICKNESS Upper floors [cm]	INTERSTORY HEIGHT Ground floor [m]	INTERSTORY HEIGHT Upper floors [m]	PLAN MODULE [m]	AREA [m^2]	SEPT DIMENSION [m]	VAULT TYPOLOGY	VAULT RISE [m]
Real	US 130	3	0	120	65_50	3,4	3,8_3,37	5,49 x 13,57	37,3	13,57	Barrel	1,5
		3	1	-	65_50	-	3,8_3,37	5,49 x 13,57	37,3	13,57	Barrel	1,5
		3	2	-	50	-	3,37	5,49 x 13,57	37,3	13,57	Barrel	1,5
Survey	US 130	3	0	75	60	3,6	4	5,49 x 13,57	37,3	13,57	Cross	1
		3	1	-	60	-	4	5,49 x 13,57	37,3	13,57	Cross	1
		3	2	-	60	-	4	5,49 x 13,57	37,3	13,57	Cross	1
Typolo gical	3C	3	0	75	60	4,3	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	1	-	60	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1
		3	2	-	60	-	4,2	4,1 x 5,8	11,8	4,1	Cross	1

Tab. 5.4.16: Resume table of the characteristics of real, survey and typological cases- US 130

As made for the typological analysis, the mechanism has been verified with a linear kinematic analysis and a non-linear kinematic analysis for each case and the values of α_0, a_0*, d_0* and d_u* have been collected and compared. Tab. 5.4.17 shows that all the cases result not verified in the linear kinematic analysis but all verified in the non-linear kinematic analysis.

As can be seen in the histograms of α_0 and d_u* variations (Fig. 5.4.33, Fig. 5.4.34) the three cases have very small differences, despite the different aspects underlined above. In general it is possible to say that the typological subdivision well represents the structural unit behavior.

CASE	HINGE LEVEL	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
			a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
Real	0	0,0619	0,6574	1,4715	0,4468	NO	0,1631	0,0560	2,9122	YES
	1	0,0945	0,8714	1,4715	0,5922	NO	0,1900	0,0560	3,3922	YES
	2	0,1221	0,9974	2,7480	0,3630	NO	0,1220	0,0765	1,5944	YES
Survey	0	0,0523	0,5121	1,4715	0,3480	NO	0,1644	0,0560	2,9351	YES
	1	0,0864	0,7818	1,4715	0,5313	NO	0,1954	0,0560	3,4896	YES
	2	0,1232	1,0013	2,4651	0,4062	NO	0,1373	0,0782	1,7569	YES
Typological	0	0,0467	0,4516	1,4715	0,3069	NO	0,1607	0,0560	2,8690	YES
	1	0,0833	0,7516	1,4715	0,5107	NO	0,1987	0,0560	3,5487	YES
	2	0,1171	0,9527	2,3528	0,4049	NO	0,1391	0,0831	1,6732	YES

Tab. 5.4.17: Verification of simple overturning mechanism – US 130

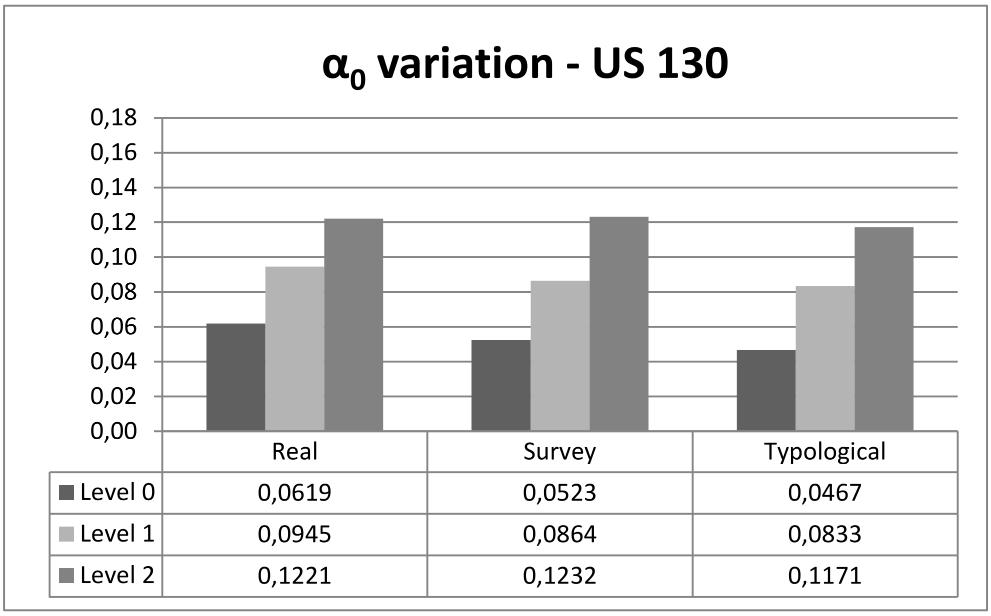


Fig.5.4.33:α_0 variation - US 130

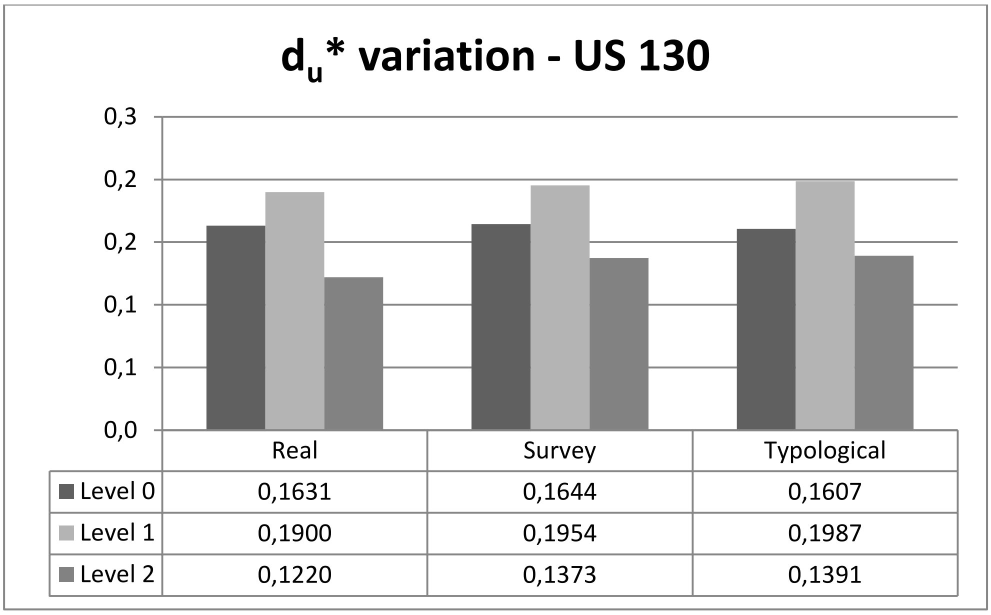


Fig.5.4.34: d_u variation - US 130*

The comparison of the simple overturning mechanism for these four structural units is helpful to define which building characteristics cause more differences with the typological analysis and which can be simplified. In cases like the US 88 and US 130 the typology well represents the building behavior, despite some differences in the interstorey height, the wall thickness and the area. In US 123 and 124 the results of the real case diverge more from the typological case due to the different horizontal structure typology. The low barrel vault in the first unit and the free standing wall in the second one cause considerably difference in the mechanism behavior, as important aspects of it. These characteristics are considered specific of the two structural units due to their not widespread presence in the city center. Obviously, there can be other particular cases that cannot be relived during a rapid survey, but they will emerge with a detailed study of the building. The aim of this thesis is to give a vulnerability assessment of the building from its easily relived characteristics, such as interstorey height, number of plan and structure typology.

5.5 VERTICAL BENDING MECHANISM

The analysis of the vertical bending mechanism is made for all the typologies that have a vaulted horizontal structure at the ground floor. The mechanism can be activated in case of a good connection of the first horizontal structure with the external wall such as to exert a restraining force at the first floor level. The analysis of this mechanism is made to consider the possible presence of restringing devices that were not relived in situ, but can make the activate of the mechanism possible.

The analysis is carried out considering the possible formation of a cylindrical hinge at the vault springer height, as a consequence of the horizontal vault thrust.

The assumptions made for the simple overturning mechanism in chapter 5.4 about mechanical properties, permanent and accidental loads and vaults trusts are valid for the vertical bending mechanism too. The only difference is that in this case the friction forces are not considered because they do not influence the mechanism.

5.5.1 Parameters description

As made for the overturning mechanism, the analysis is carried out considering a reference case and a varying set of parameters.

The wall thickness, the interstorey height, the vault type, the sept length and the analyzed area of the reference case are the same of the overturning mechanism and they are explained in chapter 5.4.1.

The set of analyzed parameters is the same as in the previous chapter but two of them are not considered: parameter 8 (free standing wall) and parameter 9 (absence of vaults at the ground floor). These parameters are not congruent to the analyzed mechanism. To avoid confusion, the same parameter numbers are kept so the analyzed parameters in this mechanism are:

> 1-<u>Constant wall thickness along the entire building height</u>: the ground floor thickness is considered constant in all the stories, from the ground to the top of the building.
>
> 2-<u>Wall thickness that decreases on each floor</u>: the wall thickness does not decrease of 15cm after the first floor and the it stays constant, but it decreases of 15cm on each floor until a minimum of 30cm.
>
> 3-<u>Minimum interstorey height for upper floors</u>

4-Maximum interstorey height for upper floors

5-Minimum interstorey height for the ground floor

6-Maximum interstorey height for the ground floor

7-Barrel vault: instead of a cross vault, a barrel vault with rise of 1m is considered.

10-Vault rise of 1.5m: the cross vault is considered to have a rise of 1.5m instead of 1m. The new thrusts are calculated with Arco program.

11-Vault rise of 0.5m: the cross vault is considered to have a rise of 0.5m instead of 1m. The new thrusts are calculated with Arco program.

The macro-typologies that have mainly vaults at the ground floor are macro-typology A and macro-typology E. The first one is characterized by masonry vertical structures, light horizontal structures and timber trusses, while the second one by masonry vertical structures, light horizontal structures and mixed concrete and timber trusses. Typologies that contain one structural unit or buildings only located in Iosefin area are not considered.

As in the case of the simple overturning mechanism, some micro-typologies are added to consider all the possible cases that can appear in the city of Timisoara (in grey in Tab. 5.5.1). In particular micro-typologies C and D are added in typology 17, with a wall thickness respectively of 75 and 90cm, micro-typologies A, C and D are added to typology 18, with a wall thickness respectively of 45, 75 and 90cm, and the micro-typology A is added to typology 20, with a wall thickness of 45 cm. The added micro-typologies do not correspond to relived structural units but take count of the possibility to extend the study to other part of the city. The interstories height and their variation are the same defined for the typology.

The set of analyzed parameters changes with the typology characteristics: typologies with one storey do not consider parameters 1, 2 3 and 4 because they refer to upper floors and typologies with two stories do not consider parameter 2 (wall thickness that decreases on each floor) because it coincides with the reference parameter.

The analyzed typologies with the respective characteristics are shown in Tab. 5.5.1.

MACRO-TYPOLOGY	TYPOLOGY	MICRO-TYPOLOGY	STORIES NUMBER	U.S.	GROUND FLOOR THICKNESS	AVERAGE INTERSTOREY HEIGHT- GROUND FLOOR	INTERSTOREY VARIATION - GROUND FLOOR	AVERAGE INTERSTOREY HEIGHT- UPPER FLOORS	INTERSTOREY VARIATION - UPPER FLOORS	AVERAGE INTERSTOREY HEIGHT- BASEMENT
A	1	1A	1	36, 37	45	3,8	± 0,5	-	-	-
		1B		152, 241	60					
	2	2A	2	7, 44, 74, 75, 98	45	4	± 0,5	4,3	± 0,5	-
		2B		8, 18, 19, 20, 29, 30, 31, 55, 58, 59, 73, 77, 84, 97, 115, 120, 121, 151, 175, 193 202, 228, 301	60					
		2C		71, 80, 117, 145, 154	75					
		2D		94, 95, 167, 185	90					
	3	3A	3	49, 50, 51	45	4,3	± 1	4,2	± 0,5	-
		3B		4, 5, 53, 76, 96, 103, 104,105, 116, 118, 123, 124, 140, 174, 178, 192	60					
		3C		81, 109, 130, 134	75					
		3D		79, 86, 87, 88, 137, 138, 141, 166, 169B, 182, 183, 188, 189, 190	90					
	4	4A	4	25, 56, 133	60	4,2	± 0,5	4	± 0,5	-
		4B		129	75					
		4C		90, 92, 93, 128, 135, 155, 156, 187	90					
	7	7A	2+SI	35, 38, 41, 61, 62, 64, 149, 150, 172, 211, 212, 213, 229, 230, 231, 233, 236, 237	60	4	± 0,5	5	± 0,5	1,3

		7B		68	75					
		7C		28, 100, 114, 217	90					
	8	8A	3+SI	46	45	4,2	± 0,5	4,4	± 0,5	1,1
		8B		9, 24, 26, 43, 47, 54, 57, 60, 65, 168, 173, 184, 200, 201, 203, 207, 221, 234, 235	60					
		8C		69, 70, 143, 153	75					
		8D		101, 170	105, 90					
E	17	17A	2	82	45	4,2	± 0,5	4,7	± 1	-
		17B		83, 85, 111, 186	60					
		17C		-	75					
		17D		-	90					
	18	18A	3	-	45	4,3	± 0,5	4,2	± 0,5	-
		18B		52, 105,125, 131	60					
		18C		-	75					
		18D		-	90					
	20	20A	3+SI	-	45	4	± 0,5	4,2	-	1,4
		20B		2	60					
		20C		148	75					
		20D		102, 159	90					

Tab. 5.5.1: Analyzed typologies and interstorey height variations

5.5.2 Verification of the mechanism

The procedure to evaluate the vertical bending mechanism is the same used for the simple overturning mechanism and it is shown in chapter 5.2. The only difference is the method to evaluate the seismic masses multiplier α_0, as exposed in chapter 5.3.2.

The typology 3 is still used as example and the verification values are shown in the following tables (Tab.5.5.2, Tab. 5.5.3, Tab. 5.5.4, Tab. 5.5.5) while the tables for the other analyzed typologies can be found in annex D2. The typology 3 is characterized by three stories high buildings, with an average interstorey height of 4.3m at the ground floor and 4.2m at the upper floors. The interstorey variation is of ±1m at for the ground floor and ±0.5m at the upper floors. It is subdivided in four micro-typologies:

- Micro-typology 3A has a wall thickness of 45cm;
- Micro-typology 3B has a wall thickness of 60cm;
- Micro-typology 3C has a wall thickness of 75cm;
- Micro-typology 3D has a wall thickness of 90cm.

3A									
PARAMETER	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
		a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0,2485	4,9289	1,4715	3,3496	YES	0,0244	0,0142	1,7155	YES
1	0,3404	8,0271	1,4715	5,4551	YES	0,0252	0,0110	2,2911	YES
2	0,2485	4,9289	1,4715	3,3496	YES	0,0244	0,0142	1,7155	YES
3	0,2209	4,1848	1,4715	2,8439	YES	0,0236	0,0152	1,5549	YES
4	0,2739	5,6769	1,4715	3,8579	YES	0,02490	0,0134	1,8602	YES
5	0,4218	9,3360	1,4715	6,3445	YES	0,0322	0,0119	2,7132	YES
6	0,1630	2,9562	1,4715	2,0090	YES	0,0195	0,0164	1,1885	YES
7	-0,1047	-1,7863	1,4715	-1,2139	NO	1,07E-5	1,4E-22	7,4E16	NO
10	0,2700	5,2499	1,4715	3,5677	YES	0,0323	0,0159	2,0380	YES
11	0,2980	6,0731	1,4715	4,1272	YES	0,0164	0,0095	1,7334	YES

Tab. 5.5.2: Verification of vertical bending mechanism – Micro-typology 3A

3B									
PARAMETER	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
		a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	0,6334	13,0170	1,4715	8,8460	YES	0,0506	0,0126	4,0160	YES
1	0,7452	17,5638	1,4715	11,9360	YES	0,0485	0,0097	5,0131	YES
2	0,7452	17,5638	1,4715	11,9360	YES	0,0485	0,0097	5,0131	YES

3	0,5856	11,4021	1,4715	7,7486	YES	0,0510	0,0135	3,7719	YES
4	0,6771	14,6447	1,4715	9,9522	YES	0,05003	0,0118	4,2350	YES
5	0,9464	22,0781	1,4715	15,0038	YES	0,0595	0,0094	6,3016	YES
6	0,4701	8,7307	1,4715	5,9332	YES	0,0449	0,0145	3,0988	YES
7	0,2139	3,8990	1,4715	2,6496	YES	0,0281	0,0172	1,6375	YES
10	0,5951	12,0476	1,4715	8,1873	YES	0,0594	0,0142	4,1840	YES
11	0,9315	19,5206	1,4715	13,2658	YES	0,0407	0,0073	5,5716	YES

Tab. 5.5.3: Verification of vertical bending mechanism – Micro-typology 3B

3C									
PARAMETER	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
		a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	1,0192	21,3690	1,4715	14,5219	YES	0,0717	0,0117	6,1250	YES
1	1,1455	26,9057	1,4715	18,2846	YES	0,0684	0,0089	7,6795	YES
2	0,9453	18,6297	1,4715	12,6604	YES	0,0587	0,0110	5,3174	YES
3	0,9490	18,7591	1,4715	12,7483	YES	0,0730	0,0126	5,7910	YES
4	1,0829	24,0046	1,4715	16,3130	YES	0,07022	0,0102	6,8515	YES
5	1,4881	35,8418	1,4715	24,3573	YES	0,0819	0,0080	10,2301	YES
6	0,7708	14,4939	1,4715	9,8497	YES	0,0652	0,0136	4,8082	YES
7	0,5463	10,4005	1,4715	7,0680	YES	0,0582	0,0151	3,8487	YES
10	0,9264	19,2056	1,4715	13,0517	YES	0,0818	0,0132	6,2025	YES
11	1,5519	33,0125	1,4715	22,4346	YES	0,0584	0,0062	9,4225	YES

Tab. 5.5.4: Verification of vertical bending mechanism – Micro-typology 3C

3D									
PARAMETER	α_0	LINEAR KINEMATIC ANALYSIS				NON-LINEAR KINEMATIC ANALYSIS			
		a_0*	Maximum verification value	Safety coefficient	VERIFICATION	d_u*	Maximum verification value	Safety coefficient	VERIFICATION
R	1,4054	29,8303	1,4715	20,2720	YES	0,0899	0,0106	8,5142	YES
1	1,5429	36,0805	1,4715	24,5195	YES	0,0860	0,0084	10,2982	YES
2	1,3273	26,7385	1,4715	18,1709	YES	0,0780	0,0102	7,6318	YES
3	1,3117	26,1546	1,4715	17,7741	YES	0,0919	0,0120	7,6721	YES

4	1,4903	33,5471	1,4715	22,7979	YES	0,08762	0,0092	9,5751	YES
5	2,0413	50,1976	1,4715	34,1132	YES	0,1013	0,0071	14,3276	YES
6	1,0674	20,2116	1,4715	13,7354	YES	0,0824	0,0129	6,3863	YES
7	0,8887	17,4385	1,4715	11,8508	YES	0,0829	0,0139	5,9475	YES
10	1,2619	26,5512	1,4715	18,0436	YES	0,1017	0,0125	8,1299	YES
11	2,1641	46,4346	1,4715	31,5560	YES	0,0722	0,0054	13,2535	YES

Tab. 5.5.5: Verification of vertical bending mechanism – Micro-typology 3D

All the cases result verified except for the parameter 7 (barrel vault) of micro-typology 3A. This case is characterized by a wall thickness of 45cm at the ground floor and 30cm at upper floors, with an horizontal thrust of the barrel vaults of 106.31 kN[15]. The percentage of almost 100% of verified cases is justified by the low vulnerability of this type of local mechanism in the analyzed cases.

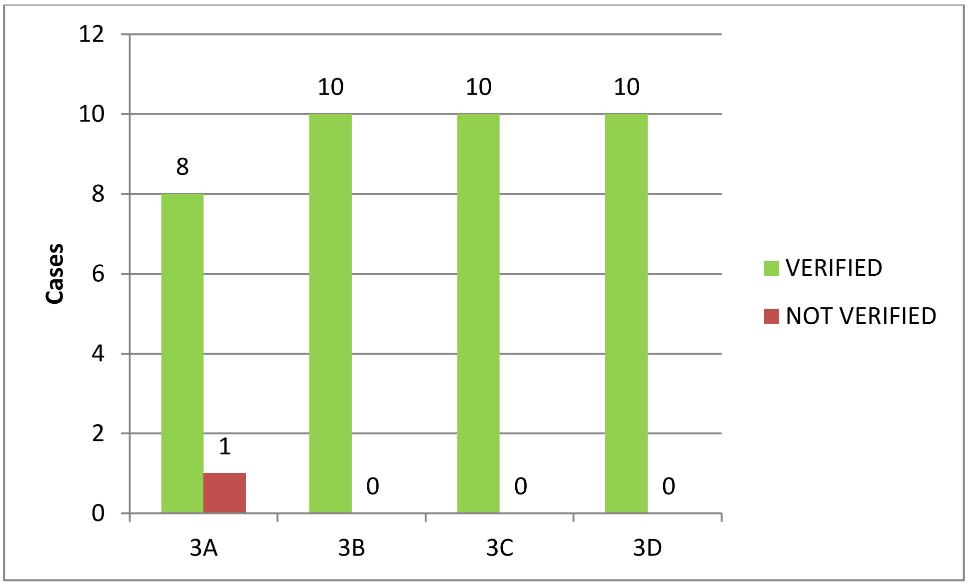


Fig. 5.4.1: Verification of simple overturning mechanism – Cases number

The wall thickness is the parameter that most influence the mechanism verification. The parameters of typology 1, which is the only one with one storey for the vertical bending mechanism result all not verified. The verification percentage of typologies 2, 7 and 17, which are characterized by two stories or two stories and basement, changes with the micro-typology and consequently with the wall thickness. In fact micro-typologies C and D (wall thickness of 75 and 90cm) result all verified, the micro-typology B (wall thickness of 60cm) results

[15] Value calculated with *Arco* program

all verified except for parameter 7(barrel vault) and the micro-typology A (wall thickness of 45cm) results not verified for more than half of the parameters. The typologies 3, 8 and 18,with three stories or three stories and basement, result all verified except for the parameter 7 (barrel vault) in micro-typology A (wall thickness of 45cm). The typology 4, which is the only one with four stories analyzed with the vertical bending, is all verified.

5.5.3 Parameters analysis

The comparison of α_0, a_0*, d_0*and d_u*values has been made for each parameter in each micro-typology. The histograms below (Fig. 5.5.3, Fig. 5.5.4, Fig. 5.5.5, Fig. 5.5.6)) show the value variations of the multiplier α_0 in each micro-typology of the building typology 3.

It is possible to observe that the value of α_0:

- Is minimum in correspondence of parameter 7 (barrel vault);
- Is maximum in correspondence of parameter 5 (minimum interstorey height for the ground floor) in micro-typologies 3A and 3B and in correspondence of parameter 11(vault rise at 0.5m) in micro-typologies 3C and 3D. While for micro-typologies 3A and 3D the maximum value is clear, for micro-typologies 3B and 3C the values of parameter 5 and 11 re very close;
- Is lower of the reference value in correspondence of parameters 2 (wall thickness that decreases on each floor), 3 (minimum interstorey height for upper floors), 6(maximum interstorey height for the ground floor) and 7 (barrel vault). Parameters 2 and 3 reduce the stabilizing contribute of the weight forces of upper floors walls, the parameter 6 increases the moment arm of the horizontal vault thrust and consequently it favors the mechanism activation, as does the parameter 7. The α_0 value of parameter 10 (vault rise at 1.5m) is lower than the reference case in micro-typologies 3B, 3C and 3D, but is higher in micro-typology 3A. This happens because increasing the distance of the central hinge from the extremities, the moment arms of the destabilizing forces rise, but in case of low wall thickness the higher vertical component of the vault thrust helps the

stabilizing forces more than the increment of the destabilizing moments helps the mechanism activation.

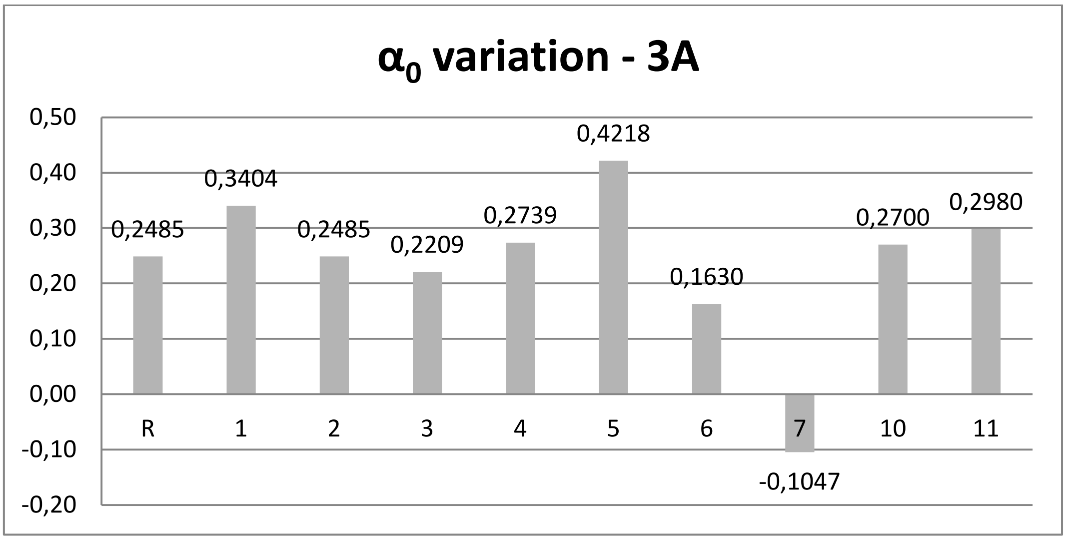


Fig. 5.5.3: Variation of α_0 *values – Micro-typology 3A*

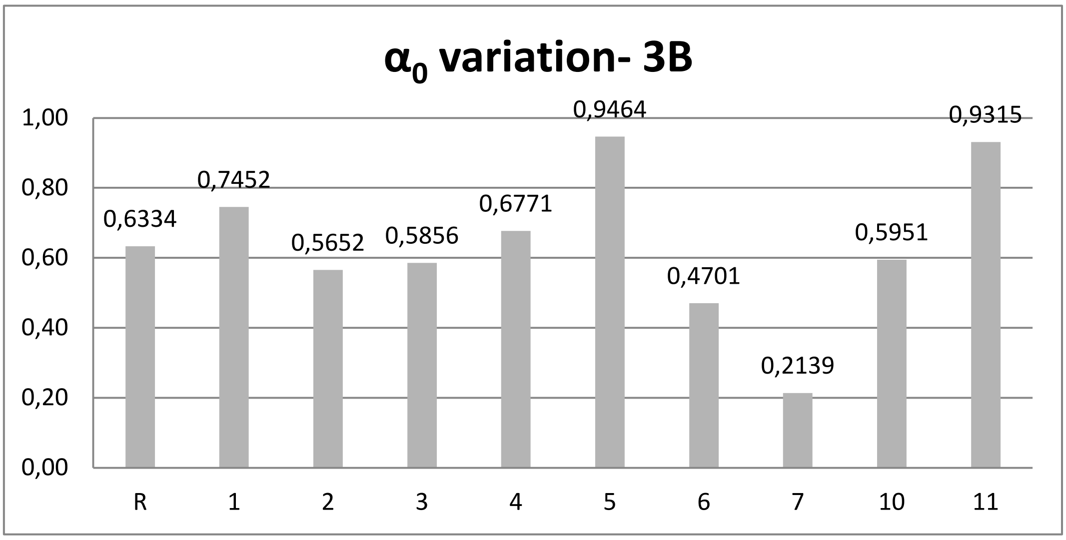


Fig. 5.5.4: Variation of α_0 *values – Micro-typology 3B*

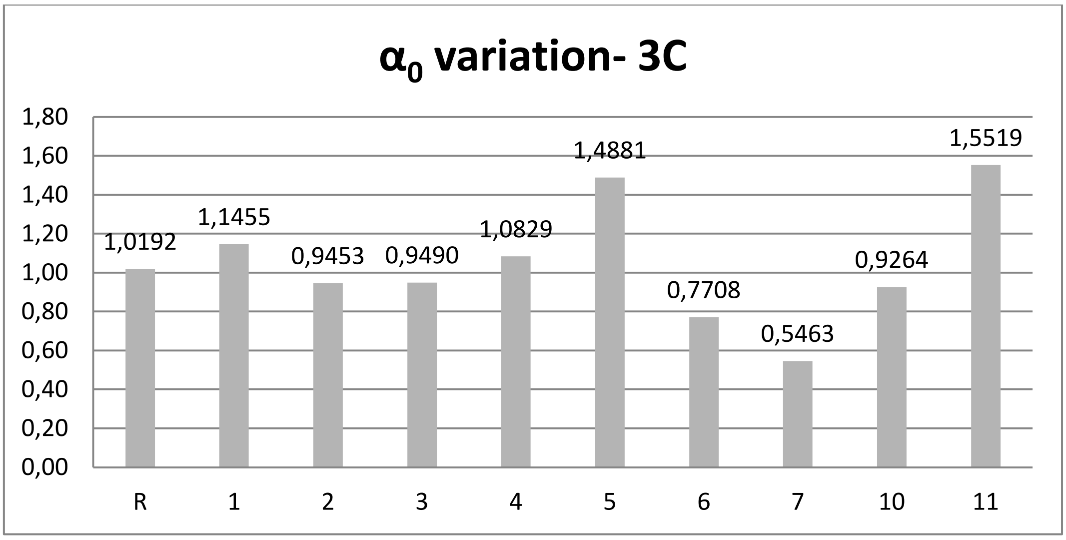


Fig. 5.5.5: Variation of α_0 *values – Micro-typology 3C*

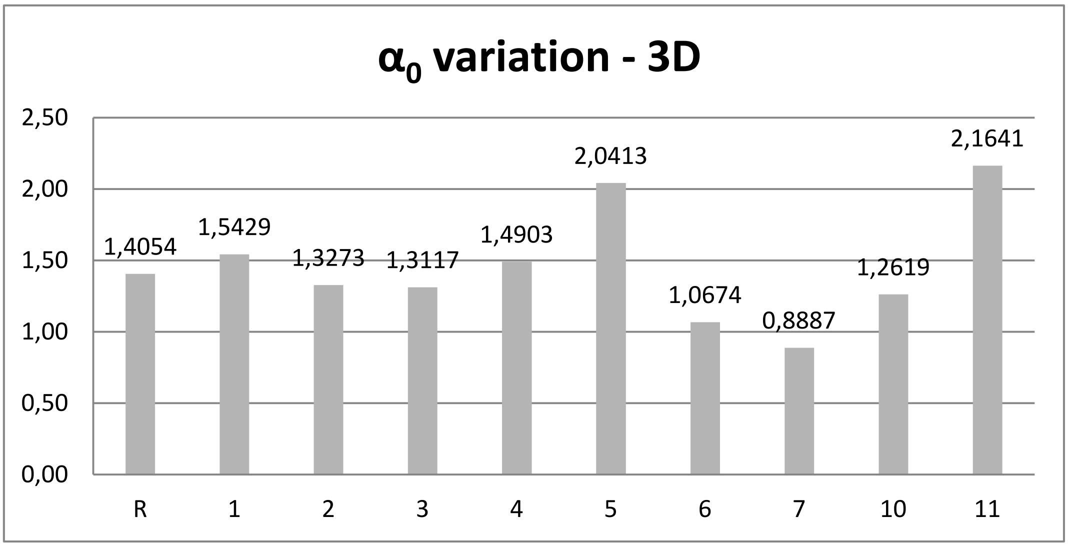


Fig. 5.5.6: Variation of α_0 *values – Micro-typology 3D*

The same comparison can be made for the ultimate displacement capacity d_u* . The histograms below (Fig. 5.5.7, Fig. 5.5.8, Fig. 5.5.9, Fig. 5.5.10) show the value variations of d_u* in each micro-typology of the building typology 3.

In general the values are quite homogeneous through all the micro-typologies. An exception is made for parameter 7 (barrel vault) of micro-typology 3A that, as already seen, represent the most vulnerable case. Low values of d_u*correspond to parameters 6 (maximum interstorey height for the ground floor) and 11(vault rise at 0.5m) for micro-typology 3A, to parameters 6, 7 and 11 for micro-typology 3B, to parameters 2 (wall thickness that decreases on each floor), 7 and 11 for micro-typology 3C. in micro-typology 3D the values are more homogenous and even if there are higher and lower values there is not an evident minimum.

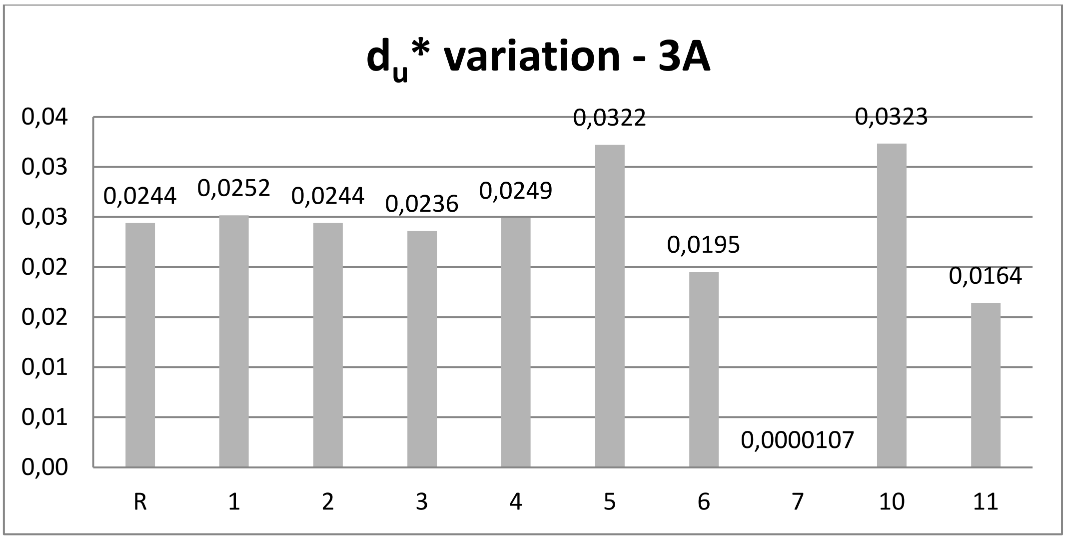


Fig. 5.5.7: Variation of d_u values – micro-typology 3A*

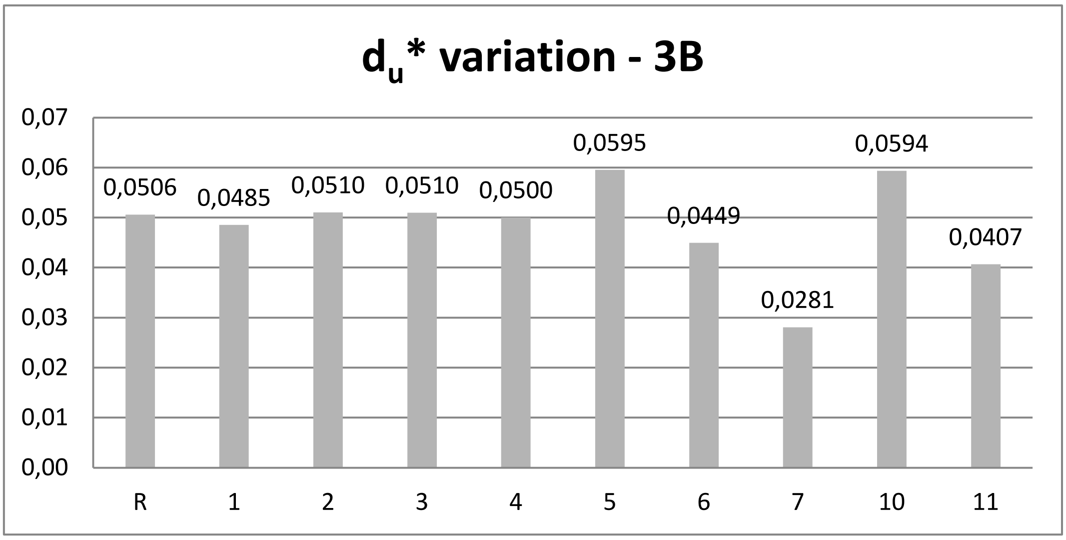


Fig. 5.5.8: Variation of d_u values – micro-typology 3B*

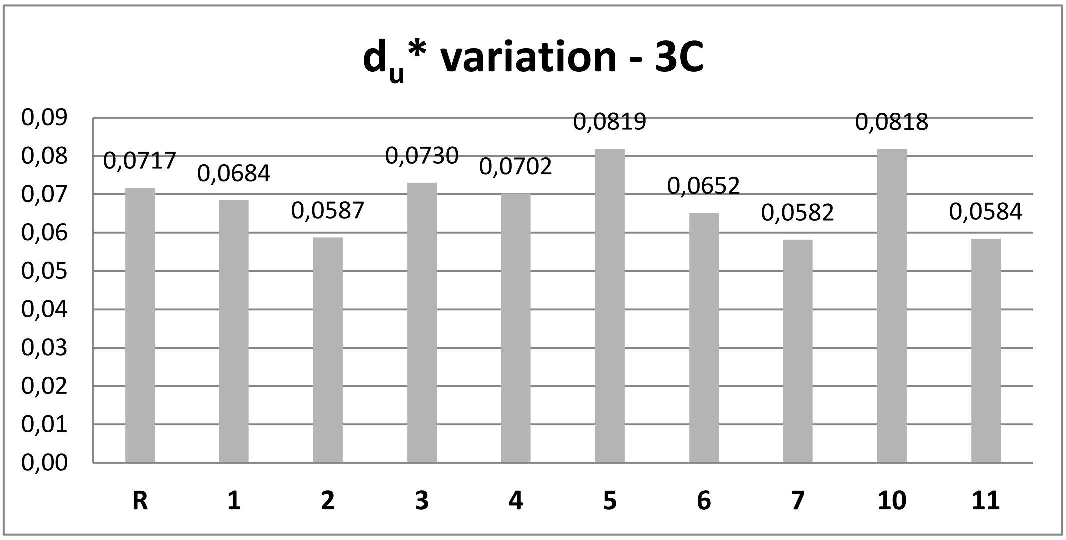


Fig. 5.5.9: Variation of d_u values – micro-typology 3C*

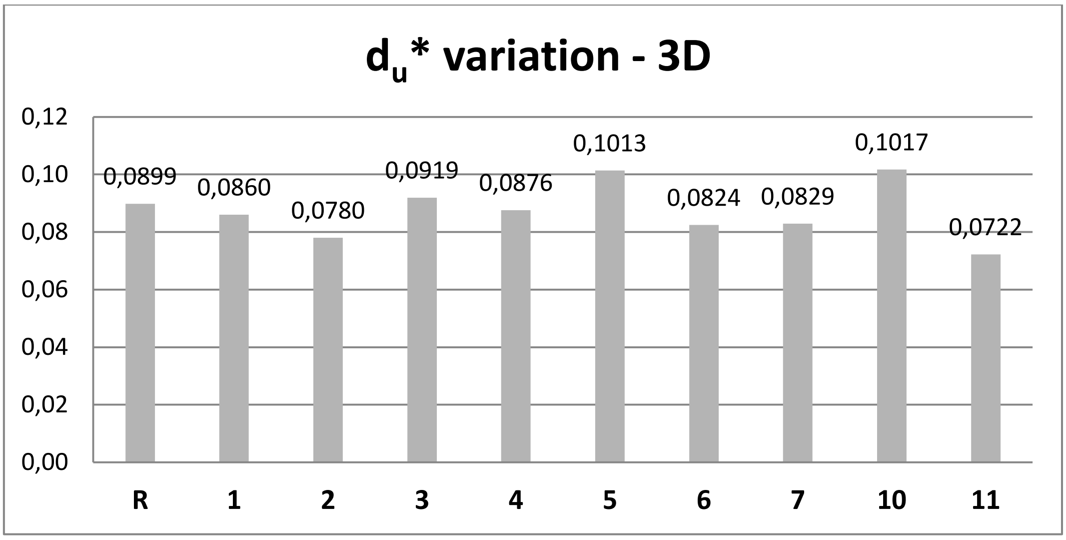


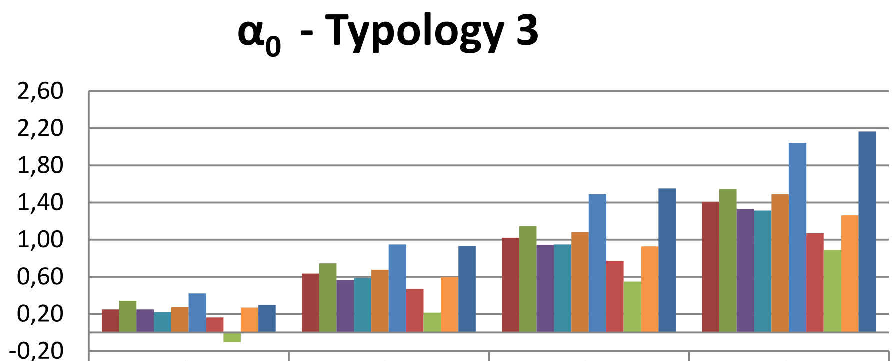
Fig. 5.5.10: Variation of d_u values – micro-typology 3D*

A comparison has been made also for a_0*and d_0* values as it has been made for α_0 and d_u*. The histograms have the same trend of d_u* ones, and the same considerations made above can be extended to a_0*and d_0*comparison.

To consider the influence of the wall thickness, the values of all the parameter for each micro-typology have been compared. (Fig. 5.5.11)

It is possible to observe that the values are deeply influenced by the wall thickness, for example the α_0 value of the reference case is 0.2485 for a wall thickness of 45cm and it is 1.4054 for a wall thickness of 90cm. The increment in percentage is 466%. Some parameters have an even higher increment, like parameter 11 (vault rise at 0.5), growing of 626%.

The wall thickness is the most influential building geometrical characteristics in case of vertical bending mechanism, followed by the values of the ground floor interstorey height (parameters 5 and 6).



	3A (wall thickness 45 cm)	3B (wall thickness 60 cm)	3C (wall thickness 75 cm)	3D (wall thickness 90 cm)
REFERENCE	0,2485	0,6334	1,0192	1,4054
Parameter 1	0,3404	0,7452	1,1455	1,5429
Parameter 2	0,2485	0,5652	0,9453	1,3273
Parameter 3	0,2209	0,5856	0,9490	1,3117
Parameter 4	0,2739	0,6771	1,0829	1,4903
Parameter 5	0,4218	0,9464	1,4881	2,0413
Parameter 6	0,1630	0,4701	0,7708	1,0674
Parameter 7	-0,1047	0,2139	0,5463	0,8887
Parameter 10	0,2700	0,5951	0,9264	1,2619
Parameter 11	0,2980	0,9315	1,5519	2,1641

Fig. 5.5.11: Variation of α_0 values for vertical bending mechanism – Typology 3

The α_0 values of typology 1, characterized by one storey, is negative for all the analyzed parameters, except for the parameter 10 (vault rise at 1.5m) in micro-typology B (wall thickness of 60cm). Consequently, the values of a_0*, d_0* and d_u* are negative too. The results negative sign leads to think that probably there are some restraint devices that were not relived in situ and that prevent the mechanism activation in the present state. Negative α_0 values appear also in correspondence of parameters 7 (barrel vault) and 11 (vault rise at 0.5m) of micro-typology 2A, and in correspondence of parameter 7 in micro-typologies 8A and 17A. In these two last micro-typologies the value of parameter 11 is positive but extremely low compared to the other parameters. The interstorey height of typologies 8 and 17 is indeed a little bit higher than the one of typology 2, and even if this building characteristic is not the most influential, in extreme cases can be determinant for the mechanism activation.

5.5.4 Capacity curves

The capacity curve of the equivalent oscillator is defined with the method exposed in chapter 5.4.4, with the calculation of the spectral acceleration a_0*, the equivalent spectral displacement d_0*, the secant *Ts* and consequently the values of a_s*, d_s* and d_u*. The values are calculated for each parameter and reported in the following tables (Tab.5.5.6, Tab.5.5.7, Tab.5.5.8, Tab.5.5.9). The same values have been calculated for each analyzed typologies.

3A					
PARAMETER	a_0*	d_0*	a_s*	d_s*	d_u*
R	4,9289	0,0610	4,1403	0,0098	0,0244
1	8,0271	0,0629	6,7428	0,0101	0,0252
2	4,9289	0,0610	4,1403	0,0098	0,0244
3	4,1848	0,0590	3,5152	0,0094	0,0236
4	5,6769	0,0623	4,7686	0,0100	0,0249
5	9,3360	0,0805	7,8422	0,0129	0,0322
6	2,9562	0,0488	2,4832	0,0078	0,0195
7	-1,7863	0,0000	-1,5005	4,3E-6	1,07E-5
10	5,2499	0,0808	4,4100	0,0129	0,0323
11	6,0731	0,0411	5,1014	0,0066	0,0164

Tab. 5.5.6: Capacity curve values – Micro-typology 3A

3B					
PARAMETER	a_0*	d_0*	a_s*	d_s*	d_u*
R	13,0170	0,1265	10,9342	0,0202	0,0506
1	17,5638	0,1213	14,7536	0,0194	0,0485
2	10,7628	0,1276	9,0407	0,0204	0,0510
3	11,4021	0,1274	9,5778	0,0204	0,0510
4	14,6447	0,1251	12,3015	0,0200	0,0500
5	22,0781	0,1488	18,5456	0,0238	0,0595
6	8,7307	0,1123	7,3337	0,0180	0,0449
7	3,8990	0,0702	3,2751	0,0112	0,0281
10	12,0476	0,1484	10,1200	0,0237	0,0594
11	19,5206	0,1016	16,3973	0,0163	0,0407

Tab. 5.5.6: Capacity curve values – Micro-typology 3B

3C					
PARAMETER	a_0*	d_0*	a_s*	d_s*	d_u*
R	21,3690	0,1793	17,9500	0,0287	0,0717
1	26,9057	0,1710	22,6008	0,0274	0,0684
2	18,6297	0,1468	15,6490	0,0235	0,0587
3	18,7591	0,1826	15,7577	0,0292	0,0730
4	24,0046	0,1756	20,1639	0,0281	0,0702
5	35,8418	0,2047	30,1071	0,0328	0,0819
6	14,4939	0,1629	12,1748	0,0261	0,0652
7	10,4005	0,1455	8,7364	0,0233	0,0582
10	19,2056	0,2046	16,1327	0,0327	0,0818
11	33,0125	0,1461	27,7305	0,0234	0,0584

Tab. 5.5.6: Capacity curve values – Micro-typology 3C

3D					
PARAMETER	a_0*	d_0*	a_s*	d_s*	d_u*
R	29,8303	0,2246	25,0574	0,0359	0,0899
1	36,0805	0,2151	30,3076	0,0344	0,0860
2	26,7385	0,1950	22,4604	0,0312	0,0780
3	26,1546	0,2298	21,9699	0,0368	0,0919
4	33,5471	0,2191	28,1795	0,0350	0,0876
5	50,1976	0,2534	42,1660	0,0405	0,1013
6	20,2116	0,2061	16,9777	0,0330	0,0824
7	17,4385	0,2072	14,6484	0,0331	0,0829
10	26,5512	0,2542	22,3030	0,0407	0,1017
11	46,4346	0,1806	39,0051	0,0289	0,0722

Tab. 5.5.6: Capacity curve values – Micro-typology 3D

A first comparison is made between the curves on the same micro-typology (Fig.5.5.12, Fig.5.5.13, Fig.5.5.14, Fig.5.5.15). The trend of the curves is similar for all the micro-typologies: the higher curve correspond to parameter 5(minimum interstorey height for the ground floor) and the lower curve correspond to parameter 7 (barrel vault) while all the other curves are included in a medium group. In the vertical bending mechanism, the curve slope changes with the parameter so the curves are not parallel to each other. The second lower curve corresponds to parameter 6 (maximum interstorey height for the ground floor) and while for low wall thickness it is quite distant from the parameter 7 curve, for higher thickness the two curves are almost coincident. It can be observed that with the increment of the wall thickness, the spectral accelerations and the ultimate displacements increase too. The slope of the secant T_s is more diversified in this mechanism than in the simple overturning mechanism. Same considerations are valid for all the analyzed typologies because the curves trend is not considerably influenced by the stories number.

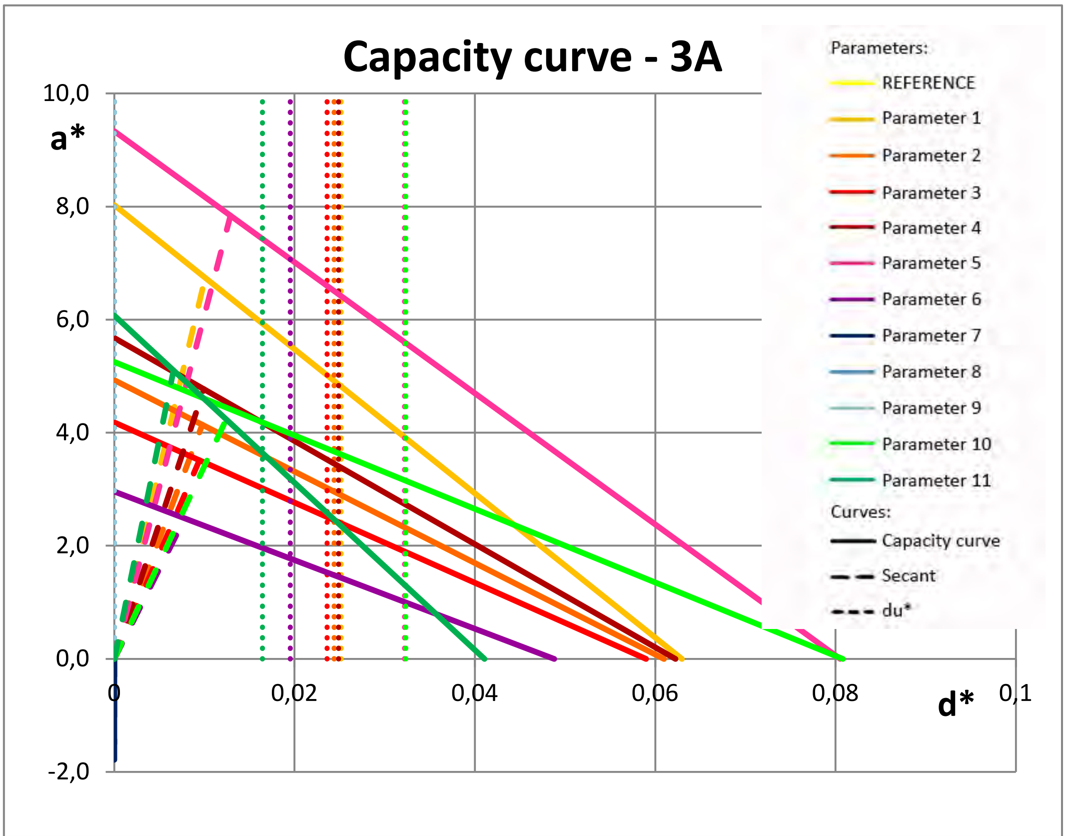


Fig.5.5.12: Capacity curve of micro-typology 3A(wall thickness 45cm)

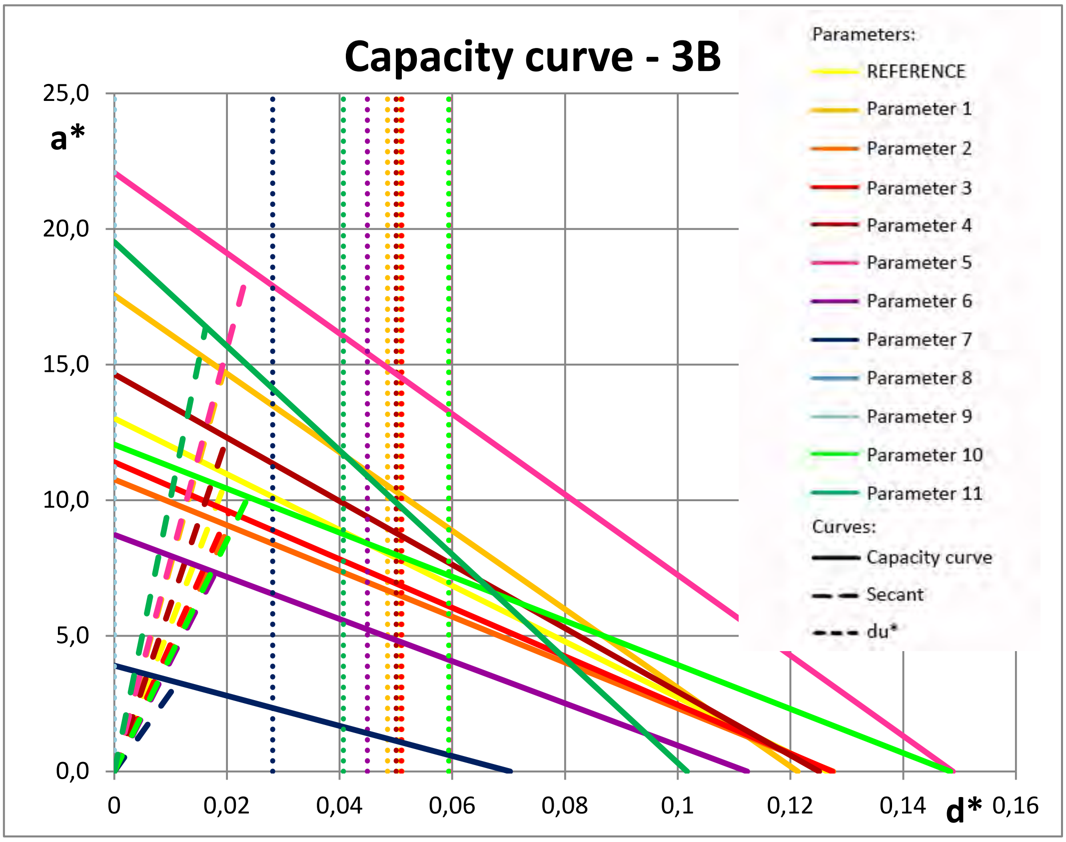


Fig.5.5.13: Capacity curve of micro-typology 3B(wall thickness 60cm)

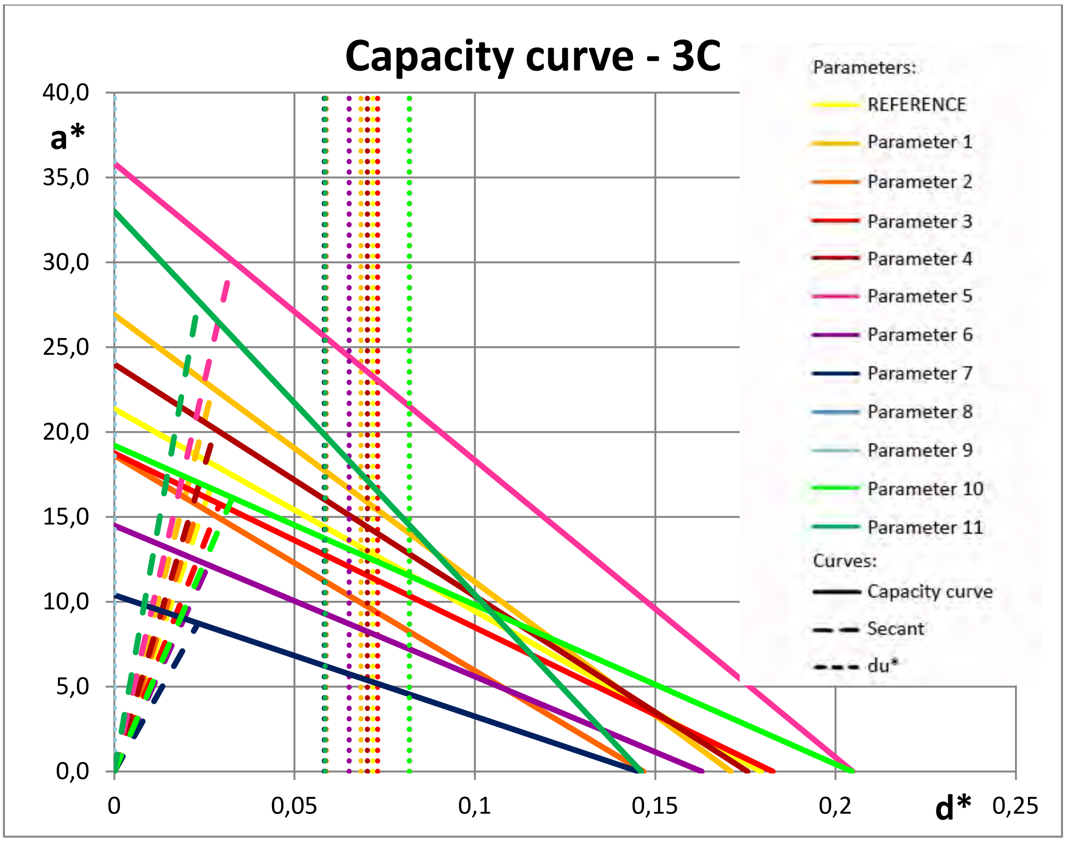


Fig.5.5.14: Capacity curve of micro-typology 3C(wall thickness 75cm)

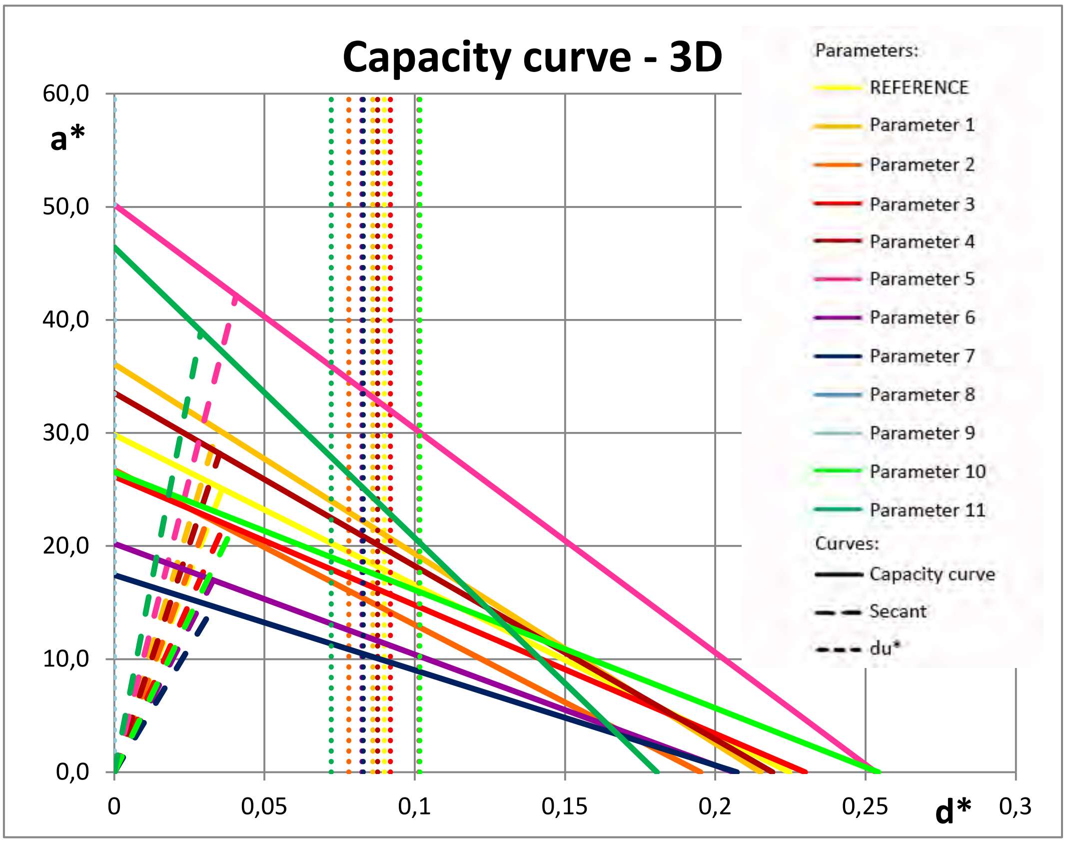


Fig.5.5.15: Capacity curve of micro-typology 3D(wall thickness 90cm)

All the curves have been then combined in the graph below (Fig.5.5.16) for the typology 3. The slopes of the curves are quite diversified , as well as the secants, but they are almost parallel to each other considering the same parameter for each micro-typology. Some of the lowest curves include:

- Parameter 7 (barrel vault) of micro-typology 3A, as already seen above;
- Parameter 6 (maximum interstorey height for the ground floor) of micro-typology 3A, which improves the moment arm of the destabilizing forces;
- Parameter 3 (minimum interstorey height for upper floors) of micro-typology 3A, which decreases the stabilizing contribute of the weight force of upper floors walls;
- Parameter 11(vault rise at 0.5) of micro-typology 3A, which is characterized by an higher value of the horizontal thrust;
- Parameter 7 (barrel vault) of micro-typology 3C.

As already noted analyzing the Fig. 5.5.11, the comparison of the capacity curves for each micro-typologies underlines the importance of the wall thickness as the characteristic that more influences the vertical bending behavior.

The comparison of the capacity curves, both inside the same micro-typology and in the global typology, for all the analyzed cases are shown in annex D2.

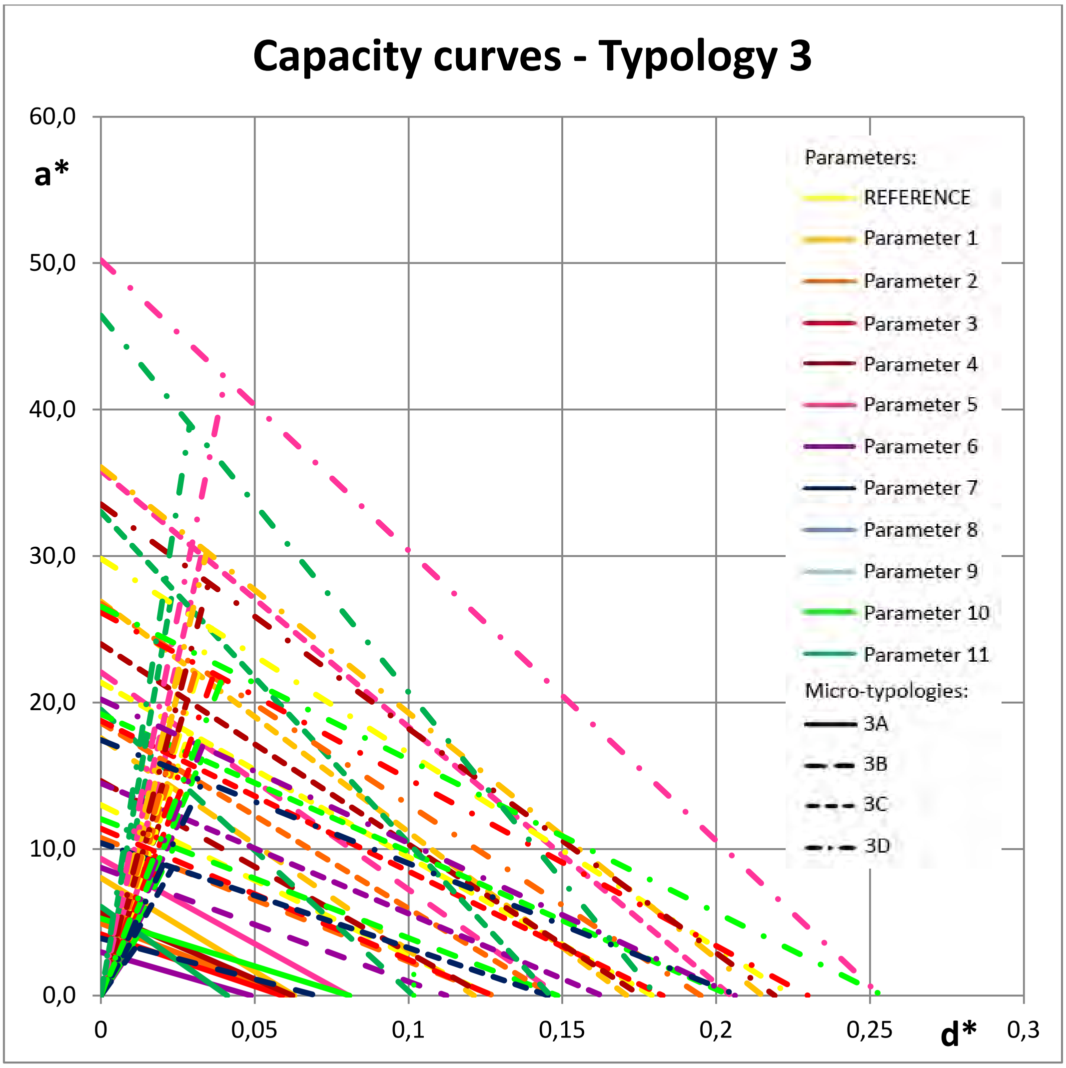


Fig.5.5.16: Capacity curves of typology 3

5.6 IN-PLANE MECHANISM

The analysis of the in plane mechanism is made for 7 of the 9 classes defined in chapter 2.4.5, because two of them represents a single corner structural units characterized respectively by four stories (class III) and four stories and basement (class IV). The classes do not consider the units in the district of Iosefin.

The analysis is carried out considering the activation of the mechanism in the most vulnerable situation: the last plan of corner units.

The assumptions made for the simple overturning mechanism in chapter 5.4 about mechanical properties, permanent and accidental loads and vaults trusts are valid for the in plane mechanism too. The only difference is that in this case the friction forces are not considered because they do not influence the mechanism. The in-plane mechanism involves only the last storey, so only the roof typology and the horizontal structure of the attic are considered.

As made for the simple overturning mechanism, the analysis for the in-plane mechanism of collapse is made considering a reference case, which is different for each micro-class, and a set of variable parameters.

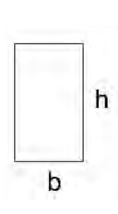
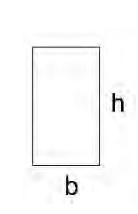
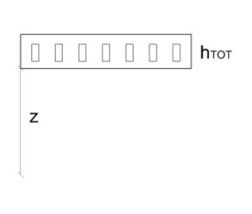
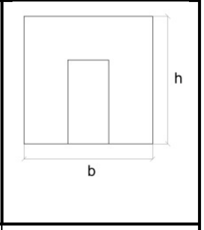
As seen in chapter 2.4.4, each façade class can be subdivided in more micro-classes, referring to the spans number. Each of these micro-classes has 8 different cases: the reference case and 7 other cases corresponding to the 7 analyzed parameters.

The reference case considers always a wall thickness of 45cm, to make the cases comparable. The interstorey height, already analyzed in chapter 2.4.4, is referred to the structural units and their relation with the typologies defined in chapter 2.4.2. The reference case considers the average value for the top floor, which defines the highness of the spandrel wall over the windows of the last considered storey, and the average value of the other stories (ground floor and upper floors if the class is more than two stories high), to define the height from the ground. The roof typology is a determinant characteristic of the class and it can be characterized by timber trusses or by concrete and timber trusses. To each of these two roof typology a weight is associated. The span length was evaluated referring to the available pictures of the buildings and to the facades length of the structural units measured in the dwg files. With the picture rectification a first span dimension has been defined, following the windows pattern. This dimension has been later compared and adjusted with the plan modules defined in chapter 2.4 for each class. All the classes are indeed subdivided in micro-classes, according to the spans number and consequently to the façade length. Finally, the analyzed parameters are:

1. Maximum interstorey height for the top floor: it defines the maximum masonry spandrel wall over the windows;
2. Minimum interstorey height for the top floor; it defines the minimum masonry spandrel wall over the windows;
3. Maximum distance from the ground; if the class is characterized by two stories buildings, it implies the maximum interstorey height for the ground floor. If the class is characterized by more than two stories buildings, it implies the sum between the maximum interstorey height for the ground floor and for the upper floors;
4. Minimum distance from the ground; if the class is characterized by two stories buildings, it implies the minimum interstorey height for the ground floor. If the class is characterized by more than two stories buildings, it implies the sum between the minimum interstorey height for the ground floor and for the upper floors;
5. Wall thickness of 30 cm
6. Wall thickness of 60 cm
7. Wall thickness of 75 cm

Parameters 1, 2, 3 and 4 consider the range defined in chapter 2.4.2.

The analyzed typologies with the respective characteristics are shown in Tab. 5.6.1.

ROOF	CLASSES	MICRO-CLASSES	SPANS NUMBER	STORIES NUMBER	WINDOW DIMENSIONS (bxh) [m]	MODULE (bxh) [m]	FACADE DIMENSIONS (h_{TOT};z) [m]		SPAN (bxh) [m]
							h_{TOT}	z	
									
CONCRETE AND TIMBER TRUSSES	I	a	2	2	1,15x2,9	7 (3,8x4,15)	4,6 ±1	4,1 ±0,5	3,8x4,6
		b	5						
		c	7						
		d	11						
	II	a	3	3	1,15x2	4 (2,9x5,15)	3,75 ±1	8,7 ±0,5	2,9x2,75
		b	6						
		c	8						
		d	10						
		e	13						
TIMBER TRUSSES	V	a	5	2	1,3x2,3	7 (3,8x4,15)	4,3 ±0,5	4 ±0,5	3,8x4,3
		b	7						
		c	9						
		d	11						
	VI	a	5	2+B	1x2,8	7 (3,8x4,15)	5 ±0,5	5,3 ±0,5	3,8x5
		b	7						
		c	9						
	VII	a	4	3	1,3x2	8 (3,4x6,75)	4,2 ±0,5	10,5 ±1	3,4x3,2
		b	6						
		c	8						
		d	9						
		e	11						
	VIII	a	8	3+B	1,1x2	4 (5,15x2,9)	4,4 ±0,5	10,7 ±1	2,575x3,4
		b	12						
		c	15						
	IX	a	5	4	1,2x2,2	3 (4,1x5,8)	4,1 ±0,5	16,8 ±1	4,1x3,1
		b	7						
		c	10						

Tab. 5.6.1: Analyzed typologies and interstorey height variations

The procedure to evaluate the in plane mechanism is the same used for the simple overturning mechanism and it is shown in chapter 5.2. All the cases result verified with the linear kinematic analysis due to the low vulnerability of this type of local mechanism. The parameter with a minor safety coefficient, in all the micro-classes, is parameter 2, characterized by a minor interstorey height of the top floor. The masonry panel dimensions remain constant so the measure variation interests the masonry spandrel wall above the windows. To reduce the spandrel wall height means to decrease the stabilizing contribute of its weight force.

Comparing the two groups of classes, characterized by the roof type, the one with the timber trusses roof has generally higher values of α_0, d_0*, a_s, d_u*, compared to the one with the same stories number of the concrete trusses type.

6 FRAGILITY CURVES[1]

The Performance Based Engineering (PBE) is the basis of the new structural design codes and it uses probabilistic concepts based on the awareness that the loads arising from usage and external events (demand), the man-made and natural hazards and the strengths of material constructions (capacity) are uncertain in nature. The risk is defined by the combination of all these aspects and it is managed by the provisions in standards and codes.

The structural response of a building is dynamic and it needs to be related to the damage that occurs under repeated (usually inelastic) cyles. The structural actions induced by a seismic event involved the entire system.

The earthquake resistance philosophy is to limit the occurrence of life-threatening damage under the design earthquake and the structure has to retain a substantial margin of safety against the overall collapse.

The performance assessment and the design process have been divided into simpler elements in terms of description, definition and quantification of earthquake intensity measures (IMs), engineering demand parameters (EDPs), damage measures (DMs) and decision variables (DVs). Examples of these parameters are:

- The peak ground acceleration and the first-mode spectral acceleration for IMs;
- The interstorey drift ratios and the inelastic component deformations for EDPs;
- The damage states of structural and nonstructural elements and dead for DMs;
- The direct financial losses and downtimes for DVs.

All these elements are considered in the “PEER Equation”:

$$P(DV) = \iiint P(DV|DM) \cdot |dP(DM|EDP)| \cdot |dP(EDP|IM)| \cdot |dH(IM)| \qquad (6.1)$$

[1] *Strategies for seismic assessment of common existing reinforced concrete bridges typologies* (Morbin, 2013, pp. 17-24);
Derivable 35 - Definition of seismic safety verification procedures for historical buildings (PERPETUATE, 2012, pp. 12-14);
Ponti in muratura: valutazione sismica mediante curve di fragilità (Thiella, 2014).

where:

- P(DV|DM) is the probability that the DV exceeds a specific value, conditioned by the structural damage DM. The estimation of DV comes from probabilistic analysis of economic losses, and it is difficult to perform;
- P(DM|EDP) is the probability that DM exceeds a specific value, when a certain value is given to parameter EDP. Considering different IMs, this term is denoted as the seismic fragility;
- P(EDP|IM) is the probability that EDP exceeds a certain value given a particular value of IM;
- H(IM) is the seismic hazard of the site, obtained by a Probabilistic Seismic Hazard Analysis (PSHA).

6.1 SEISMIC VULNERABILITY

The evaluation of the seismic damage is important to assess, during an earthquake, the risk evaluation. In order to evaluate the seismic damage is essential to identify the vulnerability elements associated to different damage levels. The capacity of the structure must be compared with the associated seismic demand and it can be represented as a displacement or an acceleration.

The seismic evaluation considers the materials properties, the intensity, the frequency and the duration of the seismic action and the characteristics of the site; these parameters represent the demand. The data considered in this analysis cannot be defined for sure, so it is necessary to associate them to a measure of uncertainty and fortuity that takes count of the probabilistic nature of the problem.

The fragility curves are graphs that express the conditional probability of an element to match or exceed a certain damage state (or performance level) for various levels of ground shaking IM, typically the peak ground acceleration (PGA) or the spectral acceleration (Sa).

The figure below (Fig. 6.1) shows how the capacity and the demand diagrams are obtainable using a probabilistic distribution. Due to this, the performance point of the structure, the intersection between the capacity curve and the demand ones, is not identified by a single value, but rather from a range of points.

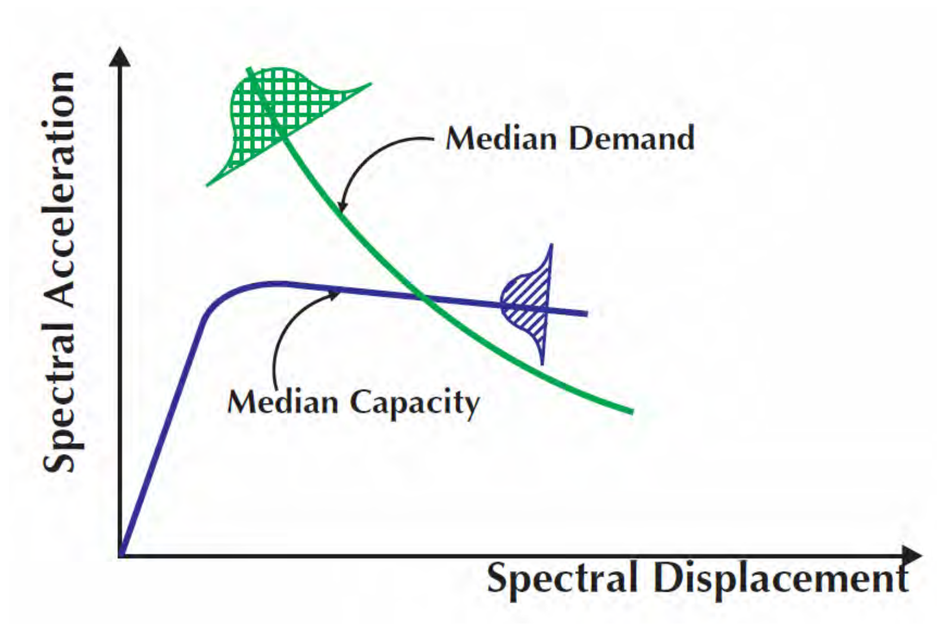


Fig. 6.1.1: Capacity-demand Acceleration-Displacement spectra showing uncertainty in structural behavior and ground motion

REFERENCE: (Mander et al.,1999)

The fragility curve is a log-normal cumulative probability function[2]. To define the curves only two parameters are necessary:

- a median (the 50th percentile);
- a normalized logarithmic standard deviation.

The cumulative probability functions is given by:

$$P(D > d_{PL} | IM) = P(S_d > S_c | IM) = \Phi\left[\frac{\ln\left(\frac{S_d}{S_c}\right)}{\beta}\right] \tag{6.2}$$

- S_d is the structural demand (damage on the structure) which changes for each IM;
- S_c is the structural capacity related to a specific performance level (median or expected value);
- β is the normalized composite log-normal standard deviation which takes into account uncertainty and randomness for both demand and capacity, it can also be computed as $\beta = \sqrt{\beta_d^2 + \beta_c^2}$ considering demand and capacity contributions respectively;
- $\Phi[\bullet]$ is the standard normal distribution function.

In literature there are two different methods to create the fragility curves: the empirical one and the analytical one.

The empirical method is based on the collection of data after a seismic event[3].As an exemple, the Hazus project and the Risk-UE are empirical methods.

The analytical method is used when there are no data collected for the post-earthquake damages, like in Romania, and it is necessary to develop a model for each structure to obtain information. In this case the necessary data, related to the

[2] (VV.AA., 1999);
(Cornell et al., 2002);
(Monti & Nisticò, 2002);
(Choi et al., 2003);
(Nielson & DesRoches, 2007)

[3] Bazos, Northridge earthquake, 1194 and Shinouzuka, Kobe earthquake, 1995

seismic response of the structure, can be derived by different types of analysis: elastic analysis, nonlinear static analysis (push-over)[4], and non-linear dynamic analysis in time history[5]. The last analysis is the most reliable but also the most onerous.

The analytical methods consist of three steps:

- First step: simulation of ground motions;
- Second step: representation of the element using an analytical model, considering the uncertainties strictly related to it;
- Third step: generation of the fragility curves from the seismic response data obtained using the analytical model.

Recently the application of non-linear static analysis to probabilistic analysis demonstrated their effectiveness and adequacy. The dynamic non-linear analysis is the most reliable method, but it is complex and unsuitable to test a large number of buildings.

This work is based on Shinozuka method: it used a non-linear static analysis based on CSM method (Capacity spectrum method). The method was applied by Shinozuka to masonry bridge structures but it can be adapted to other types of structure, like masonry buildings.

6.1.1 Definition of the performance level on the pushover curve

The identification of damage levels is fundamental in order to define fragility curves. Damage measures in earthquake engineering proposed in scientific literature are numerous and particularly they can be defined for each structural element and sub-elements (local indexes) or for the entire structure (global indexes). The most commonly used parameters for the evaluation of structural damage are the ductility, expressed by rotation, curvature or displacement, and the plastic energy dissipation.

In the non-linear kinematic analysis the i-th level of damage is associated to the displacement which has the horizontal load multiplier α of the pushover curve equal to 0 (d_0). Particularly the i-th damage level is calculated starting from the

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[4] (Shinouzuka et al., 2000)

[5] (Karim, 2001, Choi, 2003);

(DesRoches et al., 2006)

ultimate spectral displacement d_0* of the one degree of freedom equivalent oscillator.

The criteria proposed to define the damage levels are summarized in Tab 6.1. In particular:

- The PL1 level is the displacement value and correspondents to

$$a_{DL1} = 0.7a_{DL2} = 0.7a_S\,. \tag{6.3}$$

- The PL2 level is associated to the yeld point and corresponds to

$$a_{DL2}=as \tag{6.4}$$

- The PL3 level corresponds to

$$d_{DL3}=0.25d_0 \tag{6.5}$$

- The PL4 level corresponds to

$$d_{DL4}=0.4d_0\,. \tag{6.6}$$

  The PL4 level is the conventional reference point and the different analysis typologies of the Normative consider the structure characterized by the “ultimate” condition.

As explained in the “Derivable 35”, the limit values proposed to define damage states 3 and 4 have been calibrated on the basis of an extended set of nonlinear incremental dynamic analyses, performed by UNIGE: as resulting from these analyses, it may be stated that the value of 40% of the ultimate displacement capacity in most cases insures against the occurrence of some dynamic instability of the block. On the safe side, the values proposed are within this limit; actually, they could be further refined and corroborated on basis of additional numerical analyses or experimental data available for the given structure examined[6].

[6] *Definition of performance level on the pushover curve* (Derivable 35, 2012, p. 14)

DLi	Single block or Single Macro-element
Minor 1	In terms of percentage of the horizontal multiplier associated to d_{DL2} d_{DL1} corresponds to the point in which the multiplier is $\alpha_{DL1} = 0.7\ \alpha_{DL2}$
Moderate 2	In terms of percentage of d_y and check on d_{peak} $d_{DL2} = \min(d_y; d_{peak})$
Extensive 3	In terms of percentage of the ultimate displacement capacity d_0 $d_{DL3} = 0.25\ d0 \geq d_{DL2}$
Complete 4	In terms of percentage of the ultimate displacement capacity d_0 $d_{DL4} = 0.4\ d0 \geq d_{DL2}$

Tab. 6.1.1: Definition of level of damage

REFERENCE: Definition of performance level on the pushover curve (Derivable 35, 2012, p. 14)

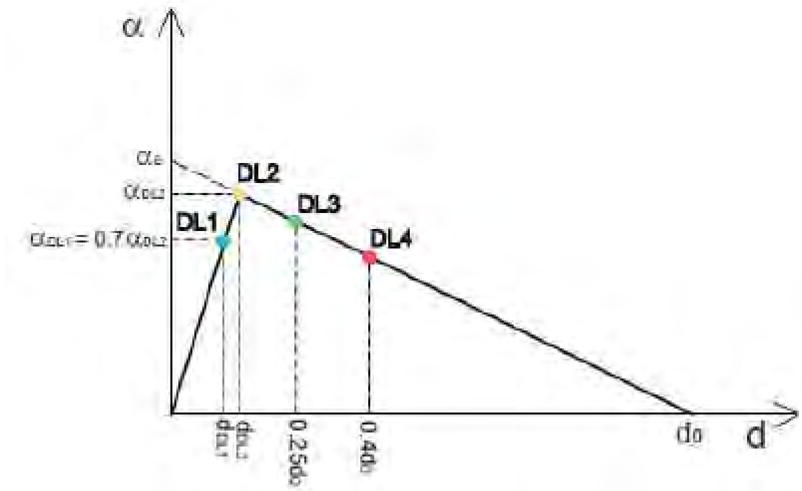

Fig 6.1.2: Criteria to define DLs on the pushover curve in case of non-linear kinematic analysis

REFERENCE: Definition of performance level on the pushover curve (Derivable 35, 2012, p. 14)

6.2 METHODOLOGY

The earthquake intensity is another factor that cannot be controlled; this work considers only one type of soil (C) but it takes count of different PGA values, with a range from 0 to 0.4 with steps of 0.05, where the normative value is the medium one (Fig 6.2.1)[7].

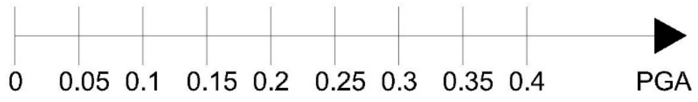

Fig 6.2.1: Acceleration range considered for the soil typology C

Eight elastic response spectra have been determined for each PGA, representing the different acceleration analyzed in this work (Fig 6.2.2).

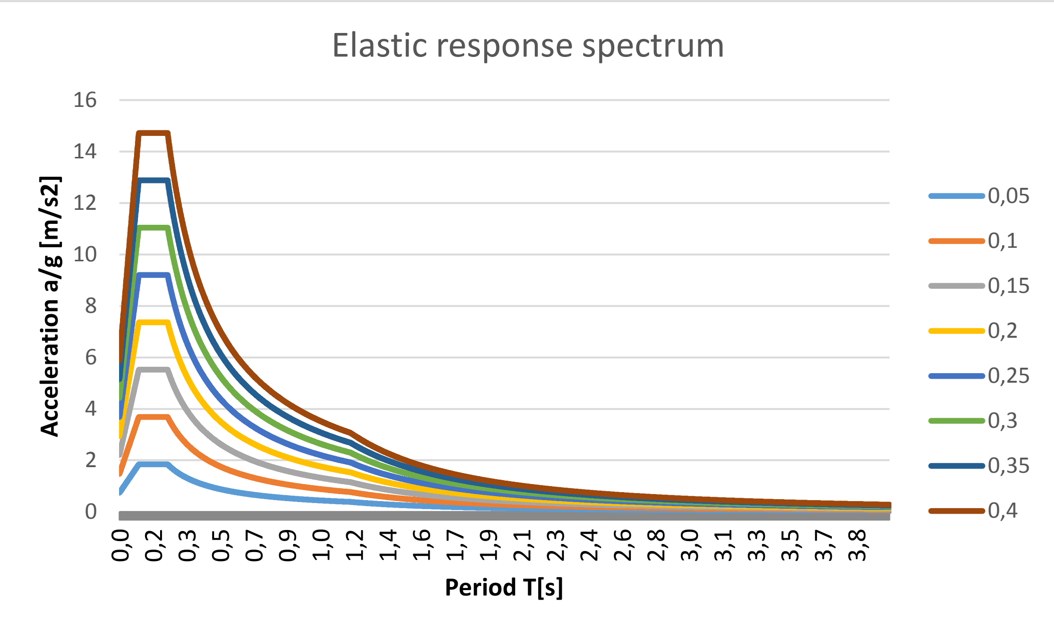

Fig 6.2.2: Elastic spectrum response for different PGA

As mentioned before, the method considered for this thesis is the one proposed by Shinozuka et al. (2000), based on the "CSM Method". A non-linear static method is used to define the intersection point between the demand curve and the capacity curve (Fig 6.2.3).

This point is the "performance point" and represents the maximum expected displacement related to the PGA considered.

[7] *Ponti in muratura: valutazione sismica mediante curve di fragilità* (Thiella, 2014).

This procedure utilizes the capacity curve and the response spectrum in terms of spectral acceleration and spectral displacement in the ADRS (Acceleration-Displacement Response Spectrum) plan, where the response spectrum becomes the demand spectrum and the capacity curve becomes the capacity spectrum.
The transformation from the response spectrum (acceleration-period) to the demand spectrum ADRS is made with the relation:

$$S_D = \frac{T^2}{4\pi^2} S_A \tag{6.7}$$

- S_D is the x-coordinate of the demand spectrum;
- S_A is the y-coordinate of the response and demand spectrum;
- T is the period to evaluate S_D.

The y-coordinate is the same for both the response and the demand spectra so that the two spectra are comparable. To transform the capacity curves to the ADRS plan are necessary:

- The multiplier α;
- The confident coefficient *FC*, in this thesis is considered *FC*=1.35;
- The participant mass *M**;
- The displacement of the control point.

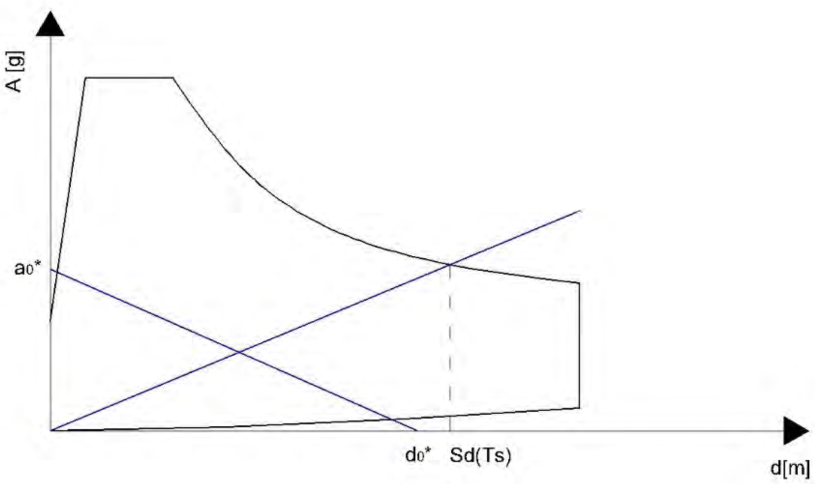

Fig 6.2.3 intersection between the ADRS spectrum and the capacity curves

The intersection between the ADSR spectrum and the capacity curve represents the "performance point". The procedure has to be repeated for all the considered PGA values.

After determining the converging point, the damage is defined as:

$$IM = \frac{d_D}{d_C} \tag{6.8}$$

- d_D is the displacement depending on the seismic demand ($S_{De}(T_s)$);
- d_C is the displacement related to the damage level.

The lognormal distribution is the best model to represent the seismic demand. In the following graph is represented the seismic demand related to one of the considered PGA (Fig 6.2.4).

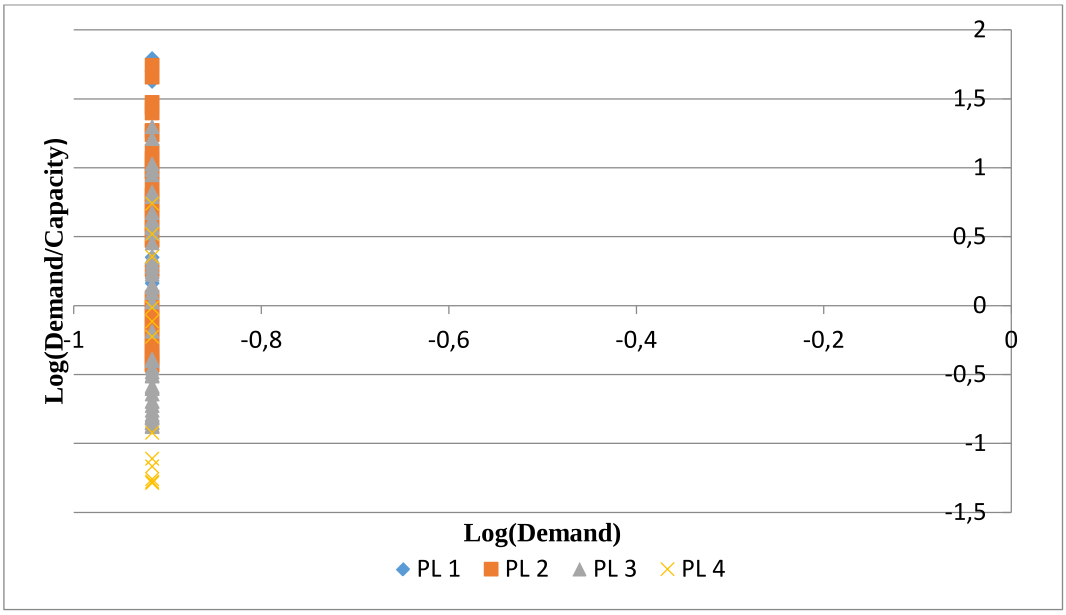

Fig 6.2.4: IM elaboration for a PGA of 0.05

The following law represents the medium demand:

$$S_d = IM^B \cdot e^A \tag{6.9}$$

In the logarithmic plan, the following regression line represents the medium demand:

$$\ln(S_d) = A + B \cdot \ln(IM) \tag{6.10}$$

A and B coefficients are defined from the regression line, considering the standard deviation of the scattergram related to the demand values, the average deviations are referred to the regression line for the considered IM.

The following diagram represents the four regression lines related to the four considered level of damage, the eight scattergram represent the data for each evaluated PGA (Fig 6.2.5).

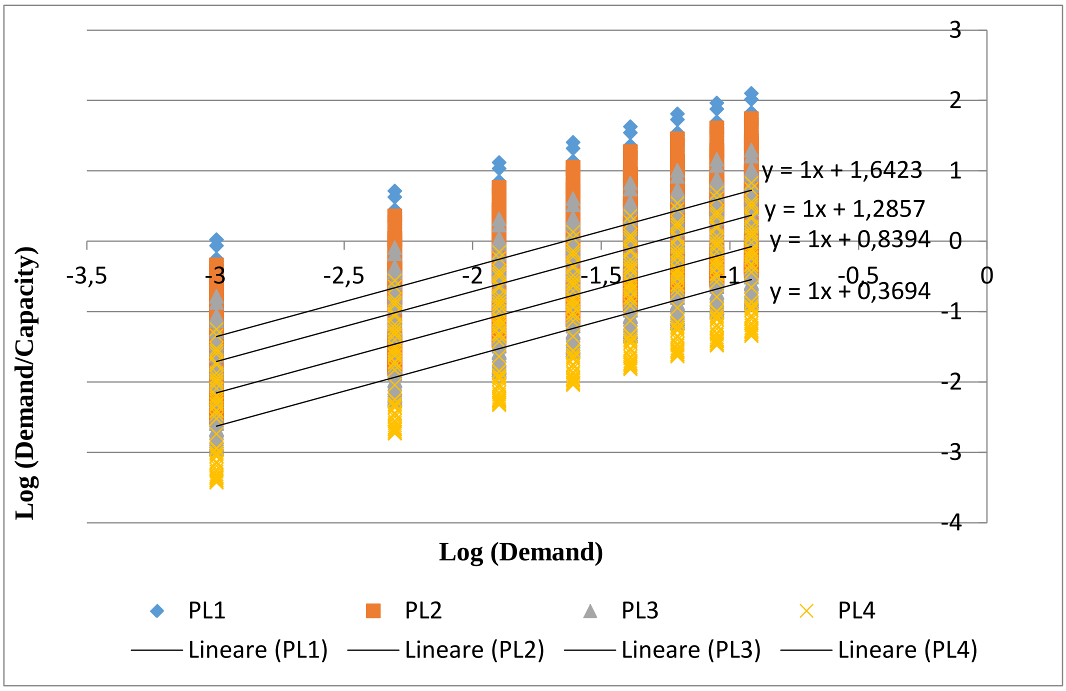

Fig 6.2.5: Four regression line for each damage level considered

Once defined the coefficients A and B and the standard deviation, the fragility curve is represented by a lognormal distribution:

$$P_{f,PL}(a) = P\left[D > d_{LP} \middle| a\right] \tag{6.11}$$

and the exceedance probability is represented by the following formula:

$$f_D = \frac{1}{\sqrt{2\pi} \cdot \varepsilon d} \exp\left[-\frac{1}{2} \left(\frac{\ln d - \lambda}{\varepsilon} \right)^2 \right] \tag{6.12}$$

- $\lambda = A + B \ln(IM)$ is the medium value of the regression line related to a IM(PGA) value;
- ε is the IM(PGA) scattergram.

The following graph represents as an example a fragility curve related to a local mechanism of simple overturning for a masonry building. In the x-axis there are the PGA values and in the y-axis there is the exceedance probability to overflow or to equal the level of damage indicated in the relative curve. The exceedance probability is expressed in percentage.

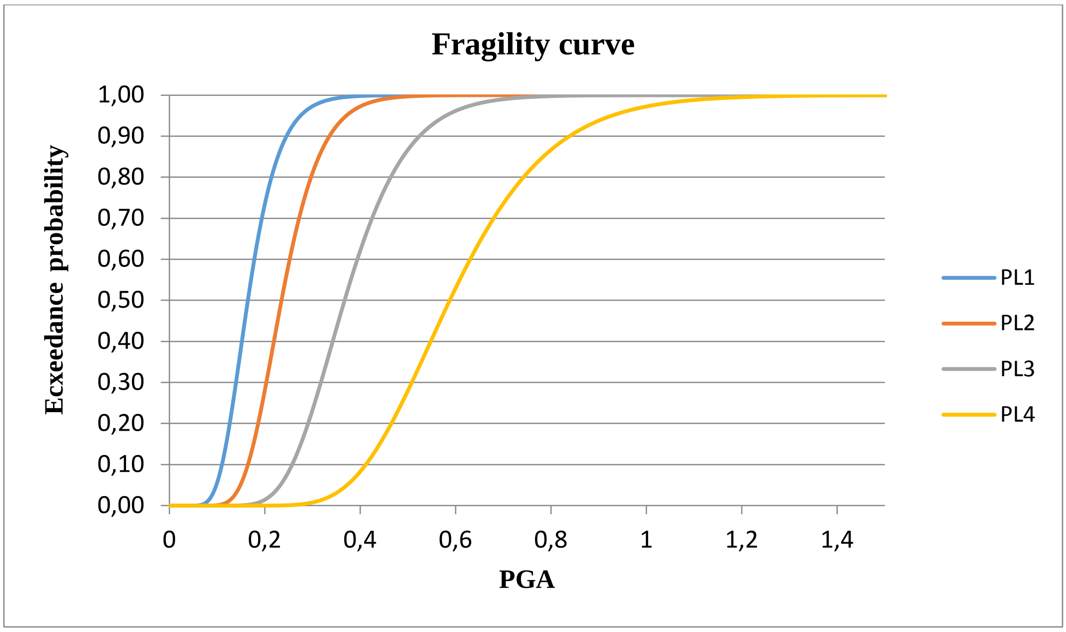

Fig 6.2.6: Fragility curves for the four different Damage Levels

6.3 FRAGILITY CURVES OF THE SIMPLE OVERTURNING MECHANISM

Local mechanisms of collapse, which defines the a_0*, d_0*, a_s* and d_s*values, allow to collect all the data that are necessary to define fragility curves for each building typology . The analyzed typologies are the same of chapter 5.4 and belong to macro-typologies A, E and G. Macro-typologies A and E are both characterized by masonry vertical structures and light horizontal structures with vaults at the ground floor, but the first one has a timber trusses roof and the second one a mixed roof with concrete and timber trusses. The macro-typology G is characterized by masonry vertical structures, moderately heavy horizontal structures and roof composed by concrete and timber trusses. The parameters related to the presence of the vault are indeed not considered in typologies 24, 25 and 26.

Some micro-typologies do not represent structural units of the historical center but have been added afterwards to consider all the possible cases typical of Timisoara buildings. These typologies are written in grey in Tab. 6.3.1, which represents the exceedance probability for a/g=0.2 for each level of damage. It is indeed the aim of this thesis the definition of a preliminary vulnerability assessment of a building, which can be included in one of the defined typologies, after the detection of its main structural characteristics, such as vertical and horizontal structures, roof typology, wall thickness, interstorey height and stories number. The addition of these micro-typologies helps to cover a major number of cases and consequently the possibility to extend the results.

The negative values of the horizontal load multiplier α_0, and consequently of a_0*, d_0* and d_u*, cause the impossibility to define the fragility curve for typology 1. Negative values are indeed unusable to define the fragility curves and the respective parameters have not been considered in the analysis. Due to this problem, just few parameters have been removed from the analysis and usually they correspond to parameters 7 (barrel vault) and 11 (vault rise at 0.5m) for typologies with the vault presence at the ground floor. In typologies 2, 7 and 17, characterized by two stories or two stories with basement, the micro-typologies A and B (wall thickness of 45 and 60cm) have a great number of parameter with negative number for the hinge at level 0. In these cases the fragility curves have been defined using all the values of micro-typologies C and D and the values of the upper floor mechanisms for micro-typologies A and B. For the ground floor of these two micro-typologies the reference mechanism is the vertical bending instead of the simple overturning one.

Each parameter is important for the fragility curve definition, because a singular value can considerably modified the curve trend. The proximity of the real value to the expected value defines the variance and consequently the standard deviation. If the real value and the expected value are close, the standard deviation is small, if they are far, the standard deviation is great. Choosing the parameters suitable to the curve definition is then an important step, that can affect the curve validity. In this analysis all the positive values have been considered, to give a complete representation of the site possibilities.

The exceedance probability percentages for a/g=0.2, which are already defined in Tab. 6.3.1, are shown in Fig. 6.3.1 It is possible to observe that:

- Typologies with the highest exceedance probability of PL1 and PL2 are typologies 2, 7 and 17, characterized by two stories or two stories with basement. Between these three typologies, the one with the highest percentage of exceedance probability of PL3 and PL4 is typology 7, which is characterized by the basement presence;
- Typology 24 is the only one in which the exceedance probability of complete damage (PL4) is 0% for a/g=0.2;
- The percentage values of typologies 4 and 26, both characterized by four stories, are comparable for each level of damage, even if the typology 4 has the horizontal thrust of the vaults at the ground floor and the typology 26 does not have it. The destabilizing force of the vault thrust at the ground floor is indeed less determinant in the vulnerability of the typology with the increment of the stories number;
- In general the presence of the basement and the different roof type do not considerably influence the exceedance probability of the fragility curve. The parameter that most influence the curve behavior is the stories number.

<table>
<tr><th rowspan="2">MACRO-TYPOLOGY</th><th rowspan="2">TYPOLOGY</th><th rowspan="2">MICRO-TYPOLOGY</th><th rowspan="2">STORIES NUMBER</th><th rowspan="2">GROUND FLOOR THICKNESS</th><th colspan="4">EXCEEDENCE PROBABILITY FOR a/g=0.2</th></tr>
<tr><th>PL1</th><th>PL2</th><th>PL3</th><th>PL4</th></tr>
<tr><td rowspan="21">A</td><td rowspan="2">1</td><td>1A</td><td rowspan="2">1</td><td>45</td><td rowspan="2">-</td><td rowspan="2">-</td><td rowspan="2">-</td><td rowspan="2">-</td></tr>
<tr><td>1B</td><td>60</td></tr>
<tr><td rowspan="4">2</td><td>2A</td><td rowspan="4">2</td><td>45</td><td rowspan="4">86%</td><td rowspan="4">69%</td><td rowspan="4">40%</td><td rowspan="4">15%</td></tr>
<tr><td>2B</td><td>60</td></tr>
<tr><td>2C</td><td>75</td></tr>
<tr><td>2D</td><td>90</td></tr>
<tr><td rowspan="4">3</td><td>3A</td><td rowspan="4">3</td><td>45</td><td rowspan="4">76%</td><td rowspan="4">54%</td><td rowspan="4">22%</td><td rowspan="4">5%</td></tr>
<tr><td>3B</td><td>60</td></tr>
<tr><td>3C</td><td>75</td></tr>
<tr><td>3D</td><td>90</td></tr>
<tr><td rowspan="3">4</td><td>4B</td><td rowspan="3">4</td><td>60</td><td rowspan="3">60%</td><td rowspan="3">38%</td><td rowspan="3">16%</td><td rowspan="3">3%</td></tr>
<tr><td>4C</td><td>75</td></tr>
<tr><td>4D</td><td>90</td></tr>
<tr><td rowspan="4">7</td><td>7A</td><td rowspan="4">2+B</td><td>45</td><td rowspan="4">80%</td><td rowspan="4">70%</td><td rowspan="4">52%</td><td rowspan="4">32%</td></tr>
<tr><td>7B</td><td>60</td></tr>
<tr><td>7C</td><td>75</td></tr>
<tr><td>7D</td><td>90</td></tr>
<tr><td rowspan="4">8</td><td>8A</td><td rowspan="4">3+B</td><td>45</td><td rowspan="4">83%</td><td rowspan="4">59%</td><td rowspan="4">24%</td><td rowspan="4">5%</td></tr>
<tr><td>8B</td><td>60</td></tr>
<tr><td>8C</td><td>75</td></tr>
<tr><td>8D</td><td>90</td></tr>
<tr><td rowspan="12">E</td><td rowspan="4">17</td><td>17A</td><td rowspan="4">2</td><td>45</td><td rowspan="4">85%</td><td rowspan="4">66%</td><td rowspan="4">37%</td><td rowspan="4">12%</td></tr>
<tr><td>17B</td><td>60</td></tr>
<tr><td>17C</td><td>75</td></tr>
<tr><td>17D</td><td>90</td></tr>
<tr><td rowspan="4">18</td><td>18A</td><td rowspan="4">3</td><td>45</td><td rowspan="4">76%</td><td rowspan="4">49%</td><td rowspan="4">15%</td><td rowspan="4">2%</td></tr>
<tr><td>18B</td><td>60</td></tr>
<tr><td>18C</td><td>75</td></tr>
<tr><td>18D</td><td>90</td></tr>
<tr><td rowspan="4">20</td><td>20A</td><td rowspan="4">3+B</td><td>45</td><td rowspan="4">79%</td><td rowspan="4">58%</td><td rowspan="4">27%</td><td rowspan="4">8%</td></tr>
<tr><td>20B</td><td>60</td></tr>
<tr><td>20C</td><td>75</td></tr>
<tr><td>20D</td><td>90</td></tr>
<tr><td rowspan="11">G</td><td rowspan="4">24</td><td>24 A</td><td rowspan="4">2</td><td>45</td><td rowspan="4">72%</td><td rowspan="4">32%</td><td rowspan="4">3%</td><td rowspan="4">0%</td></tr>
<tr><td>24B</td><td>60</td></tr>
<tr><td>24C</td><td>75</td></tr>
<tr><td>24D</td><td>90</td></tr>
<tr><td rowspan="4">25</td><td>25A</td><td rowspan="4">3</td><td>45</td><td rowspan="4">64%</td><td rowspan="4">36%</td><td rowspan="4">10%</td><td rowspan="4">1%</td></tr>
<tr><td>25B</td><td>60</td></tr>
<tr><td>25C</td><td>75</td></tr>
<tr><td>25D</td><td>90</td></tr>
<tr><td rowspan="3">26</td><td>26B</td><td rowspan="3">4</td><td>60</td><td rowspan="3">53%</td><td rowspan="3">32%</td><td rowspan="3">13%</td><td rowspan="3">3%</td></tr>
<tr><td>26C</td><td>75</td></tr>
<tr><td>26D</td><td>90</td></tr>
</table>

Tab. 6.3.1: Exceedance probability of each level of damage, referred to a/g=0.2

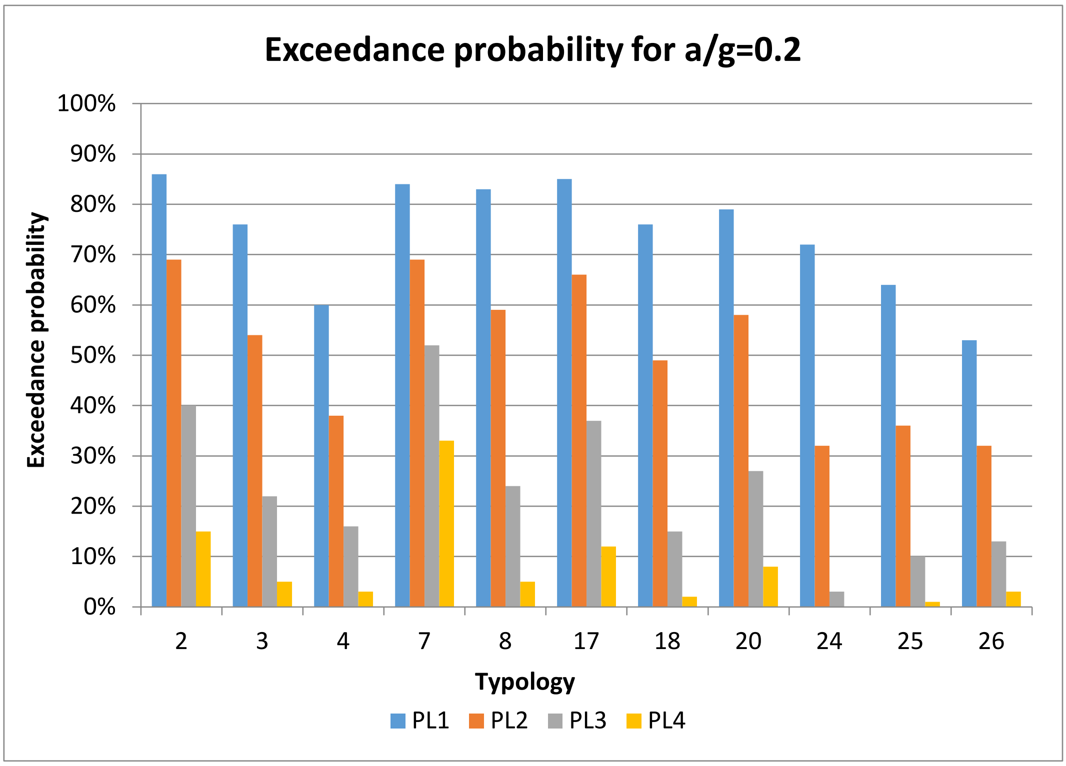

Fig 6.3.1: Exceedance probability for a/g=0.2 in each typology

The following figures represent the fragility curves for the analyzed typologies and the value of a/g=0.2 is indicated. The most vulnerable typologies have a fragility curve that is shifted to the left of the graph, corresponding to lowest PGA values, like typologies 2 (Fig. 6.3.2), 7 (Fig. 6.3.5) and 17 (Fig. 6.3.7). The less vulnerable typologies, like typology 4 (Fig. 6.3.4) and 26 (Fig. 6.3.12) have a fragility curve that is shifted to the right, in correspondence of higher PGA values. The PL4 curve reach the 100% of exceedance probability for lower PGA values in the first case (around 1.5) and for higher values in the second case (around 3).

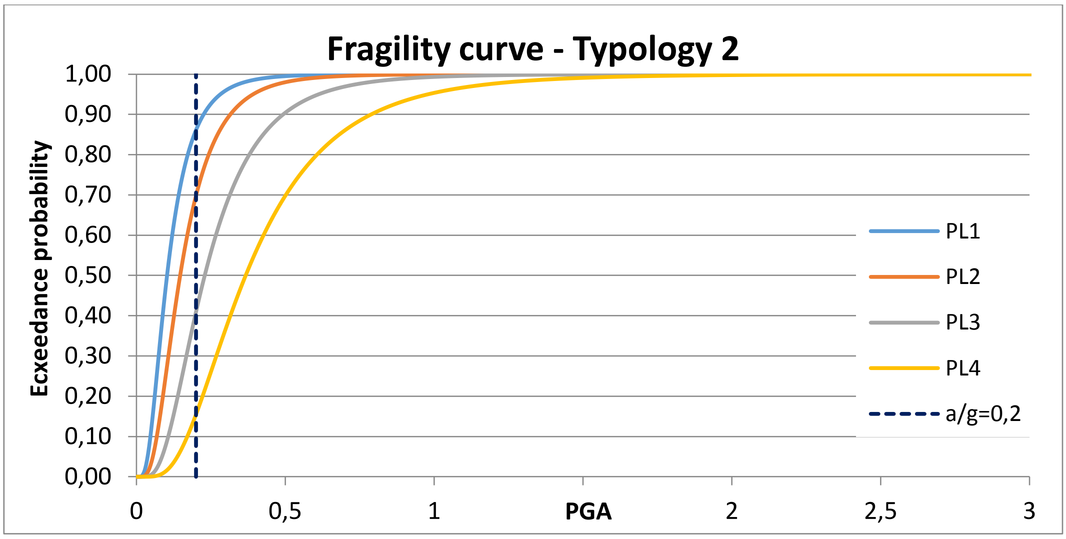

Fig 6.3.2: Fragility curve – Typology 2

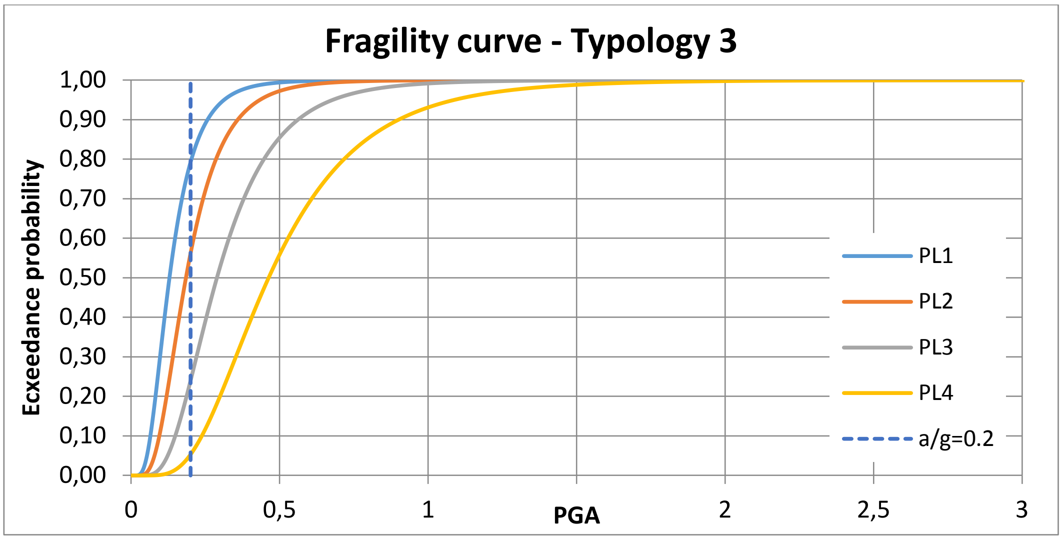

Fig 6.3.3: Fragility curve – Typology 3

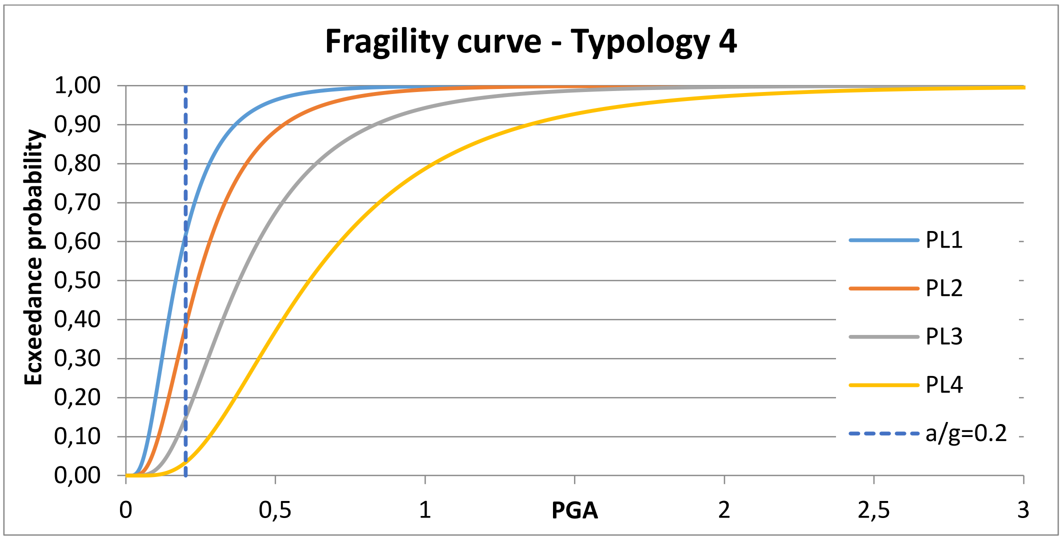

Fig 6.3.4: Fragility curve – Typology 4

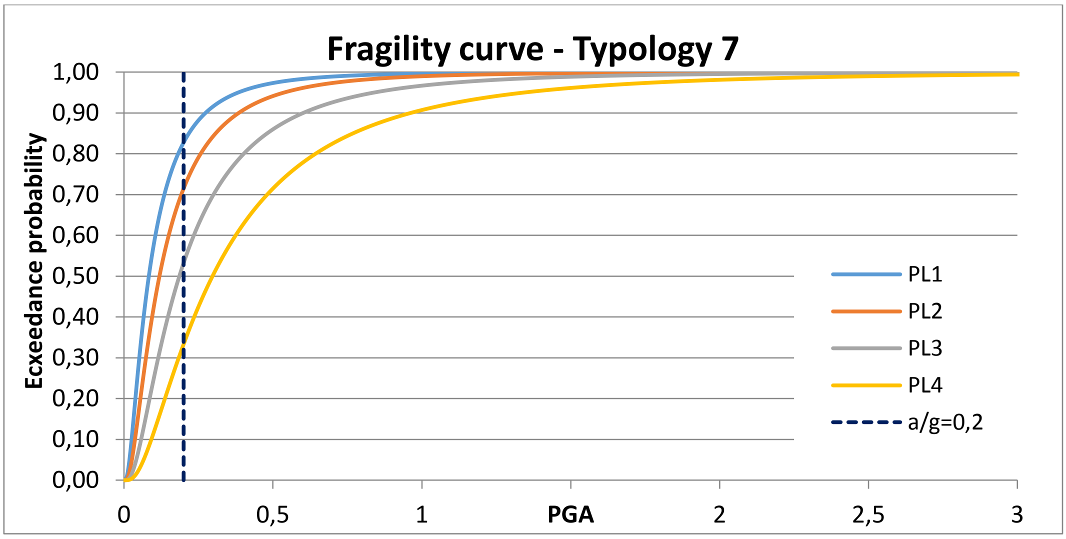

Fig 6.3.5: Fragility curve – Typology 7

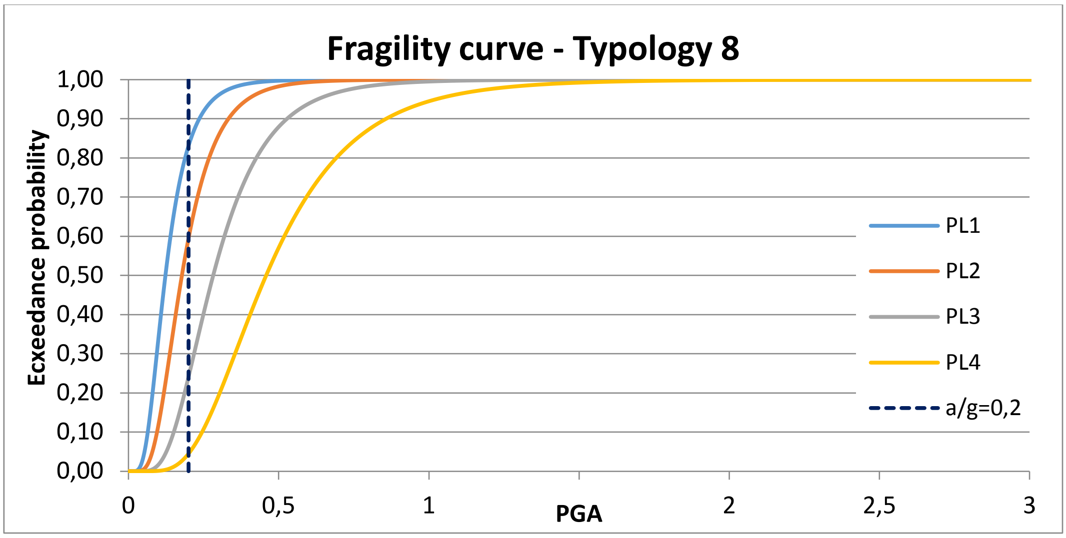

Fig 6.3.6: Fragility curve – Typology 8

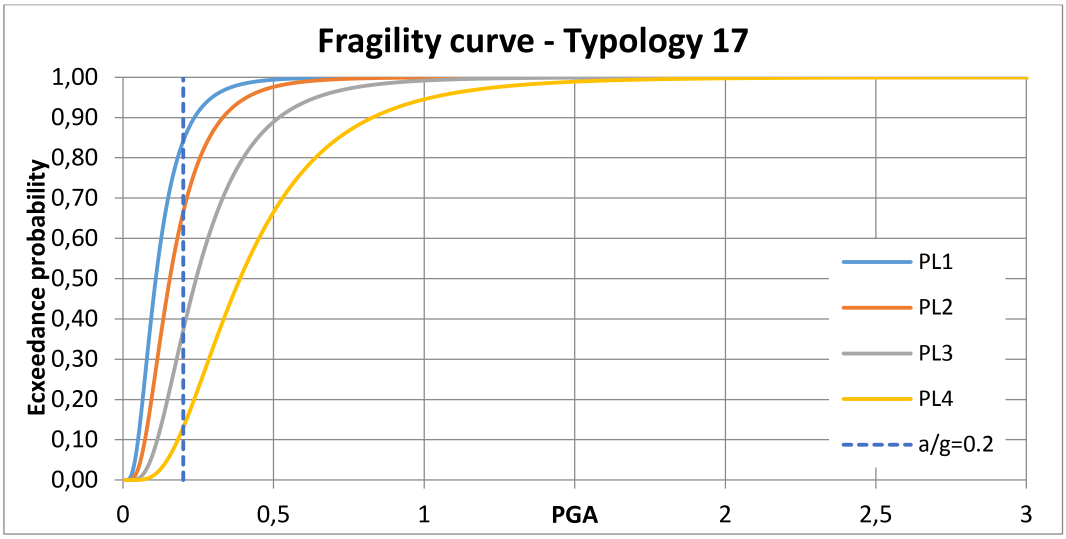

Fig 6.3.7: Fragility curve – Typology 17

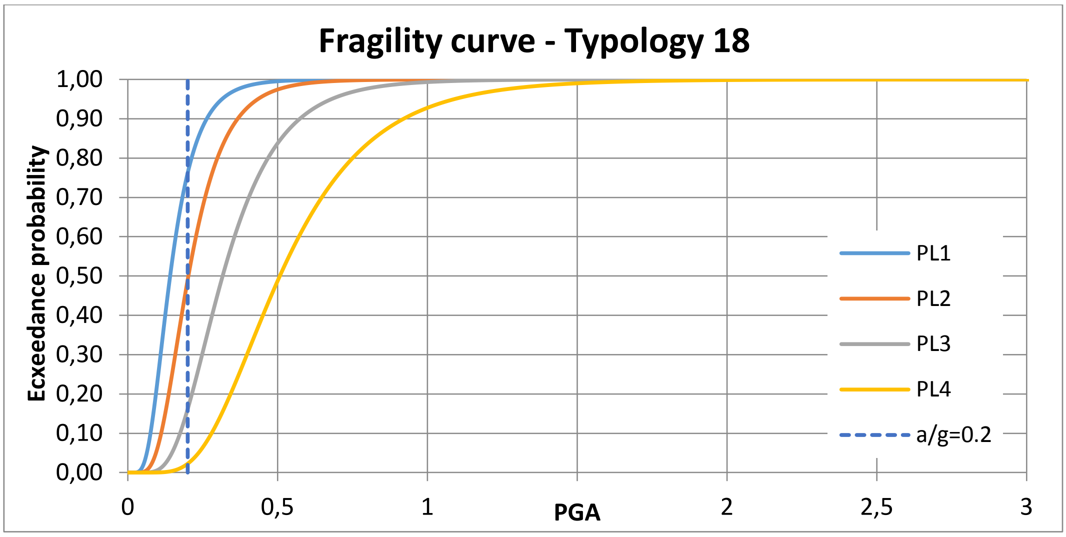

Fig 6.3.8: Fragility curve – Typology 18

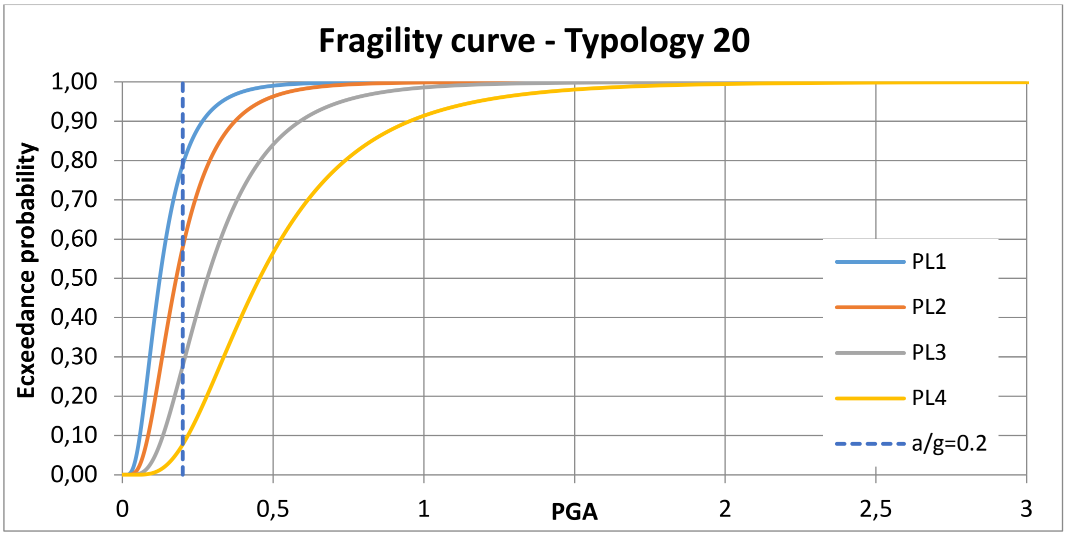

Fig 6.3.9: Fragility curve – Typology 20

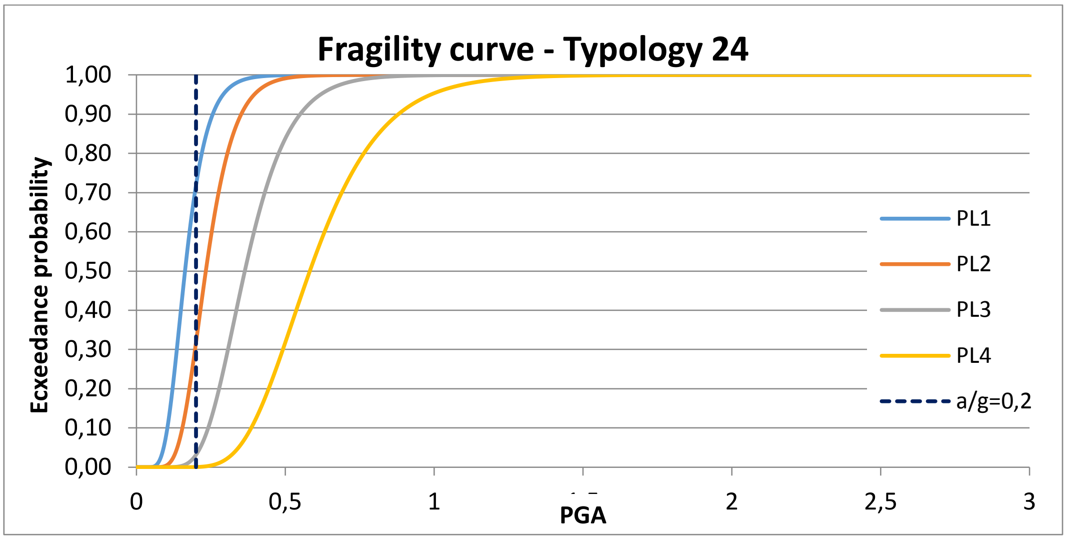

Fig 6.3.10: Fragility curve – Typology 24

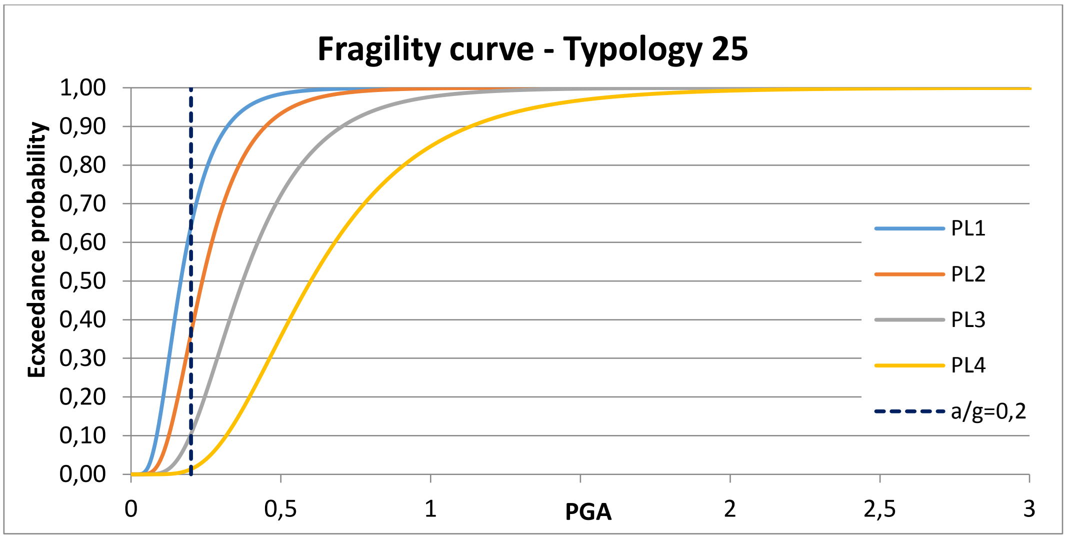

Fig 6.3.11: Fragility curve – Typology 25

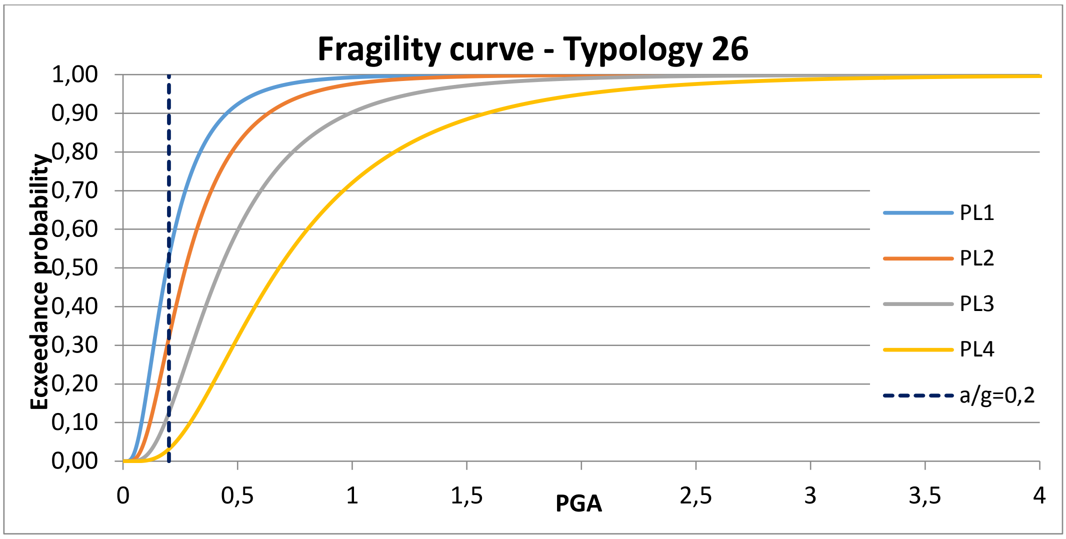

Fig 6.3.12: Fragility curve – Typology 26

6.4 FRAGILITY CURVES OF THE VERTICAL BENDING MECHANISM

The typological study and the analysis of local mechanisms aim to define the fragility curve for each building typology. The analyzed typologies belong all to macro-typologies A and E, which are both characterized by masonry vertical structures and light horizontal structures with brick vaults at the ground floor. The difference between the two macro-typologies is the roof type: the macro-typology is characterized by a timber trusses roof, while the macro-typology E is characterized by a mixed roof with concrete and timber trusses.

The Tab. 6.4.1 represents all the analyzed typologies and the exceedance probability for a/g=0.2 for each level of damage is indicated. As made in chapter 6.3, the parameters characterized by negative values are not considered in the fragility curve definition. Due to this problem, the parameter 7 (barrel vault) has been removed in typologies 3, 8, 17 and 20 and parameters 7 and 11 (vault rise at 0.5) have been removed in typology 2.

The percentage values of Tab. 6.4.1 are represented in Fig. 6.4.1, which gives a visual representation of the exceedance probability for a/g=0.2 for each level of damage. It is possible to observe that:

- Typologies 2 and 17, both characterized by two stories, have the highest percentage of exceedance probability and consequently are the most vulnerable typologies to the vertical bending mechanism. They are the only two typologies which present a probability of exceedance the complete level of collapse (PL4) and the percentage of exceedance probability for PL4 are respectively the 13% and the 12%;
- Typology 4 is the less vulnerable typology and, like typology 7, it shows a probability of exceedance only for PL1 and PL2 levels, never reaching an extensive level of damage;
- In typology 7 the micro-typology A, corresponding to a wall thickness of 45 cm, has not been added afterwards because it can be considered included in micro-typology 2A, which has two stories too and the presence of the basement in typology 7 can be assimilated to the maximum interstorey height at the ground floor of micro-typology 2A (parameter 6);

- The absence of the most vulnerable micro-typology decreases considerably the exceedance probability of typology 7, proving once again the great influence that the wall thickness has in the local mechanism behavior, particularly in the vertical bending one.

<table>
<tr><th rowspan="2">MACRO-TYPOLOGY</th><th rowspan="2">TYPOLOGY</th><th rowspan="2">MICRO-TYPOLOGY</th><th rowspan="2">STORIES NUMBER</th><th rowspan="2">GROUND FLOOR THICKNESS</th><th colspan="4">EXCEEDENCE PROBABILITY FOR a/g=0.2</th></tr>
<tr><th>PL1</th><th>PL2</th><th>PL3</th><th>PL4</th></tr>
<tr><td rowspan="23">A</td><td rowspan="2">1</td><td>1A</td><td rowspan="2">1</td><td>45</td><td rowspan="2">-</td><td rowspan="2">-</td><td rowspan="2">-</td><td rowspan="2">-</td></tr>
<tr><td>1B</td><td>60</td></tr>
<tr><td rowspan="4">2</td><td>2A</td><td rowspan="4">2</td><td>45</td><td rowspan="4">60%</td><td rowspan="4">45%</td><td rowspan="4">29%</td><td rowspan="4">13%</td></tr>
<tr><td>2B</td><td>60</td></tr>
<tr><td>2C</td><td>75</td></tr>
<tr><td>2D</td><td>90</td></tr>
<tr><td rowspan="4">3</td><td>3A</td><td rowspan="4">3</td><td>45</td><td rowspan="4">39%</td><td rowspan="4">18%</td><td rowspan="4">4%</td><td rowspan="4">0%</td></tr>
<tr><td>3B</td><td>60</td></tr>
<tr><td>3C</td><td>75</td></tr>
<tr><td>3D</td><td>90</td></tr>
<tr><td rowspan="3">4</td><td>4B</td><td rowspan="3">4</td><td>60</td><td rowspan="3">21%</td><td rowspan="3">2%</td><td rowspan="3">0%</td><td rowspan="3">0%</td></tr>
<tr><td>4C</td><td>75</td></tr>
<tr><td>4D</td><td>90</td></tr>
<tr><td rowspan="3">7</td><td>7B</td><td rowspan="3">2+B</td><td>60</td><td rowspan="3">29%</td><td rowspan="3">4%</td><td rowspan="3">0%</td><td rowspan="3">0%</td></tr>
<tr><td>7C</td><td>75</td></tr>
<tr><td>7D</td><td>90</td></tr>
<tr><td rowspan="4">8</td><td>8A</td><td rowspan="4">3+B</td><td>45</td><td rowspan="4">30%</td><td rowspan="4">15%</td><td rowspan="4">4%</td><td rowspan="4">0%</td></tr>
<tr><td>8B</td><td>60</td></tr>
<tr><td>8C</td><td>75</td></tr>
<tr><td>8D</td><td>90</td></tr>
<tr><td rowspan="12">E</td><td rowspan="4">17</td><td>17A</td><td rowspan="4">2</td><td>45</td><td rowspan="4">62%</td><td rowspan="4">44%</td><td rowspan="4">26%</td><td rowspan="4">12%</td></tr>
<tr><td>17B</td><td>60</td></tr>
<tr><td>17C</td><td>75</td></tr>
<tr><td>17D</td><td>90</td></tr>
<tr><td rowspan="4">18</td><td>18A</td><td rowspan="4">3</td><td>45</td><td rowspan="4">34%</td><td rowspan="4">18%</td><td rowspan="4">4%</td><td rowspan="4">0%</td></tr>
<tr><td>18B</td><td>60</td></tr>
<tr><td>18C</td><td>75</td></tr>
<tr><td>18D</td><td>90</td></tr>
<tr><td rowspan="4">20</td><td>20A</td><td rowspan="4">3+B</td><td>45</td><td rowspan="4">25%</td><td rowspan="4">13%</td><td rowspan="4">2%</td><td rowspan="4">0%</td></tr>
<tr><td>20B</td><td>60</td></tr>
<tr><td>20C</td><td>75</td></tr>
<tr><td>20D</td><td>90</td></tr>
</table>

Tab. 6.4.1: Exceedance probability of each level of damage, referred to a/g=0.2

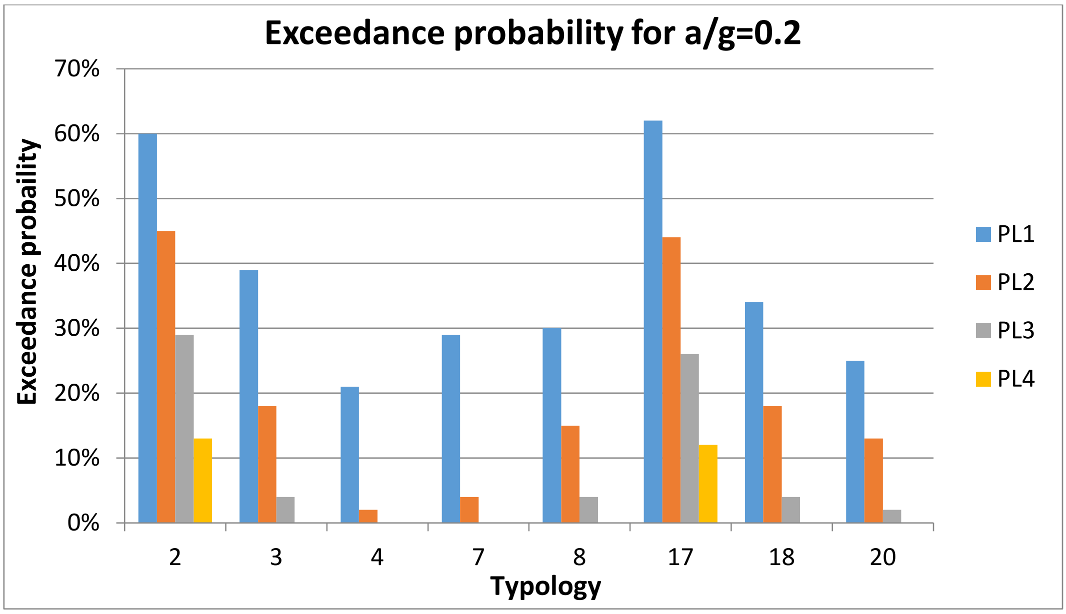

Fig 6.4.1: Exceedance probability for a/g=0.2 in each typology

After the considerably difference in the results of typology 7 fragility curves, the micro-typology 7A with a thickness of 45cm has been added. In Tab. 6.4.2 the new values of the exceedance percentage of typology 7 are shown and the Fig. 6.4.2 represent the new situation for an a/g of 0.2. It is possible to observe that with the addition of the most vulnerable micro-typology, the exceedance probabilities increase, reaching values comparable with micro-typologies 2 and 17.

TYPOLOGY	MICRO-TYPOLOGY	STORIES NUMBER	GROUND FLOOR THICKNESS	EXCEEDENCE PROBABILITY FOR a/g=0.2			
				PL1	PL2	PL3	PL4
7	7A	2+B	45	55%	40%	22%	7%
	7B		60				
	7C		75				
	7D		90				

Tab. 6.4.2: Exceedance probability of each level of damage, for typology 7 referred to a/g=0.2

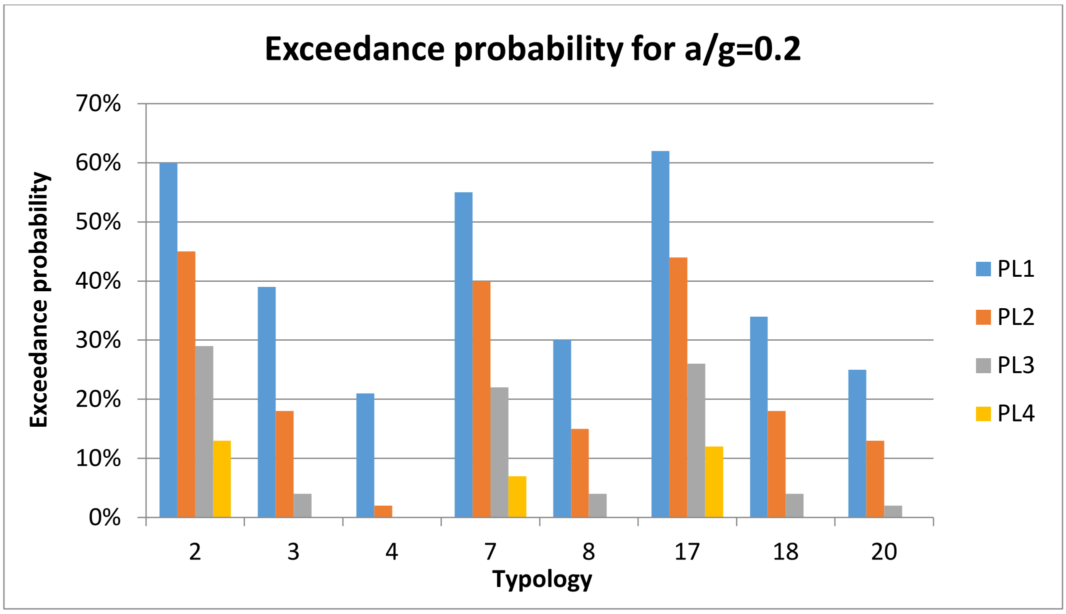

Fig 6.4.2: Exceedance probability for a/g=0.2 in each typology – 7A added

The following figures represent the fragility curves of each analyzed typology and the value of a/g=0.2 is indicated. It is possible to see that the curves of the vertical bending mechanism have a different shape than the simple overturning ones: the curves reach the 100% of exceedance probability of PL4 for higher values of PGA. The vertical bending is in fact a less vulnerable mechanism and it must be subjected to greater values of ground acceleration to activate. The reason why curves, particularly PL4 curves, reach the 100% of exceedance probability for high values of PGA is related to very small values of a micro-typology parameter. In fact very small values of a_0*, d_0* and d_s* cause very high values of the logarithm of the Demand/Capacity ratio. High values that considerably diverge from the regression line cause high values of the standard deviation, which is the reason why the curves need high values of PGA. Low values of a_0*, d_0* and d_s* are common for parameter 7 (barrel vault) and 11 (vault rise at 0.5m).

The fragility curves of typologies 2 (Fig. 6.4.3), 7 (Fig. 6.4.6) and 17 (Fig. 6.4.8) are the most vulnerable ones and they are shifted to the left side of the graph, corresponding to lower values of PGA. They are characterized by two stories and so the stabilizing weight of upper floors is lower compared to buildings with 3 or 4 stories. The curve of typology 4 (Fig. 6.4.5) is in fact shifted to the right, in correspondence to higher values of PGA.

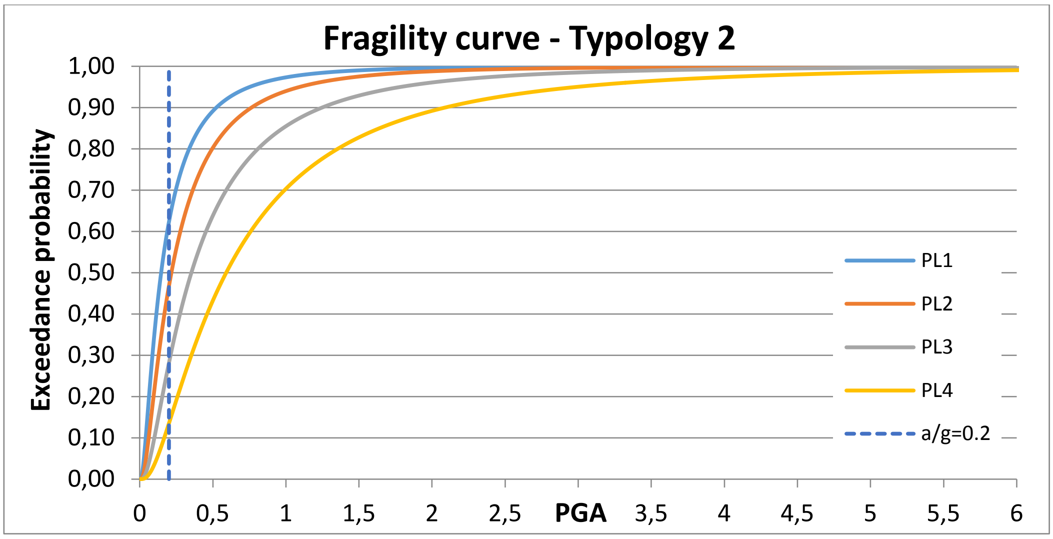

Fig 6.4.3: Fragility curve – Typology 2

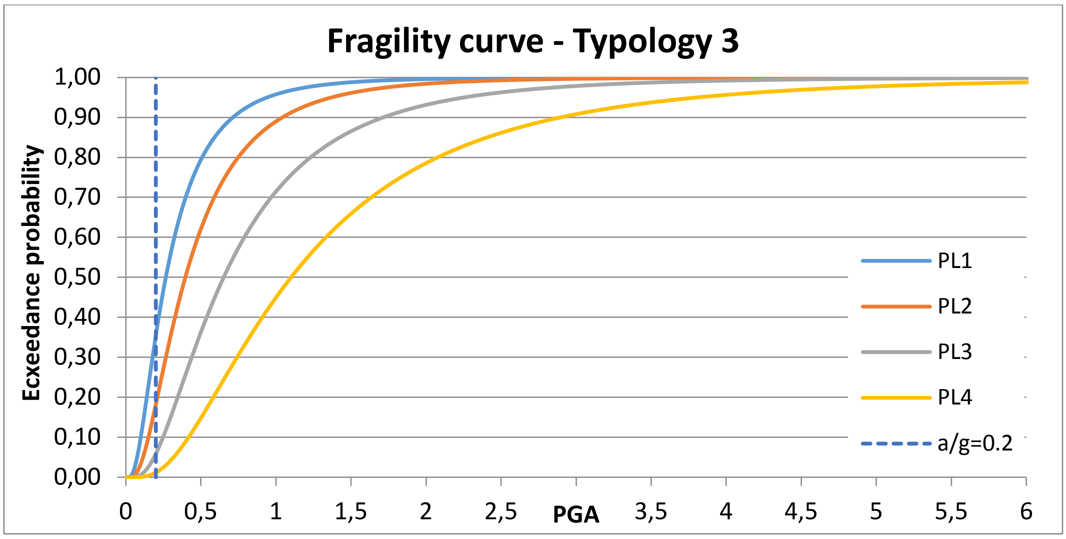

Fig 6.4.4: Fragility curve – Typology 3

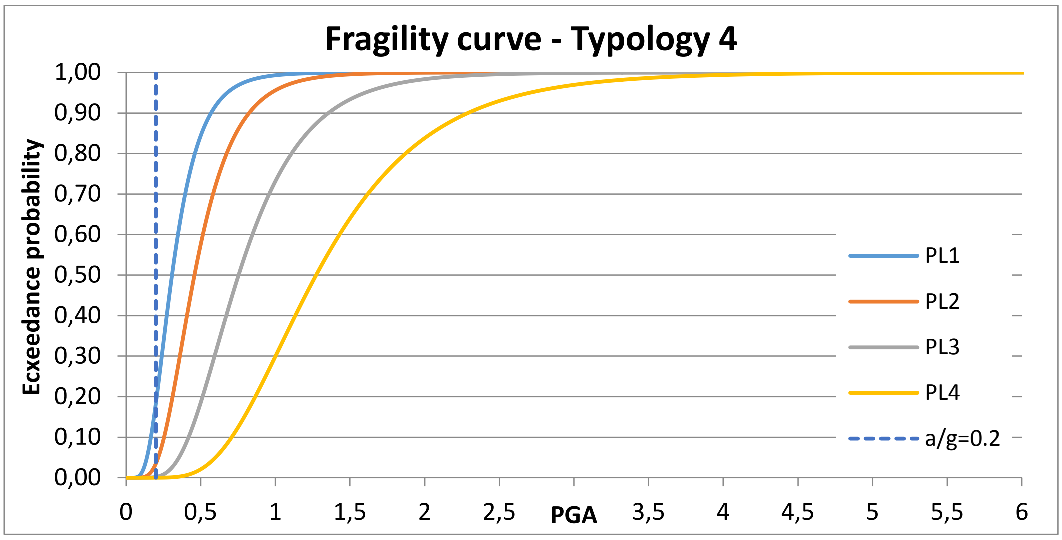

Fig 6.4.5: Fragility curve – Typology 4

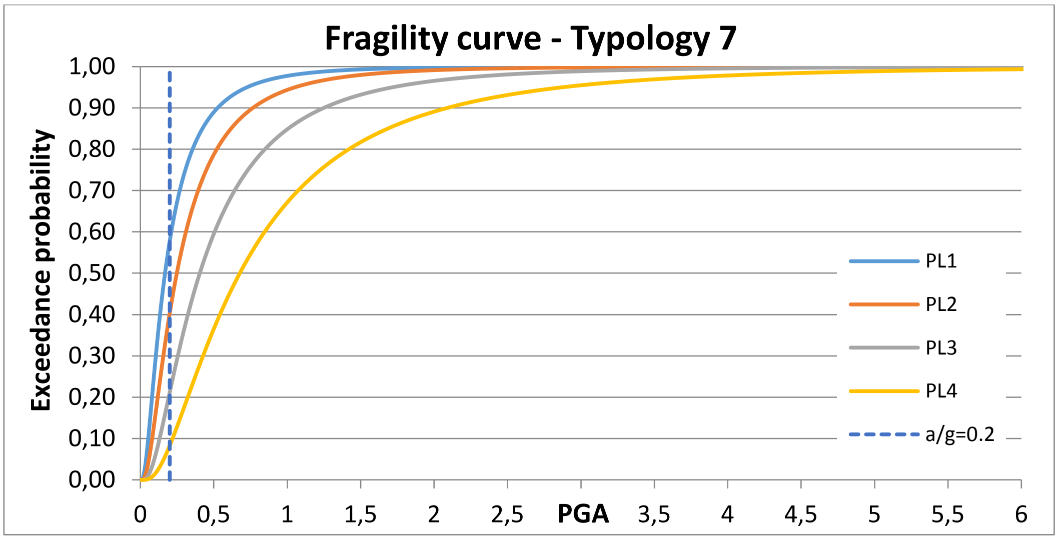

Fig 6.4.6: Fragility curve – Typology 7

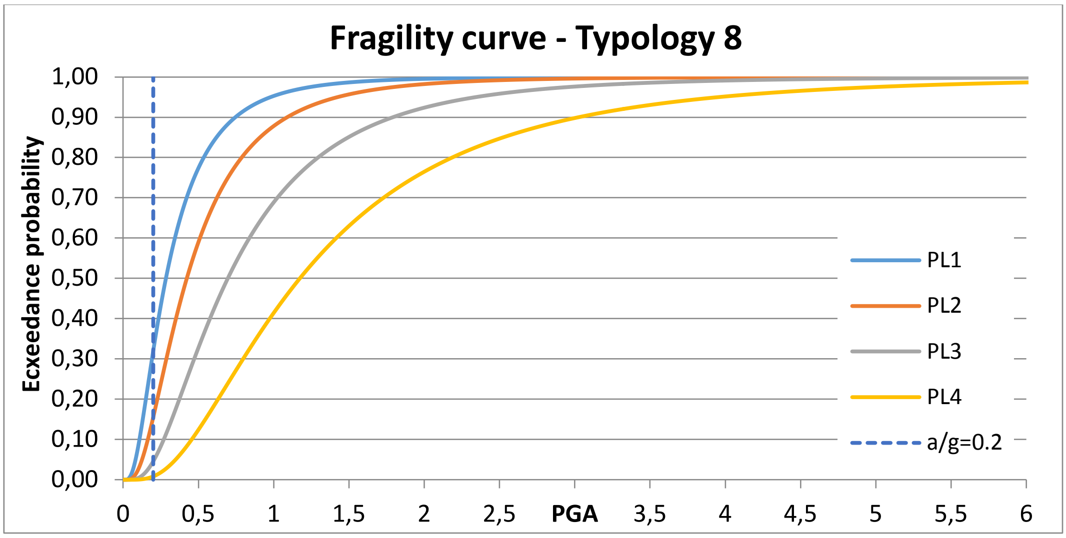

Fig 6.4.7: Fragility curve – Typology 8

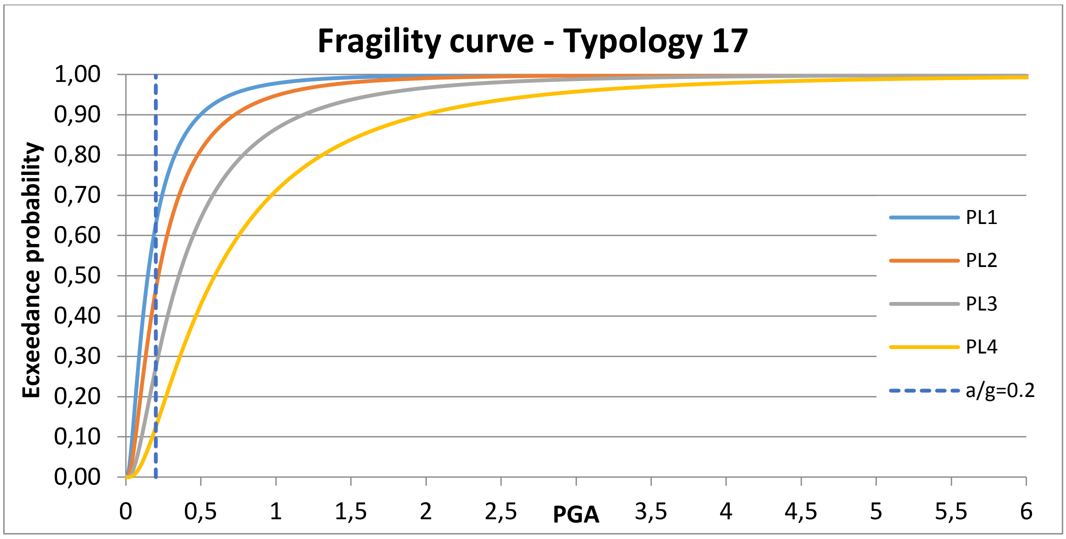

Fig 6.4.8: Fragility curve – Typology 17

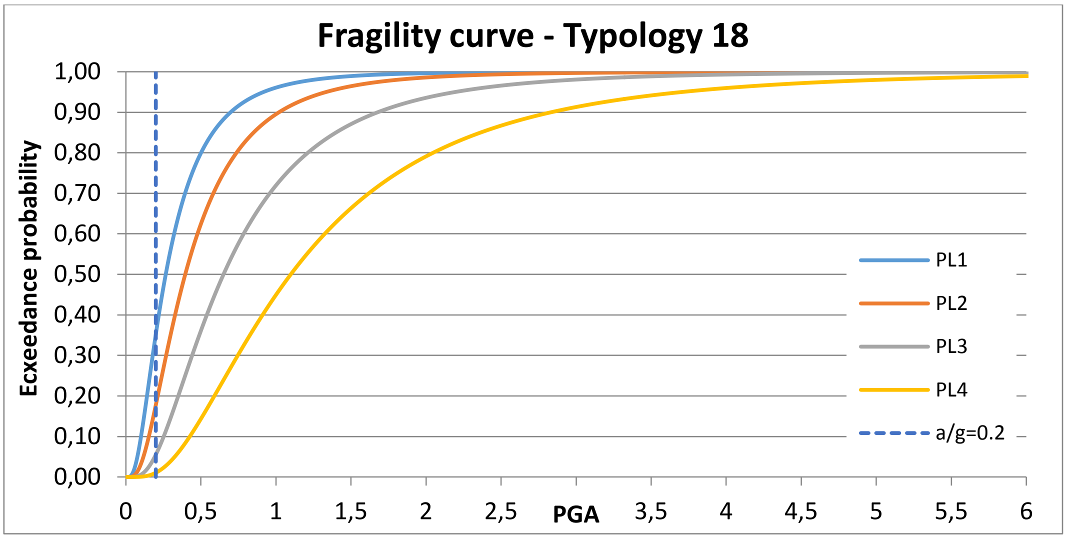

Fig 6.4.9 Fragility curve – Typology 18

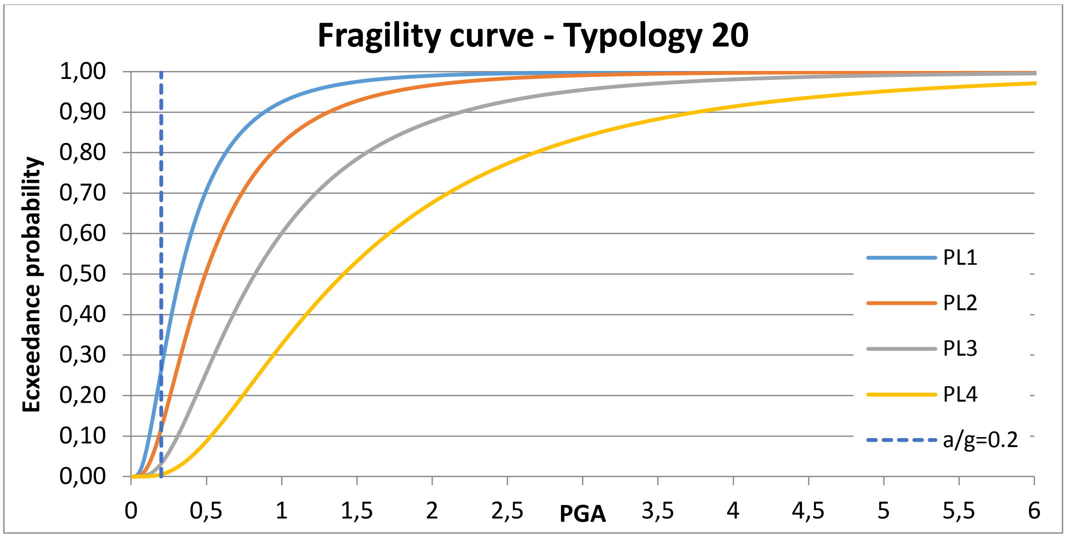

Fig 6.4.10: Fragility curve – Typology 20

6.5 FRAGILITY CURVES OF THE IN PLANE MECHANISM

The determination of typologies that describe all the corner buildings of Timisoara historical center (chapter 2.4.5) and the comparative analysis with the variation of parameters set (chapter 5.6.3) were both necessary to create fragility curves for in-plane mechanism.

The activation of the in-plane mechanism demands greater PGA values than the other two mechanisms and the development of the mechanism is instantaneous: while for the out of plane mechanisms there is a range of about 3 m/s^2 between the 0% and the 100% exceedance probability of the same PL curve, for the in-plane mechanism the range is about 0.5 m/s^2 or even less.

The slope of the curves is steep, differently from the other mechanisms, and represents the rapidity of the mechanism development.

In the following table (Tab 6.5.1) the exceedance probability for each level of damage, referred to the normative acceleration of a/g=0.2, is represented.

ROOF	CLASSES	MICRO CLASSES	BAYS NUMBER	STORIES NUMBER	EXCEEDENCE PROBABILITY PL1	EXCEEDENCE PROBABILITY PL2	EXCEEDENCE PROBABILITY PL3	EXCEEDENCE PROBABILITY PL4	a/g FOR DL4=1
CONCRETE AND TIMBER TRUSSES	I	a	2	2	0%	0%	0%	0%	3,305
		b	5						
		c	7						
		d	11						
	II	a	3	3	8%	0%	0%	0%	1,345
		b	6						
		c	8						
		d	10						
		e	13						
TIMBER TRUSSES	V	a	5	2	0%	0%	0%	0%	1,91 1,92
		b	7						
		c	9						
		d	11						
	VI	a	5	2+B	0%	0%	0%	0 %	2,23
		b	7						
		c	9						
	VII	a	4	3	0,8%	0%	0%	0%	1,265
		b	6						
		c	8						
		d	9						
		e	11						
	VIII	a	8	3+B	26%	0%	0%	0%	0,985
		b	12						
		c	15						
	IX	a	5	4	3%	0%	0%	0%	1,17
		b	7						
		c	10						

Tab. 6.5.1: Exceedance probability of each level of damage, referred to a/g=0.2

The exceedance probability of all the levels of damage, referred to a/g= 0.2, is very low, in particular the levels of damage PL2, PL3 and PL4 never exceed the 0%. The PL1 level (minor damage) is the only one which exceeds the 0% and the higher percentage is referred to the VIII class with a 26% exceedance probability.

Fragility curves for in-plane mechanism are carried on for 7 of the 9 classes exposed in chapter 2.4.4, because classes III and IV, respectively having four stories and four stories plus basement, are both represented by one structural unit. These two units are considered comparable with the class II and these three classes are represented by the same fragility curve (Fig 6.5.2).

The class V is characterized by two fragility curves because of a double façade configuration: in the first one the windows are located at 1 m from the floor (Fig 6.5.3) and in the second one the windows are located at the height of the analyzed floor (Fig 6.5.4), as already exposed in chapter 2.4.4. The two fragility curves are very similar: the percentage of exceedance probability is the same for all the levels of damage and the PGA value to which corresponds PL4=1 is very close, 1.91 for the first and 1.92 for the second. From these results it is possible to deduce that the difference of 1 m in the window height from the ground level is not a influent parameter.

The classes analyzed are divided in two groups characterized by the roof typology: concrete and timber trusses (class I and II) and timber trusses (class V, VI, VII, VIII, IX). The classes are characterized by the stories number and divided in micro-classes corresponding to the different façade length observed in situ, defined by the bays number. The analyzed parameters and their variations considered for the creation of the fragility curves are :

- Last plan height;
- Height from the ground;
- Window dimensions (bxh);
- Bays number.

Between the two classes characterized by the timber trusses roof, the class I (two stories) and class II (three stories), the most vulnerable is the second one, which has the 8% of exceedance probability to overflow the PL1 and it reaches the collapse (PL4) for a PGA of 1.345. The class I reaches the collapse with PGA of 3.305 and its fragility curve (Fig 6.5.1) is characterized by a lower slope and involves a larger range of PGA.

The five classes of the timber trusses roof group are characterized by similar fragility curves to each other, with a strong slope. The most vulnerable class is the

VII class, characterized by three stories and the basement (Fig. 6.5.7). In general the classes vulnerability growths with the stories number.

From the comparison of the two groups (characterized by the roof type), it is possible to notice that the fragility curves of the building with two stories (Fig 6.5.1, Fig. 6.5.3, Fig. 6.5.4) are similar, but the ones with the timber trusses roof are more sloping and more vulnerable, due to the lower stabilizing weight of the roof.

The fragility curves of the classes characterized by three stories (Fig. 6.5.2, Fig 6.5.5) are quite similar. The roof weight in case of buildings with more than two stories is then not an influent parameter.

In the following figures the fragility curves of all the analyzed classes are shown.

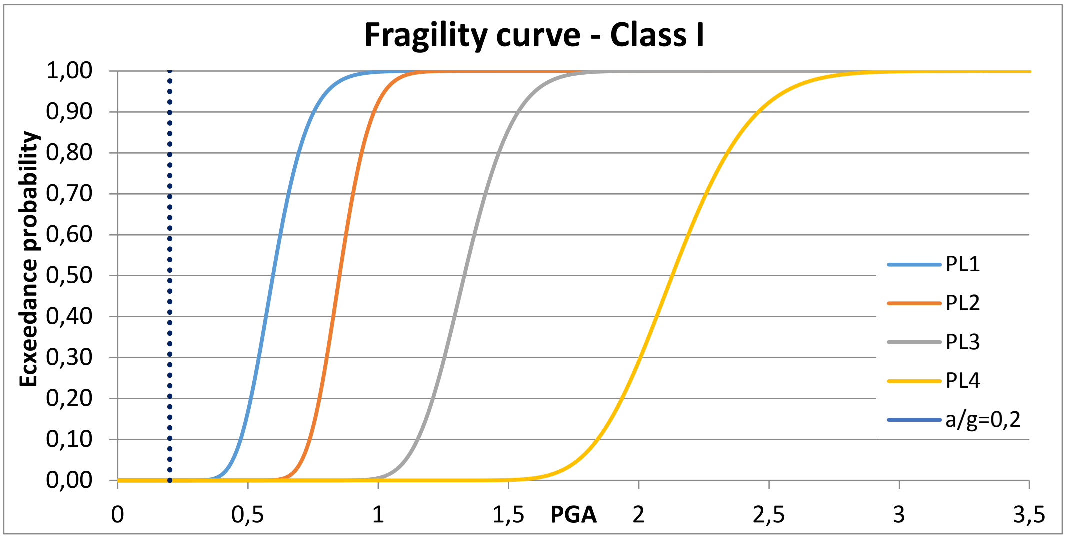

Fig. 6.5.1: Fragility curve for the I class

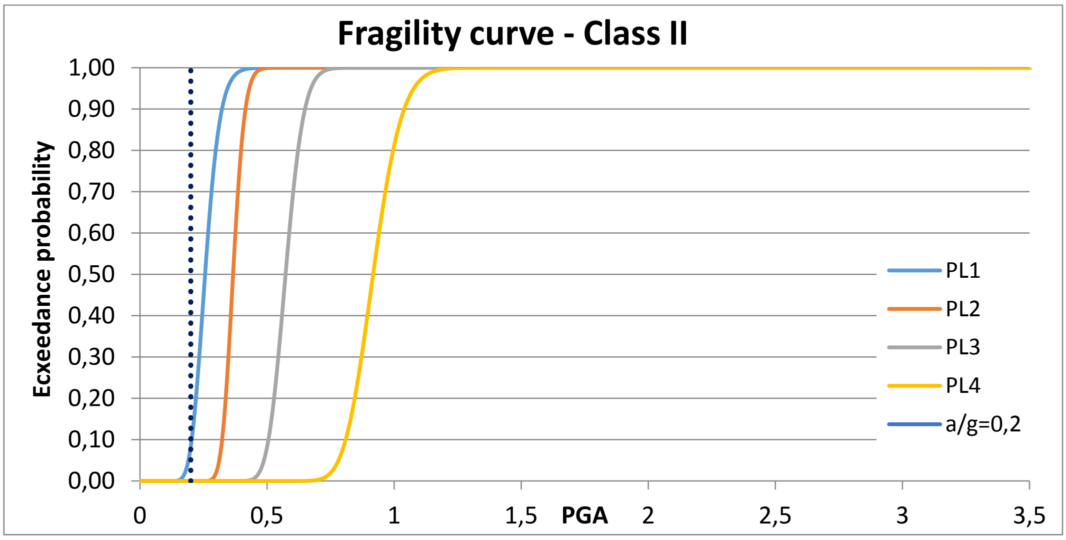

Fig. 6.5.2: Fragility curve for the II class

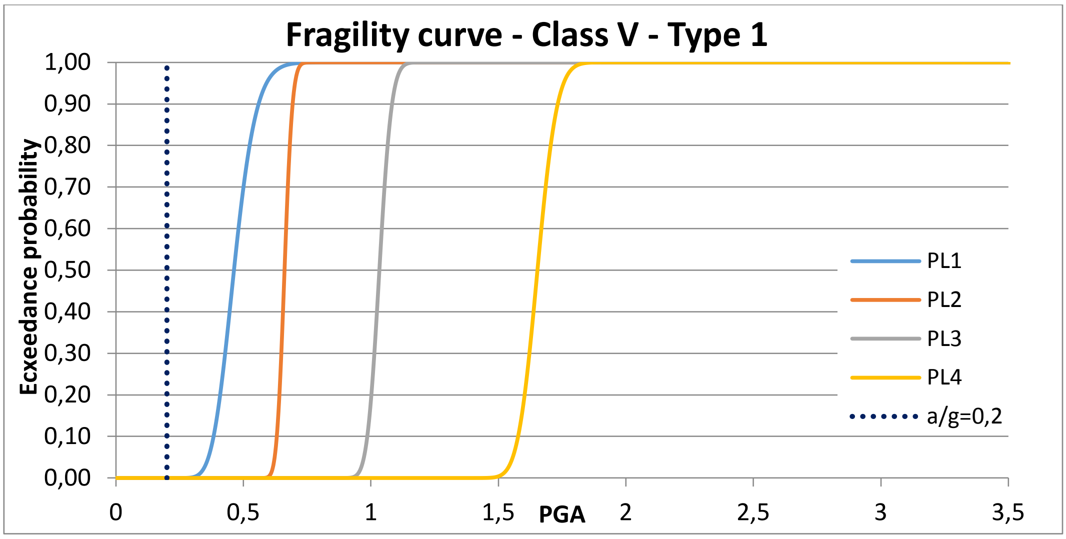

Fig. 6.5.3: Fragility curve for the V class _first version

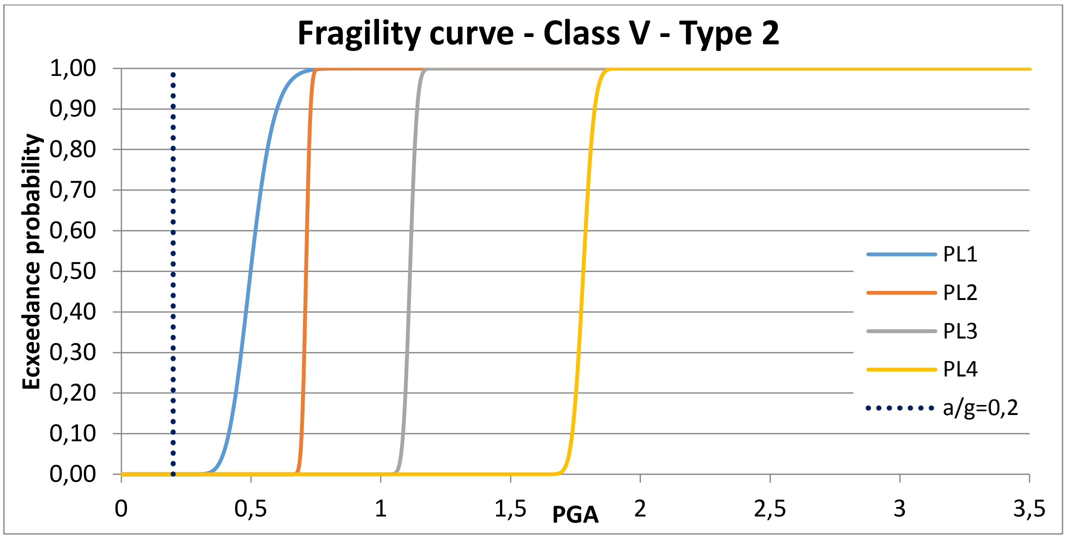

Fig. 6.5.4: Fragility curve for the V class _ second version

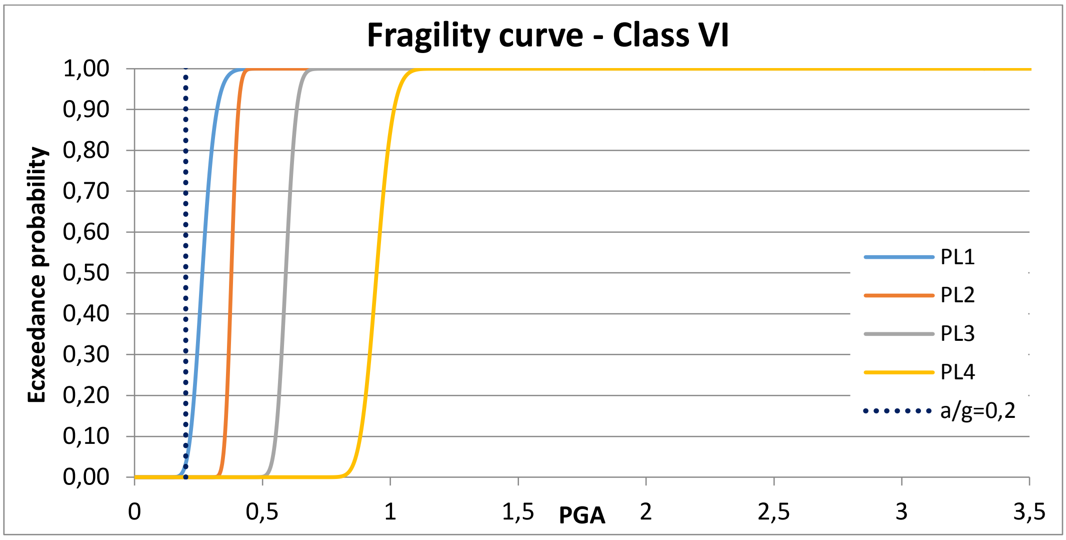

Fig. 6.5.5: Fragility curve for the VI class

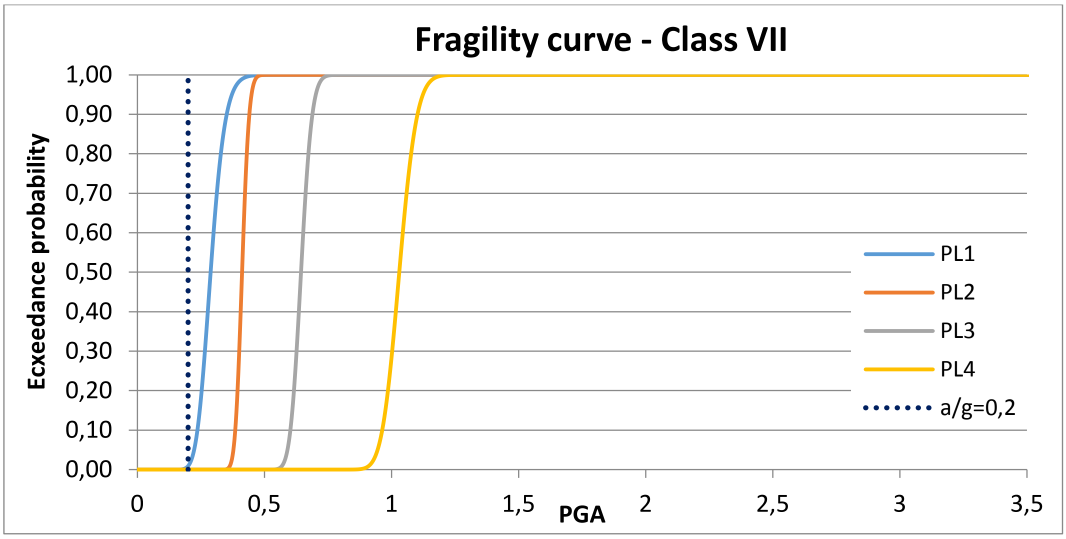

Fig. 6.5.6: Fragility curve for the VII class

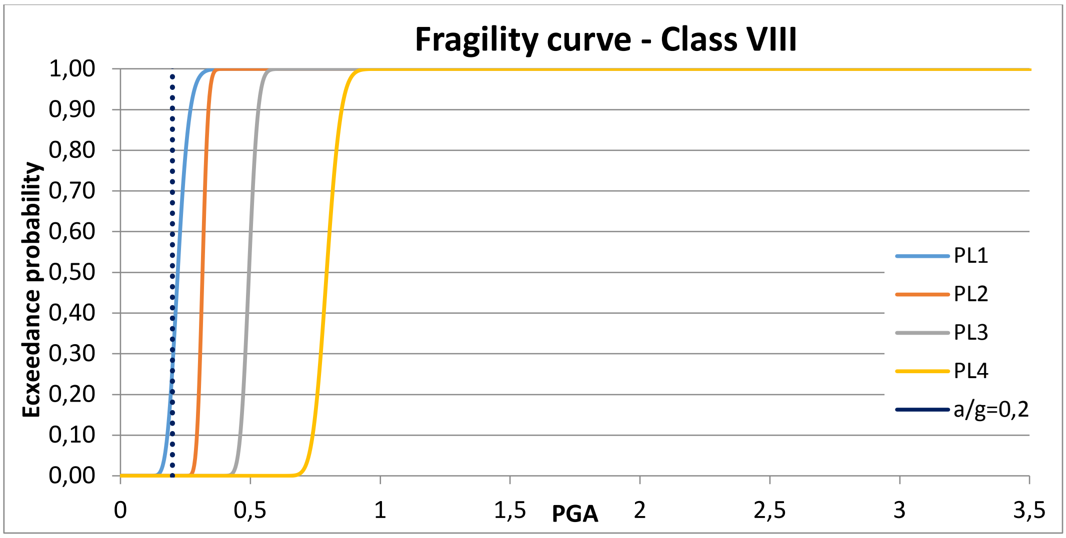

Fig. 6.5.7: Fragility curve for the VIII class

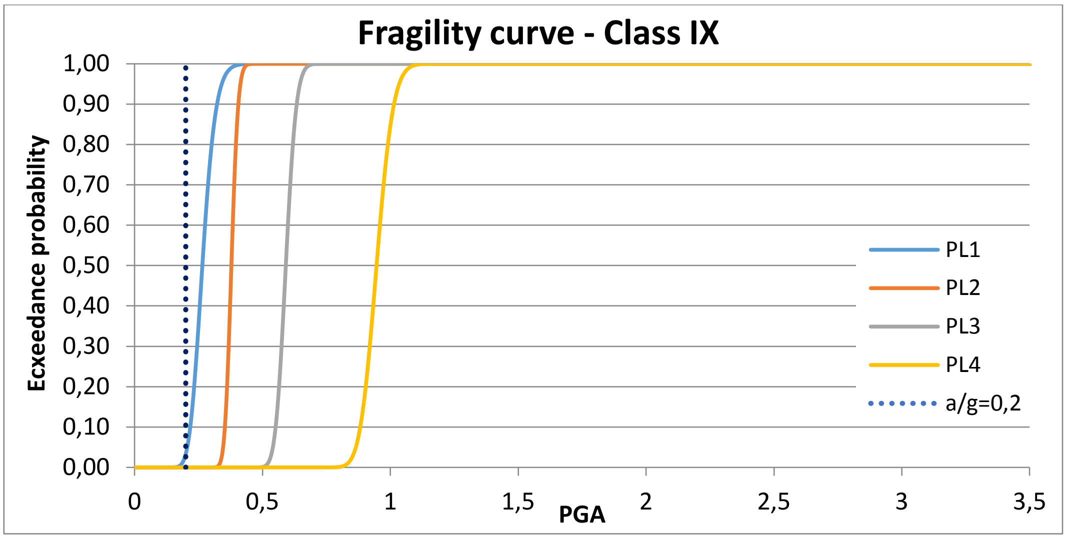

Fig. 6.5.8: Fragility curve for the IX class

6.6 VULNERABILITY ASSESMENT MAP

The results of the fragility curves have been graphically resumed in the annex B. In the following figures the exceedance probability is referred to the simple overturning, the vertical bending and the in plane mechanisms (Fig.6.6.2, Fig.6.6.3, Fig.6.6.4), in correspondence of the a ground acceleration of a/g=0.2 for the city of Timisoara.

The following color scale (Fig 6.6.1) represents the exceedance probability in reference to the considered level of damage: the red represents the maximum possibility of exceeding the considered level of damage (>90%) while the pink the minimum possibility (<10%)

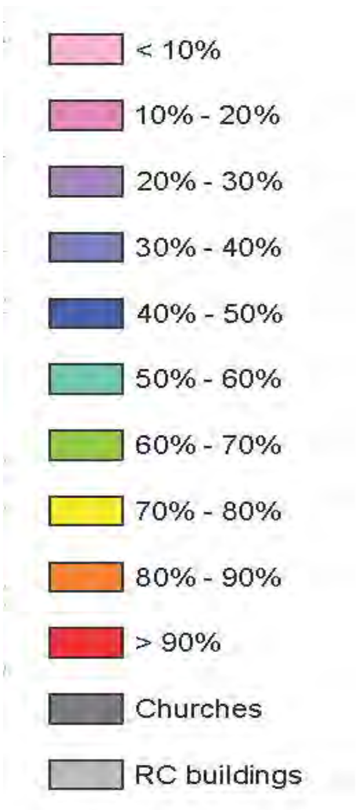

Fig. 6.6.1: Exceedance probability scale

Fig. 6.6.2: Vulnerability assessment of PL2 - simple overturning mechanism

In Fig 6.6.2 the exceedance probability for level of damage PL2 (moderate damage) is represented, referring to the simple overturning mechanism. As already resumed in Tab. 6.3.1 the biggest part of the typologies is characterized by a significant percentage of possibility of exceeding the level of moderate damage. Typologies 2, 3, 7, 8, 18 and 20 are characterized by a percentage between 50-70%, while typologies 4, 8, 24, 25 and 26 between 30-50%. All the typologies, except the "unicum" units, are represented in the map, confirming the results of the local mechanism analysis: the simple overturning mechanism is the most vulnerable one and it is more probable in case of bad connections between horizontal structures and walls.

Fig. 6.6.3: Vulnerability assessment of PL2 - vertical bending mechanism

In Fig. 6.6.3 the exceedance probability referred to the level of damage PL2 is represented. Many typologies are involved by this mechanism and almost the entire historical center is included. The exceedance percentages are characterized by lower values of the simple overturning mechanism and they are ranging between the 10% and the 30 % , with some peaks close to the 40%.

Fig. 6.6.4: Vulnerability assessment of PL1 - in-plane mechanism

Finally the in plane mechanism considers only the corner units and the exceedance probability is referred to the level PL1 (minor damage), because for the considered a/g= 0.2, is the only level of damage different from zero. The higher value corresponds to class VIII, characterized by a exceedance percentage of 26%, while classes II,VII, IX have values that correspond to less than 10% .The activation of the in plane mechanism requires high ground accelerations and the map confirms that the in-plane mechanism is the less vulnerable one, even less probable than the vertical bending mechanism.

In the following table (Tab. 6.6.1) all the typologies with their exceedance probability are resumed. For each percentage the number of the interested structural units is indicated for the three local mechanisms, referring to the defined level of damage.

EXCEEDANCE PROBABILITY	SIMPLE OVERTURNING (PL2)		VERTICAL BENDING (PL2)		IN-PLANE MECHNISM (PL1)	
	Interested typologies	Number of interested US	Interested typologies	Number of interested US	Interested typologies	Number of interested US
<10%	-	0	4,	11	II, VII, IX	36
10-20%	-	0	3, 8, 18, 20	70	-	0
20-30%	-	0	-	0	VIII	12
30-40%	4, 24, 25, 26	24	-	0	-	0
40-50%	18	4	2, 7, 17	61	-	0
50-60%	3, 8, 20	66	-	0	-	0
60-70%	2, 17	39	-	0	-	0
70-80%	7	22	-	0	-	0
80-90%	-	0	-	0	-	0
>90%	-	0	-	0	-	0

Tab. 6.6.1: Exceedance probability for each analyzed mechanism – interested US

CONCLUSIONS

This thesis aims to define the seismic vulnerability assessment of the historical clustered buildings of the city of Timisoara, starting from the rapid survey on a urban scale and the most easily relived building characteristics.

The analysis is based on a typological study of the data collected during the on-site activity and on the information of constructive techniques and building peculiarities collected during the preliminary phase. The typologies are defined referring to the main building characteristics, such as vertical and horizontal structures, roof typology, wall thickness, interstorey height and stories number.

The typologies definition is the starting point for the analyses both applying Vulnus methodology and the local mechanisms of collapse, because of its relevance in the definition of buildings most common structures, dimensions and geometries. On a urban scale it is not possible to analyze in detail each construction and some peculiar characteristics cannot be detected. It is determinant for the analysis validity that the typological simplifications do not excessively deviate the results from the real case. For this reason the comparison of results obtained with different levels of information is carried out, both in Vulnus methodology and in the local mechanism analysis. The results comparison brought out the general validity of the survey assumptions and consequently of the typological analysis. In fact the results usually overestimate the seismic vulnerability, due to preliminary choices made in favor of safety. It must be underlined that a rapid and external survey cannot consider every structural aspect of a building and particular structural combinations, which can considerably increase the vulnerability assessment, were not surveyed. It is indeed important to underline the preliminary character of the typological vulnerability assessment exposed in this thesis.

In the analysis of the local mechanisms the varying parameters, used to carry out the process, are chosen in order to represent all the surveyed possible cases and to consider the uncertainties of the rapid survey, particularly about the horizontal structure typology, which are difficult to detect with an external survey. The parameter that more influences both the local mechanisms of collapse is the wall thickness, even if the vertical bending is more sensible to its variations than the simple overturning. The vertical bending mechanisms has been analyzed only for the typologies with brick vaults at the ground floor, hypothesizing the hinge formation in correspondence of the vault shoulder. The vault presence is in fact an element that increases the possibility of the mechanism activation and it has been detected always at the ground floor of the analyzed typologies.

This mechanism results to be less vulnerable than the simple overturning one, as typical of clustered masonry buildings. The parametric analysis adopted in the methodology can be extended to other useful application, such as the definition of iso-acceleration curves, which proved the spectral acceleration in relation of the main geometrical characteristics of the construction.

For each building typology the fragility curve is defined, for both the analyzed local mechanisms. In both cases the fragility curves result to be more vulnerable in correspondence of two stories typologies, decreasing their vulnerability with the increment of the stories number. In the simple overturning mechanism the probability of exceeding the levels of minor and moderate damage, in correspondence of the ground acceleration indicated in the Romanian code, often exceeds respectively the 70% and the 50% while for the level of complete damage the percentage of exceedance probability has a range between 5 and 15%. The percentages for the vertical bending mechanisms are considerably lower and for the same ground acceleration many typologies do not even develop the level of complete damage.

In conclusion the results of this thesis allow the definition of a preliminary assessment of a building vulnerability to the analyzed out of plane mechanisms after the detection of its main structural characteristics, such as vertical and horizontal structures, roof typology, wall thickness, interstorey height and stories number. This assessment is immediate and it can be identified on-site after a first rapid survey of the construction. It allows to define the building inclination to a level of damage and consequently the urgency of an eventual intervention. It is therefore an important device to develop prevention strategies to protect the existing cultural heritage as well as the human life.

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