Guidelines of the assessment procedure for earthquake vulnerability in HR test site

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Summary

Abstract

The protection of built heritage in historic cities located in seismically active areas is of great importance for the safety of inhabitants. Systematic care and planning are necessary to detect the seismic vulnerability of buildings, in order to determine priorities in rehabilitation projects and to continuously provide funds for the reconstruction of the buildings. In this study, the methodology for seismic vulnerability assessment of the buildings, that are representative for the towns and settlements located along the Croatian side of Adriatic coast, has been developed. The procedure for vulnerability assessment has been demonstrated in the Croatian test site Kaštel Kambelovac through an approach based on the calculation of vulnerability indexes. The chosen test site consists of a historical core with stone masonry buildings built between the 15th and 19th centuries and of the parts outside the historical core with newer buildings dating from the beginning of the 20th century to the present day. Later buildings were constructed in different periods according to different technical regulations before 1948, from 1949 to 1964, from 1964 to 1982 and from 1982 to 2005. The most modern buildings have been built from 2005 onwards. The main structural and material characteristics, as well as design rules of each construction periods are presented.

The seismic vulnerability method has been derived from the Italian GNDT approach, with some modifications resulting from the specificity of the buildings in the investigated area, and expressed with seismic vulnerability indexes. A new damage-vulnerability-peak ground acceleration relation has been developed using the vulnerability indexes and the yield and collapse accelerations of buildings obtained through non-linear static analysis. Developed procedure presents a basis for calculation of the seismic vulnerability index, damage index, critical accelerations and index of seismic risk for the yield, significant damage and collapse states of the buildings at the test site.

1 Introduction

1.1 A short description of Activity 3.3 “Assessment of climate-unrelated hazards exposure in urban and coastal areas (seismic action)”

Many countries with moderate to high seismic risks, including Croatia, have old towns with stone or brick masonry buildings, built long before the approval of the first seismic regulations. Some of these towns are categorized as cultural heritage sites and should be preserved for future generations. Strong earthquakes cause significant damage and failure of such buildings. Rehabilitation requires significant financial resources that cannot be allocated suddenly. Therefore, systematic care and planning are necessary to detect the seismic vulnerability of buildings, in order to determine priorities in regard to their rehabilitation and allocate funds for reconstruction.

Activity 3.3 “Assessment of climate-unrelated hazards exposure in urban and coastal areas (seismic action)” of the PMO-GATE project, among other tasks, aims to develop guidelines of the assessment procedure for earthquake vulnerability, to determine seismic vulnerability indexes for masonry historical buildings located in the HR test site and to develop seismic vulnerability maps for the HR test site. Deliverable 3.3.1 is focused on development of guidelines for the seismic assessment procedure which will be used to determine seismic vulnerability of the buildings based on vulnerability index method and to create seismic vulnerability map of the test site. The guidelines also represent basis for determination of seismic risk of the buildings needed for development of a map of a spatial distribution of the critical zones most prone to seismic risk to give priorities to intervention to authorities and involved parties (a part of the Deliverable 5.1.6).

In order to develop methodology for seismic vulnerability and seismic risk estimation of the buildings at the test site, detail investigation of the literature and approaches suitable for large scale assessment have been analysed and presented.

The methodology for seismic vulnerability and risk assessment on large scales, based on the calculation of vulnerability indexes, proposed in this project, demands information about the geometry and material characteristics of the buildings. Therefore, the study presents main characteristics of the buildings in the test site Kaštela City, following with an analysis of the building codes in different construction periods and their influence on the building characteristics regarding the used construction materials, level of the details that influences the seismic vulnerability and mechanical characteristics of the materials. The classification of the buildings according to their construction period, considering the fact that application of different building codes in the past reflected on the structural earthquake resistance and vulnerability, has great importance and may give answers, both when using the vulnerability index method, as well as in a detailed analysis of earthquake resistance of the buildings.

After that, the vulnerability index method, which is used for calculation of seismic vulnerability indexes of the buildings and development of seismic vulnerability maps, is presented. The procedure for calculation of vulnerability parameters which influence to final value of seismic vulnerability index are also given, as well as form for calculation of vulnerability index.

The vulnerability index is not a relevant indicator of seismic risk because it does not give information about the behaviour of the building under a specific seismic action. Therefore, the last part of this deliverable represents a methodology for development of new damage-vulnerability-peak ground acceleration relationships (vulnerability curves), based on the vulnerability indexes and the results of a static non-linear pushover analysis for the typical buildings at the test site. The developed procedure is used to calculate seismic risk of the buildings in terms of critical peak ground accelerations. The derived vulnerability curves allow us to estimate the damage to buildings in the event of a specific seismic action.

1.2 Overview of the methods for seismic vulnerability assessment of historical cities

Evaluating the seismic vulnerability and capacity, as well as the damage state, is a demanding task even for a single building. It requires complex non-linear methods such as nonlinear static (pushover) analyses [1,2], in which the structure is gradually loaded according to a uniform or a modal pattern up to the point of collapse, or incremental dynamic analyses [3], in which ground acceleration is increased up to the point of structural collapse. Both types of analysis allow the determination of the collapse load as well as the monitoring of the damage level, which is continuously increasing because of the nonlinear dissipative processes, including the fracturing and plasticity of the structural components. Due to the restrictions of the non-linear static analysis for structures that oscillate predominantly in the first mode, multi-modal nonlinear static analysis [4] can be used in the cases of horizontal and vertical irregularities.

An estimation of the seismic vulnerability of a large number of buildings at an urban scale is much more demanding because it is not possible to carry out a nonlinear analysis for all of these buildings.

One of the possible ways to solve this problem is the definition of fragility curves by means of a numerical analysis, coupled with statistical processing of the results. The fragility curves relate the probabilities of exceeding a specific damage measure for a certain intensity [5–7]. Several studies using fragility curves were recently performed to evaluate the seismic vulnerabilities and damage scenarios of urban centres to identify the main criticalities [8–16].

Empirical methods like the damage probability index method [17] and the vulnerability index method [18,19] have also been widely employed to define the vulnerability of urban areas.

Studies based on the damage probability index method [20,21] use probabilistic matrices of damage for the prediction of the damage patterns corresponding to different seismic events. The main finding of the damage probability index method is that certain structural typologies share the same probability of damage for a given intensity of earthquake. Many studies have been developed using different macro seismic scales, such as the Medvedev–Sponheuer–Karnik (MSK) scale, the Mercalli–Cancani–Sieberg (MCS) scale, and the European macroseismic scale (EMS-98) [22]. Among these, the EMS-98 scale is the most commonly used [22], which identifies five categories of damage. These damage categories very roughly estimate damage to both structural and non-structural elements.

Vulnerability assessments based on the vulnerability index method mostly use an improved version of the original Italian GNDT approach, developed by the Italian National Group for the Defense against Earthquakes [19]. It calibrates the weights of vulnerability parameters based on a large database of buildings damaged by past earthquakes [23–25]. To calculate the vulnerability of each building, this approach uses information about geometrical, structural, and material parameters and other relevant characteristics, such as structural typologies, horizontal and vertical irregularities, age, and conservation state.

Another established approach to the estimation of earthquake-induced losses is HAZUS [26,27], originally developed for the building typologies typical of regions in the USA.

A significant contribution to the simulation of earthquake risk scenarios in Europe was achieved by the RISK-UE project [21,28]. This project has developed a methodology for the evaluation of direct and indirect damages caused by different earthquake scenarios and the consequences of the damage. The method has been applied to seven European cities. Moreover, a building classification for typical European buildings has been proposed. According to this method, vulnerability and fragility curves are represented in two ways. The first way is through damage–seismic intensity curves, where the damage is defined in the interval 0–5, whereas the seismic intensity is determined using the EMS-98 macroseismic scale. The second way uses capacity curves, obtained through non-linear analyses of buildings typical of a certain class, for the purpose of deriving the fragility curves. The main difficulty in the application of this approach is how to assign an appropriate class to a particular building.

Most of the aforementioned approaches require the calibration of vulnerability and fragility curves via post-earthquake damage observations. Some other studies have analytically evaluated these vulnerability functions [29].

If the methodology of the vulnerability index is used for the seismic vulnerability assessment, a relation between the vulnerability index, the earthquake intensity, and the damage of the building can be established, using the observation data regarding the damage to buildings induced by past earthquakes [18,30].

Several contributions in the literature discuss various vulnerability assessment methodologies and case studies, in which the level of vulnerability and damage depend on the intensity of the seismic event [31,32]. In particular, we refer to [33] for a comprehensive review of the most relevant vulnerability assessment methods applicable at different scales. The applied approach depends on the study area (settlement, city, region, country) and the available data about the building stock, the purpose of the study, and the seismic hazard and damage levels induced by past earthquakes.

Considering the scarcity of post-earthquake damage observation data, especially in places where there have been no significant recent earthquakes, modern analytical approaches based on non-linear pushover analysis or incremental dynamic analysis offer the possibility to define vulnerability functions and maps independently of the available information about the damage level.

In general, methodologies for assessing seismic vulnerability and earthquake risk are being devised and applied in developed countries with high seismic risk that are able to allocate significant financial resources for seismic risk management. Croatia is a country with relatively limited resources dedicated to seismic risk prevention and management. To date, there is no technical regulation that prescribes the rules for assessing seismic vulnerability or seismic risk. The main contributions to the investigation of the seismic vulnerability mainly are drawn from individual research work, mainly focusing on the classification of the building stock and the development of vulnerability curves in the cities of Zagreb and Osijek. The vulnerability assessment in Zagreb concerns building classification and the development of fragility curves, whereas damage probability matrices related to earthquake intensity and structural systems have been used in assessments of damage according to the EMS-98 scale [34]. The vulnerability assessment of old confined masonry buildings in Osijek was recently performed using vulnerability indexes with behaviour modifiers and damage states assessed according to EMS-98 [35]. An approach based on damage index coefficients and single-degree-of-freedom systems was applied to a historical building located in Tvrdja, Osijek [36]. Mean damage levels, μD, according to the macro-seismic method [35] and the analytical capacity spectrum method, have been used to estimate the vulnerability of a few blocks of buildings, representative of the Osijek city, and the relevant fragility curves have been calculated [37]. All these investigations consider masonry buildings that are typical of continental Croatia. In addition, they have mostly estimated the damage for some building typologies merely based on the structural system and period of construction, while neglecting other relevant peculiarities.

Cities and settlements on the Adriatic coast have been gradually built and expanded over the centuries, and some of them, such as Split, are almost two millennia old. Some of these settlements are old towns with massive stone masonry buildings, which are often protected and declared architectural heritage sites. Around the cores of these sites, settlements have spread, especially since the second half of the 20-th century. Buildings in old city cores are very sensitive to earthquakes for several reasons. Firstly, they were constructed with unconfined stone walls and wooden ceilings, which cannot effectively transmit earthquake force. Secondly, their state of maintenance is often critical, considering their age. On the other hand, these city centres are densely populated, and the large number of tourists during the summer conspicuously increases the number of inhabitants. The seismic vulnerability assessment of the buildings of such towns and cities is crucial for the safety of permanent inhabitants and tourists.

In the present study, a seismic vulnerability assessment approach based on the calculation of seismic vulnerability indexes was applied to Kaštel Kambelovac, a Croatian settlement located in the coastal Dalmatian area. The town consists of a historical core with stone masonry buildings built between the 15th and 19th centuries and more peripherical buildings built from the beginning of the 20th century up to the present. These buildings were erected according to different technical regulations as they date from different periods, namely, before 1948, from 1949 to 1964, from 1964 to 1982, from 1982 to 2005, and, finally, from 2005 onwards.

The seismic vulnerability method applied in this study was derived from the Italian GNDT approach, with some modifications resulting from specific aspects of the buildings and the construction materials typical of the investigated area. Instead of the field observation of the damage state caused by past earthquakes, a static non-linear pushover analysis of the typical buildings at the test site was used to determine the yield and collapse accelerations and, subsequently, new relations between damage, vulnerability, and peak ground acceleration. The damage index was expressed in the 0–1 space by means of a tri-linear law defined by the yield acceleration, PGA_y, which represents the beginning of the damage, and the acceleration of the collapse of the building, PGA_c.

Seismic vulnerability indexes for the buildings were used to develop a new damage curves for the estimation of the damage to the building under specific seismic conditions.

The proposed approach has some similarities with a hybrid procedure for the definition of seismic vulnerability in Mediterranean cross-border urban areas [38]. However, in that paper [38], the vulnerability curve proposed by Guagenti and Petrini [23] was calibrated using the numerical results for prototype buildings that were representative of the most widespread building typologies [38]. On the contrary, the investigation presented in this study was based on the estimation of the damage state through non-linear pushover analysis of a large number of buildings, more specifically, a total of 11 buildings in historical core and 8 buildings outside of the historical core. Furthermore, a new damage-vulnerability-peak ground acceleration law has been derived.

2 Description of the test site

2.1 City of Kaštela

The selected test site for the application of the methodology developed in the project is the City of Kaštela (Figure 1) consisting of seven settlements developed from the 15th century until today. The structure of each settlement from the aspect of the architectural, urban and constructive feature is similar. Each settlement was formed around an old historical centre built in the 15th century. The settlements gradually spread over the years in the surrounding area. While developing, the settlements merged and the whole area is forming today's agglomeration the City of Kaštela. Nowadays, the city has seven separated historical centres, each composed of stone masonry buildings regarding the combination of smaller family houses, old mansions and public objects shown in Figure 1 and Figure 2. Since the area of old centres was bounded with walls, forts and the sea, objects inside the historical core are built on small courts. The lack of free space inside the enclosed area resulted in a high density of buildings. Buildings are leaned one against the other and merged into blocks with small streets between them. Each historical centre contains a church with a bell tower, also built in stone. Within the wider area around the old core, there are newer objects for public and residential purpose shown in Figure 2. In the entire area, there are no tall buildings typical for the urban vistas. Most buildings are limited on 4-5 floors, which is a consequence of developing small settlements toward the City of Kaštela. Historical development through the years, uniformity of the architecture and urbanism, significantly affect the selection of the test site. While observing a potential test site it can be noticed that with selecting the right area inside the City of Kaštela it is possible to cover all types of object from the aspect of characteristics important for this project. Selecting a single settlement with the corresponding wider area, it can cover characteristics of building history of whole Kaštela area from the 15th century onwards. After considering all the arguments for choosing the area to be observed and considering possibilities of extrapolation of results, Kaštel Kambelovac has been chosen as a narrower part of the test site.


a) Kaštel Sućurac


b) Kaštel Štafilić

c) Kaštel Gomilica

d) Kaštel Lukšić

e) Kaštel Stari

f) Kaštel Novi

e) Kaštel Kambelovac

Figure 1. Photos of forts and houses within the historical centre of City of Kaštela

Figure 2. Photos of typical residential houses on the wider area the City of Kaštela

2.2 Test site Kaštel Kambelovac

The relevant test area of Kaštel Kambelovac covers around 45,000 square meters and includes more than 400 buildings. Observed coastal part of Kaštel Kambelovac extends from the western border with Kaštel Lukšić to the eastern border with Kaštel Gomilica. Northern boundary of the area is the "Old road of Kaštela", ie. the Road of Dr. Franjo Tuđman.

The benefit of the chosen area is diversity of objects considering construction, architecture and material, built from the 15th century until today. According to [39] the oldest objects in the area date back to 1467. Those buildings were made of stone with a wooden floor construction. Undergoing minor modifications over the years they remained preserved until today. A historical centre founded in the 16th century around the tower of Cambi and the church of St. Mihovil with a bell tower from 19th century represent important arhitectural and historical heritage within the observed area. Nowdays, the settlement consists of a historical centre with stone masonry buildings built between the 15th and 19th centuries (Figures 3, 4) and of the parts outside the historical core dating from the beginning of the 20th century to the present (the north, east and, west parts are shown in Figure 5). These buildings were constructed in different periods according to different technical regulations. The oldest buildings were constructed before 1948; then, some blocks were erected from 1949 to 1964, from 1964 to 1982, and from 1982 to 2005. The most modern buildings have been built from 2005 onwards. Area is the mixture of private and public facilities.

Figure 3. The view on a historical centre, Kaštel Kambelovac

Figure 4. Historic centre with typical buildings, Kaštel Kambelovac

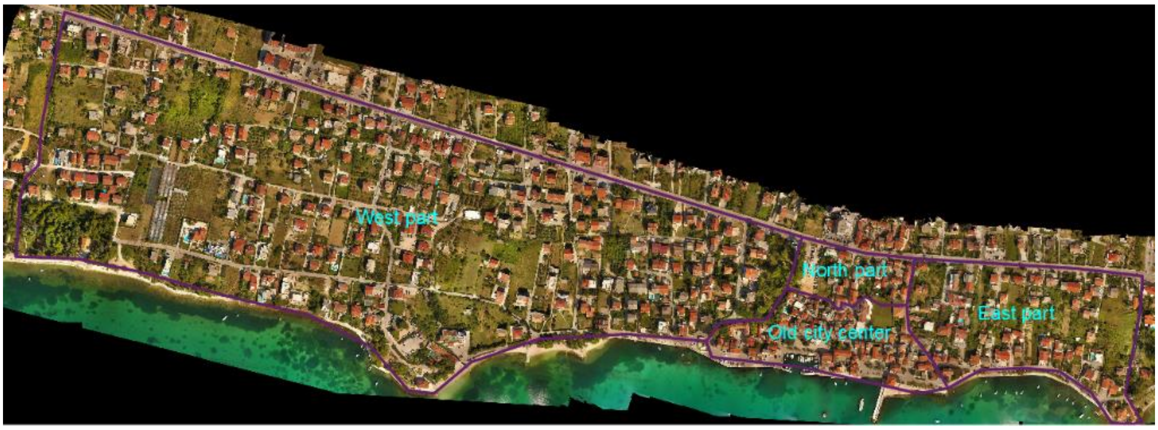


Figure 5. Kaštel Kambelovac test site with four characteristic parts

Plan view of the selected area is shown in Figure 6, where the green line defines the border of the chosen test site, purple line defines the border of the historical centre, while the red line shows position of the natural coastline. Within the historical centre (Figure 6 - purple line) the buildings are densely arranged and leaned against each other, opposite to the outer observed area where the buildings are spaced apart.

According to [39] the line of the existing coast was changing throughout the history. Original coastline (Figure 6. – red line) was formed of reefs that served as foundations for the historical fortresses. Through the time, the inhabitants of Kaštel Kambelovac were gradually expanding the coast which resulted in the present state where the former fortresses are integrated within the urban tissue (Figure 3).

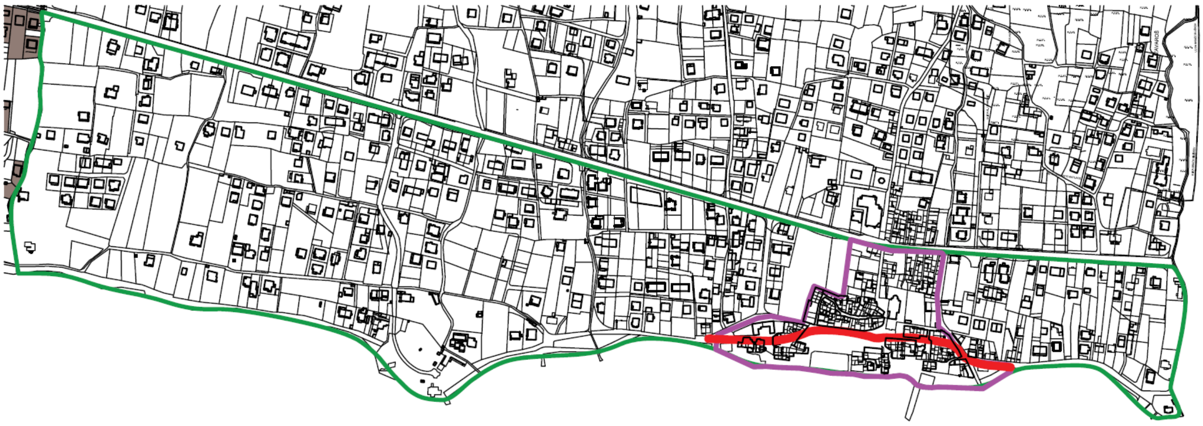


Figure 6. Plan view of the selected area (green line) with the mark of the natural coastline (red line) and the historical centre (purple line)

3 Investigation of the building codes in different construction periods and their influence to the building vulnerability

3.1 Review of regulations throughout the years

Over the years, under the impact of different political authorities, the advancement of the industry and the development of urban entities, the building sector is constantly exposed to changes. Consequently, during those periods, there are various types of buildings with different materials and different approaches to the construction design. First regulations defining the building design in the territory of Croatia were introduced during the Austro-Hungarian regency. By further advancement of the industry, the regulations are developing and adapting. At the beginning of the 20th century, the use of reinforced concrete began, that was significantly reflected in the buildings constructive systems.

The first regulations for the design of buildings to specific categorized loads were published in 1948. Further, the next generation came after the catastrophic earthquake in 1963, magnitude 6.1, which struck the City of Skopje. This seismic event brought significant changes in the design of buildings and it is a turning point for the formation of more detailed regulation for seismic loads that were published in 1964. After 1964, the upgraded regulations were coming from 1980 to 1990. Those regulations are valid until 2005 when the common European regulations are introduced in these areas. The first common regulations in the form of Croatian pre-standards are introduced in 2005 and 2007, while European standards Eurocode 8 with Croatian National Annexes are introduced in 2011.

If we observe the development of regulations through the years and compare it with today's regulations, the conclusion about structural and material characteristics of the buildings could be made. Namely, according to the year of construction and design, all buildings belong to certain regulations/rules that were valid at the time, and this allows the identification of some important characteristics about the observed buildings. In the Kaštela area there are mostly family houses built as masonry buildings or buildings made of unreinforced concrete. Due to the lack of information and detailed documentation for a particular building, it is possible to estimate way of building, characteristics of structural elements important for the earthquake resistance of the building, as well as material characteristics, using the construction period of the building. Following this conclusion, the buildings of Kaštela could be divided into 5 groups that are presented in the next chapters. Considering the selected area, the emphasis of division is on the parts of regulations concerning masonry buildings while unreinforced concrete as material is not covered in detail by regulations.

3.2 Building classification according to the construction period

3.2.1 Buildings built before 1948

For this group of buildings, the use of certain building materials, their quality and durability is prescribed by the Construction Regulation from 1893 (Part III relating to building regulations) [40]. The regulation suggests that public buildings must be made of brick or stone. Standard brick dimensions

29x14x6.5 cm were defined in 1894 [40]. Thickness of the load-bearing walls is specified, so that the thickness of the last-floor-walls should be minimally 45 cm. Lower floors have an increase of thickness in amount of 15 cm/floor starting from the last floor towards the ground floor. According the rule 3-storey building would have walls with thickness of 75cm, 60cm and 45cm counting from the first floor (ground floor) to the third floor.

According to [41] buildings built at the end of 19th and beginning of the 20th century are mostly made from industrial materials, resulting a uniform quality of construction products produced with constant professional supervision and accompanying attestations for each of the products. In this period most of the buildings are built as masonry structures of stone or brick-mortar walls. Buildings built before 1925 usually had floor construction made as wooden structure (beams + floor covering) that can be characterised as reduced in-plane stiffness construction considering seismic load transfer. In a later period with developing reinforced concrete (RC), floor constructions become stiffer and more resistant on horizontal loads. Since 1925, the use of reinforced concrete has begun gradually, primarily for the construction of multi-story buildings and public buildings, and then in the construction of smaller residential houses. Reinforced concrete (RC) in smaller residential buildings was used to build ceilings (floor structures). The characteristics of the first RC ceilings are very low thickness, minimal protective layers and segregation problems (low concrete quality). With the development of reinforced concrete, the floor structure is no longer performed only monolithic, it is increasingly used semi prefabricated ceilings, mostly made of concrete and in later periods with a brick elements. Semi prefabricated systems such as "Isteg", "Herbst", "Ferjan", "Ferenčak-Steinman" and "Avramenko" are used. System „Isteg“ is shown in Figure 7. Those systems are performed as semi prefabricated systems; prefabricated beams are brought on the construction site and placed on the supports, after, a steel sheet is attached to the construction nailed to the RC beams with the burned wire. After fixing the elements and the sheeting, a concrete (pressure) plate is poured, often unreinforced with a thickness of 5 cm. Ceilings are closed with light cover and it is sometimes difficult to recognize at first this type of floor. Such type of ceilings has different stiffness in two perpendicular directions (planar plate). The buildings were constructed without concrete boundary elements (beam and columns). Roof structures were mostly made of wood and covered with the old type of roof tiles (terracotta or concrete). The buildings were not designed considering earthquake load, the only horizontal load in the calculation is the wind load.

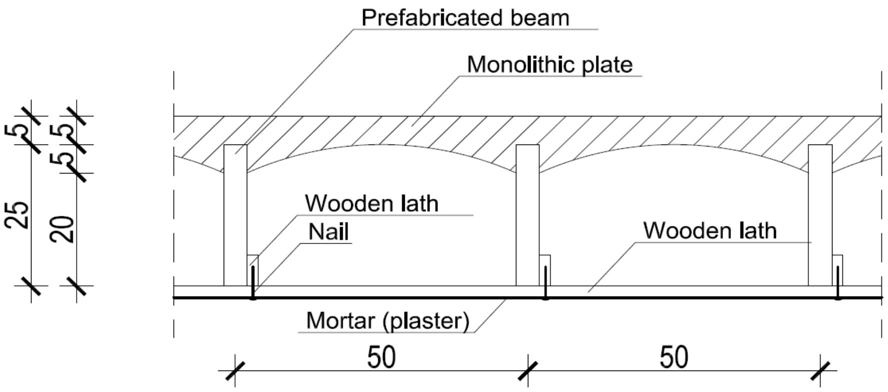


Figure 7. Semi prefabricated system „Isteg“

3.2.2 Buildings built in the period from 1949 to 1964

Regulations and design

The Temporary Technical Regulations for Building Loads (Off. Gazette FNRJ 61/48) [42] are the first regulations introducing the basis for design of buildings on a different type of loads. Loads are divided in different types defined as basic, additional and special loads. The static calculation is not required for structures with dimensions "according to the specified defined types" [43]. Vertical loads are divided into constant, moving and snow loads while horizontal loads are divided in loads caused by wind and earthquake. The regulation includes a map of wind speed zones and a map of seismological zones shown in Figure 8. Larger buildings are designed considering the horizontal loads, while "ordinary smaller brick, stone or concrete buildings, which are conventionally constrained by transverse walls and floor structure" are not calculated considering horizontal loads [42]. The horizontal force H used to control building stability is defined as the percentage of the sum of the permanent load and a half of the moving vertical load. Those forces are applied to the floor constructions. Buildings according to the regulations are divided into:

- buildings with massive walls and massive storey constructions $H_{min} = 1.0\%$
- buildings with massive walls and light storey structures $H_{min} = 1.2\%$
- lightweight building structures $H_{min} = 1.5\%$

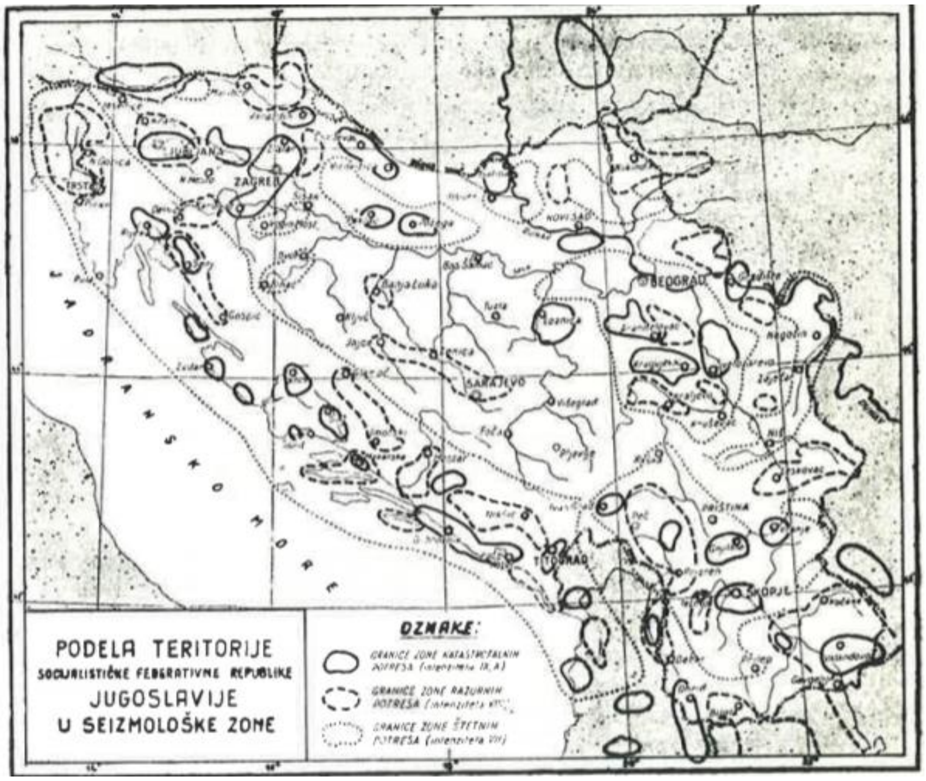


Figure 8. Seismic hazard map SFRJ [43]

The earthquake, as an extraordinary load, was taken into account with simplified methods, but only for larger objects. For the buildings calculated on the seismic load, the minimum horizontal force used in calculation would increase. Seismological zones are divided into three parts: the area of minor damage (intensity VII), the area of major damage (intensity VIII) and catastrophic destruction zone (intensity IX, X). Depending on the zone, H_{min} would increase by 0%, 50% and 100% respectively. The City of Kaštela, according to this distribution, belongs into the area of minor damage. Considering most buildings are family houses that are classified as smaller facilities, they are not design to horizontal forces.

Materials and structures

The Temporary Technical Regulations on Brick Walls (Off. Gazette SFRY 10/49) [43] defines in detail the necessary mechanical properties of the elements used for building masonry structures and their resistance to various impacts. The brick elements are divided into normal shape bricks, hollow masonry blocks and radial bricks. For those elements, the flexural strength and the compressive strength needed to be tested before embedding into construction. Flexural and compressive strength is given in Table 1 for the standard format brick. In the original table [43] stress is defined in kp/cm^2, in this text stress is expressed in MPa for easier comparison. For hollow masonry blocks, the standard mark is 150 and 110 with corresponding strength in compression of 12,0 and 8,5 MPa, while for facade blocks mark 200 is used with a minimum compressive strength of 16,0 MPa. The minimum compressive strength for mortar is 5,0 MPa. During this period, masonry buildings were built with vertical and horizontal reinforced concrete (RC) beams. Floor constructions are performed as monolithic or semi-prefabricated constructions.

Types of brick wall are: full brick in limestone, full brick in general purpose mortar, full brick in cement mortar, hollow brick in limestone, clinker in cement mortar, chamotte bricks and walls of hollow concrete blocks.

Table 1. Compressive and flexural strength values for different types of brick [43]

Brick type	Brick mark	Compressive strength MPa		Flexural strength MPa	
		Average	The individual minimum	Average	The individual minimum
Solid brick	200 150 110 70	20,0 15,0 11,0 7,0	16,0 12,0 8,5 5,5	4,5 3,6 3,0 2,6	3,6 2,9 2,4 2,0
Cellular brick	50	5,0	4,0	-	-
Facade (standard)brick	200 150	20,0 15,0	16,0 12,0	4,5 3,6	3,6 2,9
Radial brick	200 150	20,0 15,0	17,0 12,5	- -	- -

Compressive strength for brick walls expressed in MPa are shown in Table 2. Stress represent the maximum pressure on the edge and for the eccentric load. Allowed maximum for flexural stress is of one-tenth of the compressive strength from Table 2. The shear strength for the wall made of cement mortar is one-tenth of compressive strength and for the wall of general purpose mortar is equal to one-twelfth of compressive strength.

Table 2. Compressive strength values for brick masonry walls [43]

Brick mark (medium strength) (kg/cm^2)		Compressive stress for brick masonry wall (MPa)			
		In cement mortar	In general purpose mortar	In limestone	In clay mortar
	200	2,2	1,8	-	-
	150	1,6	1,4	1,0	-
	110	-	1,1	0,8	-
	70	-	0,8	0,6	0,5
(Exceptional Approval)	40	-	-	0,4	0,4
(porous)	30	-	-	0,3	0,3

3.2.3 Buildings built in the period from 1964 to 1982

Regulations and design

Temporary technical regulations for construction in seismic areas (Off. Gazette SFRY 39/64) [44] from 1964. defined more detailed calculation for structures placed in seismic zones. As an addition to this regulation, a more detailed seismic map is defined (Figure 9.) The objects are divided into four groups, depending on their category, seismicity of the area is increased starting from seismicity of the area in which they are located. The first category of buildings is those predicted for the gathering a large number of people (hospitals, schools, ...), the second category is the buildings with a large number of people which are not in the first category (hotels, residential buildings, restaurants, ...). The third category is buildings with minor importance but they are also design to the earthquake forces (industrial buildings, stalls, ...), while the fourth category buildings are those that do not pose a threat to human lives and those are not calculated for the earthquake. In calculation, seismic forces are defined as the sum of building weight, all additional load (fixed equipment in the building), snow load and half of the live load used for building design (with the exception that it can be counted all if it is determined). The calculation is made for two orthogonal directions; the forces are applied to the height of the storey structures.

Seismic forces are calculated according to the formula $S_{ik}=K_c \cdot \beta_1 \cdot \eta_{ik} \cdot Q_k$, where S_{ik} is design seismic force acting in the point "k" caused by "i" oscillating form. K_c is a coefficient of seismicity which depends on the area, soil type and quality and the type and purpose of the building, and it is given in Table 3. β_1 represents a dynamics coefficient for the "i" oscillation form of the building, while η_{ik} is a coefficient regarding on the oscillation of the structure and structural height. Finally, Q_k represents the weight of the building concentrated in point k.

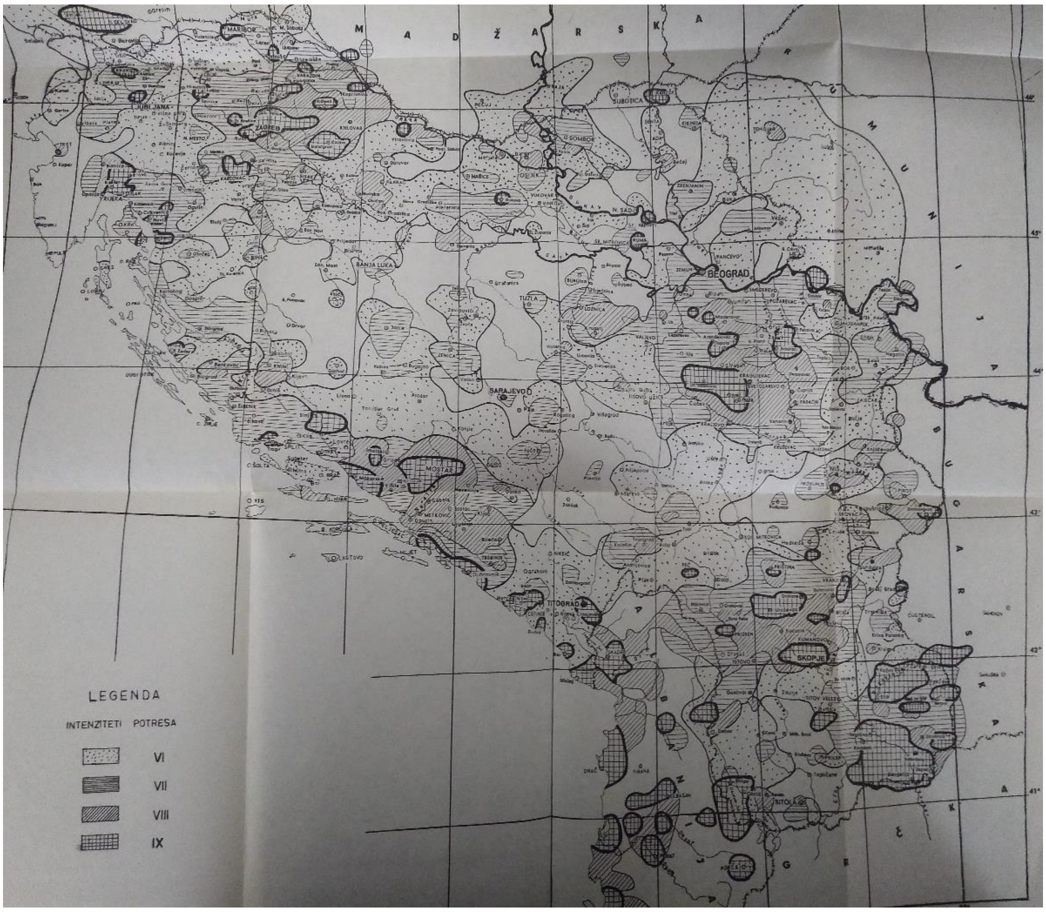


Figure 9. Seismic hazard map SFRJ [44]

Table 3. The coefficient of seismicity K_c [44]

Seismicity	VII	VIII	IX
Weak soil	0,03	0,06	0,12
Medium soil	0,025	0,05	0,10
Solid soil	0,02	0,04	0,08

Coefficient β_1 is determined by the expression $\beta_1=0{,}75/T_i$ where T_i is a period of observed oscillation form of the building. The value of the coefficient β_1 is limited with $1{,}5 \geq \beta_1 \geq 0{,}5$.

Design seismic force for the Kaštela according to equivalent static load method is given as $S_{ik}=0{,}02 \cdot 1{,}5 \cdot \eta_{ik} \cdot Q_k = 0{,}03 \cdot \eta_{ik} \cdot Q_k$.

In masonry buildings there are restrictions in the number of floors, the selection of material and floor plan shape. Brick buildings are divided into two groups: buildings with RC beams and rigid horizontal diaphragms, and buildings with RC beams, RC columns and rigid horizontal diaphragms. Depending on the type of building and design seismic acceleration (where the building is located), the number of floors is limited. For the two-floor buildings in the seismic area up to VIII degree, RC vertices (columns) are not required, only the prescribed dimensions of the brick columns (short brick walls) between the openings. As the area of Kaštela belongs to VII zone the minimum length of wall between the openings is 64 cm. Stone and concrete block masonry are limited on two floors. For buildings constructed of irregular stone, it is necessary to perform horizontal reinforcement in height of 4m, 2m or 1 m depending on the degree of seismicity (VII, VIII, and IX respectively).

Materials and structures

The use of semi-prefabricated ceilings "Avramenko", "Herbst", etc., is permitted for buildings bounded with RC columns. The most used building material in this period is a brick. The regulation provides some guidelines for brick buildings, referring primarily to the earthquake resistance of masonry elements. The influence of the position of the joints on the rows above and below, the influence of the compressive strength of the element on the bearing capacity, the influence of the mortar and elements affecting the increase of the mortar and brick adhesion are investigated. Results are recommendation to use walls strengthen with vertical and horizontal RC elements and not to use mortar made only of lime and of cement. The lime mortar wall is exposed to a large subsidence, different from the behaviour of reinforced concrete elements. That difference is leading to cracks between elements and resulting adverse effects from the aspect of bearing capacity. Cement mortar is forbidden due to brittle behaviour that leads to stress concentration and fracture. It is imperative to use a general purpose mortar. The Regulation on Technical Measures and Requirements for Building Walls (Off. Gazette SFRY 17/70) [45] provide rules for the construction of walls. According to regulation minimum class for clay elements is M 100 and for concrete elements M 75. Table 4 shows compressive strength of load applied in the centre of a specimen in dependence on the mark of elements, mortar class and the slenderness of the wall.

Table 4. Values of compressive strength (MPa) for walls according to elements and quality of mortar [45]

Mark of wall elements	Mark of mortar	Wall slenderness h/d					
		10	12	14	16	18	20
75	5	0,40	0,35	-	-	-	-
	25	0,50	0,45	0,35	-	-	-
100	5	0,50	0,45	0,35	-	-	-
	25	0,60	0,50	0,45	0,35	-	-
	50	0,70	0,60	0,50	0,40	-	-
150	25	0,90	0,80	0,65	0,55	0,40	-
	50	1,10	0,95	0,80	0,65	0,50	-
	100	1,30	1,15	0,95	0,80	0,60	-
200	25	1,00	0,85	0,70	0,60	0,45	-
	50	1,30	1,15	0,95	0,80	0,60	-
	100	1,60	1,40	1,20	0,95	0,75	0,50

The average shear stress must not exceed 8% of the actual pressure stresses in the wall. Only the walls in load direction (horizontal) are taken for the calculation of the shear stress. Elasticity modulus for brick walls is obtained according to expression E = 6000· $\sigma_{0,dop}$ (in Mpa), where $\sigma_{0,dop}$ is compressive strength for h/d = 10 (slenderness).

3.2.4 Buildings built in the period from 1982 to 2005

Regulations and design

The Regulation on Technical Standards for Constructions in Seismic Areas (Off. Gazette SFRJ 31/81) [46] published and valid in 1981 with new seismic hazard map. The new regulation, with minor modifications, has been used for more than 20 years for designing objects in seismic areas. According to the regulation, the objects are designed not to collapse for the earthquake of the highest intensity, predicted for certain area. Construction can suffer only limited damage, depending on type and purpose of object. The regulations divide objects into five groups, of which four categories are typical buildings with different purpose. All other economic, social and security objects that are classified as "out of category" belong to the fifth group. Buildings that belong to category I are mainly of public purpose, described as buildings predicted for gathering large numbers of people, for example hospitals and schools. The second category buildings are residential and public buildings that are not classified in group I. Third category buildings are described as ancillary and production buildings, while the fourth category buildings are those whose collapse do not endanger human lives. Buildings that are classified in the fourth category are not design to the earthquake. The soil is divided into three categories of which the first is rocky ground, the second group is compacted and semi-solid soil and the third is soft soil. The maximum horizontal displacement of the object is limited to f_{max} = H/600, where H is the total height of the object. The total weight of the object is calculated as a sum of weight, additional load, mobile load and snow. Seismic forces are calculated according to equivalent static load method or dynamic analysis method.

Seismic force S according to equivalent static load method is calculated as S=K·G, where K is seismic coefficient for horizontal direction, while G is the total weight of the object. The minimum value of seismic coefficient K is 0.02. The seismic coefficient K is calculated as a product of four coefficients in a form $K=K_0 \cdot K_s \cdot K_d \cdot K_p$, where K_d is a dynamic coefficient, K_p is a coefficient of ductility and damping, K_0 is a seismic intensity coefficient and K_d is a coefficient of object category shown in Figure 10.

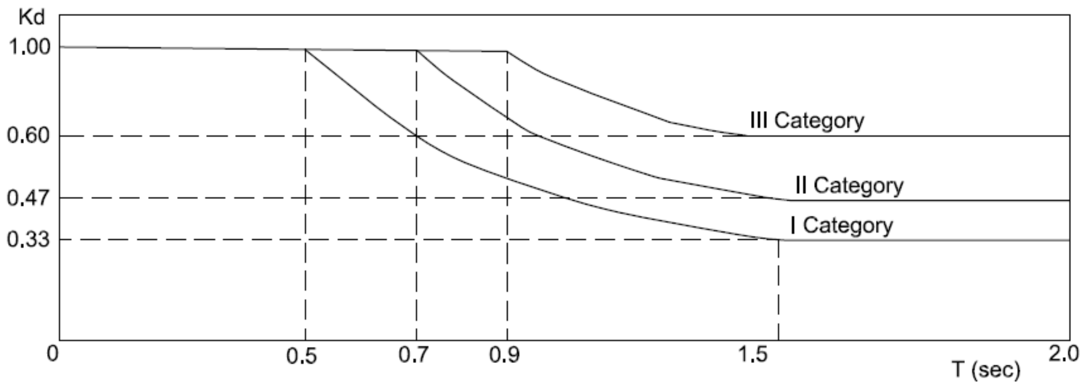


Figure 10. Graphical representation of coefficient K_d depending on T and soil category [46]

Seismic coefficient for vertical direction is calculated as $K_v=0{,}7\cdot K= 0{,}7\cdot K_0\cdot K_s\cdot K_d\cdot K_p$.

The seismic coefficient for horizontal direction for residential buildings in Kaštela area is equal to $K=K_0\cdot K_s\cdot K_d\cdot K_p=1{,}0\cdot 0{,}05\cdot 1{,}0\cdot 1{,}6=0{,}08$.

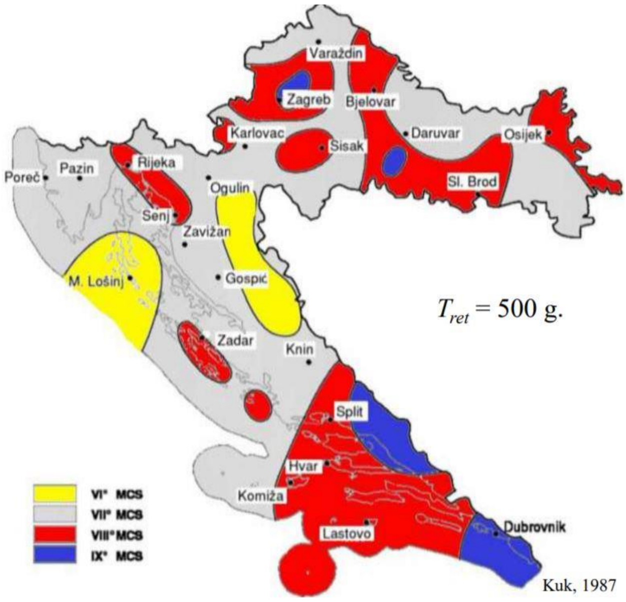


Figure 11. Croatian seismic hazard map [46]

The regulation provides guidelines for different type of structures, depending on the expected behaviour in the earthquake. For RC structures proposed types are frame system, a wall system or a combination of a frame with walls or core. Masonry structures are divided into unbounded masonry structures, masonry structures with vertical RC columns and a reinforced masonry structure (reinforcement in horizontal joints and in centre of the wall and on the outer part of the wall). Floor construction is a rigid diaphragm with a minimum thickness of 4 cm for the pressure plate for the semi-prefabricated system. The maximum wall distance depends on the thickness of wall (5m for the wall 19cm wide, 6m on the wall 24cm wide). For long walls the maximum distance between the vertical RC columns is 5m, vertical RC columns are placed on the corners of the building, in the place of vertical wall joints and at the free ends of the walls with thickness greater than 19 cm. Horizontal RC beams must be performed on all walls thicker than 19 cm. The use of unbounded masonry is allowed only in areas of very low seismicity <0.06g. For zone VIII it is allowed to build two-storey masonry construction without vertical RC columns, three-storey for structures with vertical RC columns, and P + 7 for structures with vertical and horizontal columns and beams, with the condition that these objects are calculated for the seismic load. If buildings are not calculated for the earthquake, the permissible level for all is ground floor + 1 for VIII degree of seismicity.

The regulation on technical standards for the design and construction of prefabricated structures of unreinforced and reinforced cellular concrete (Off. Gazette SFRJ 6/81) [47] defines strength for walls made of concrete (wall) blocks shown in Table 5.

The stresses apply only to a free height of up to 3m, for the walls of 4m the permissible stress is two-thirds of the value from table. It is compulsory to use the general purpose mortar minimum mark M25. The minimum length of the wall is 90cm, and if the wall is between 60cm and 90cm then the same rules apply as well as for a 4m wall.

Table 5. Values of compressive strength for concrete block walls [47]

Cellular concrete mark	Compressive strength in MPa (MPa) for different wall thickness			
	15 cm	20 cm	25 cm	30 cm
25	0,25	0,30	0,35	0,40
30	0,27	0,32	0,37	0,42
35	0,30	0,35	0,40	0,45
40	0,32	0,37	0,42	0,47
45	0,35	0,40	0,45	0,50
50	0,35	0,40	0,45	0,50
55	0,35	0,40	0,45	0,50
60	0,35	0,40	0,45	0,50

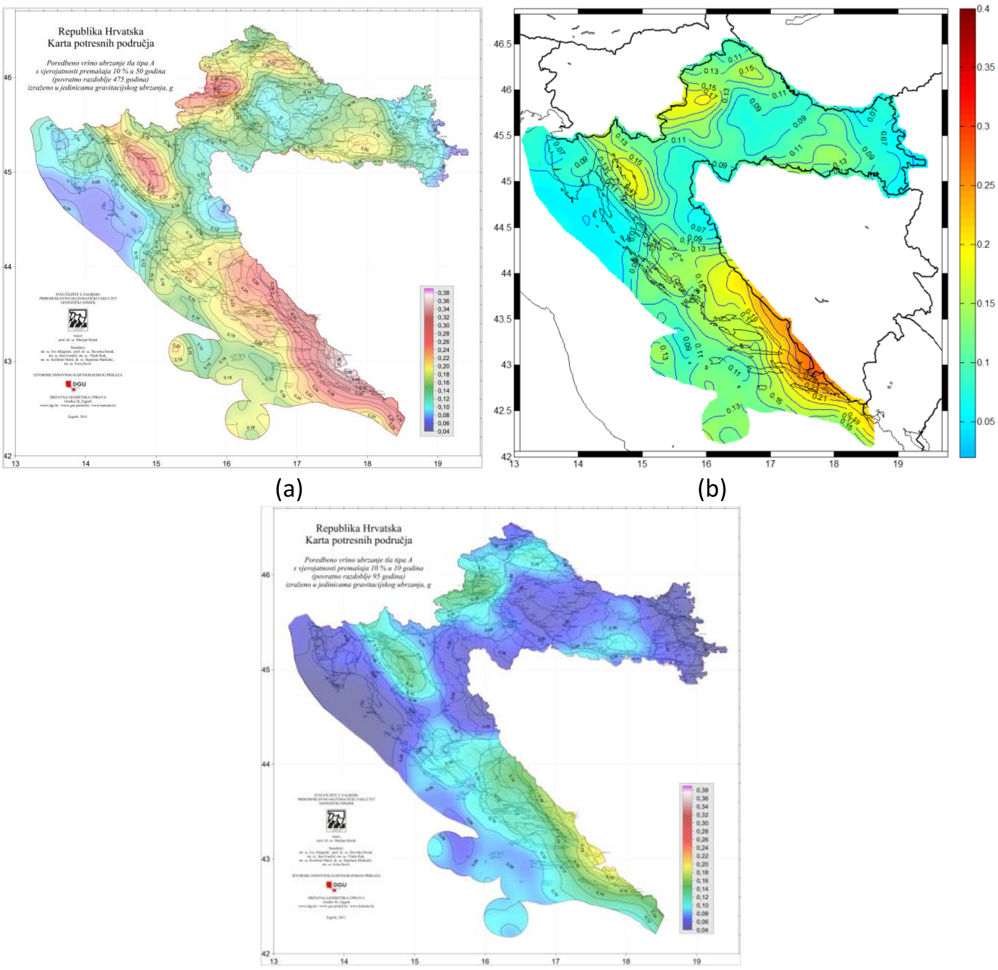
3.2.5 Modern buildings built from 2005 onwards

Regulations and design

Modern European norms for design of the earthquake resistant structures [48] have firstly introduced into Croatian technical regulations in 2005 [49], based on the European prestandard ENV 1998-1:1994 [48], which significantly increased the safety of the structures to the influence of the earthquakes.

The HRN EN 1998-1 norm [50] is the current regulation related to the design of earthquake resistance of structures based on European norm EN 1998-1:2004 [51]. It consists of multiple parts, the first four are general requirements, while the fifth to the ninth chapter is written as a set of special rules for different types of constructions made of different materials. Other parts of the standard 1998-2 to 1998-6 consist special provisions relating to bridges, reconstructions, silos, geotechnics and tall structures such as industrial chimney or masts. The new maps of seismic areas [52] shown in Figure 12 are defined and used in design of earthquake resistance of structures.

The basic requirements for the earthquake resistance are that the building shall be designed to withstand calculated loads without local or global collapse while retaining its structural integrity and a residual load bearing capacity after the seismic event. The structures are designed to meet no collapse and damage limitation requirements. In order to satisfy the fundamental requirements an ultimate limit state (ULS) and the damage limitation state (DLS) should be checked. Ultimate limit state defines structural system resistance and energy-dissipation capacity, assigning behaviour factor q to every structure. Beside ULS and DLS there are some specific measures relating on seismicity requirements of structures such as simple and regular floor plan and elevation, foundations capable of transmitting actions to the ground, mass equilibrium and proper dilatation on the same set. The tendency of these requirements is to avoid brittle structural failure and increase its ductility. Special attention to these regulations is directed to a detailed categorization of the soil into seven different groups depending on the average velocity of the shear wave $v_{s,30}$. Therefore, the soil categories are as follows: A, B, C, D, E, S1, and S2.



(a) (b)

(c)

Figure 12. Croatian seismic hazard maps: (a) T=475g; (b) T=225g; (c) T=95g [52]

The earthquake motion at a point on the surface is represented by an elastic ground acceleration response spectrum called an "elastic response spectrum" or by time-history representation (acceleration, velocity and displacement). For the elastic spectrum, the horizontal seismic activity is described by two orthogonal components assumed to be independent and represented by the same response spectrum. Elastic response spectrum is shown in Figure 13. The response spectrum depends on the soil type, which is expressed in the lower and upper boundaries of the period with the constant spectral acceleration (T_B and T_C), soil factor S, and the value that defines the beginning of a constant displacement response range of the spectrum T_D [50, 51].

Eurocode 8 also considers the ability of the structure to dissipate energy with ductile behaviour, and consequently, the calculation is carried out for a reduced elastic spectrum, so-called "design spectrum". The reduction is achieved by factor q. The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various parts of EN 1998. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

Time history representation of the earthquake motion may be made by using artificial or recorded accelerograms [50, 51].

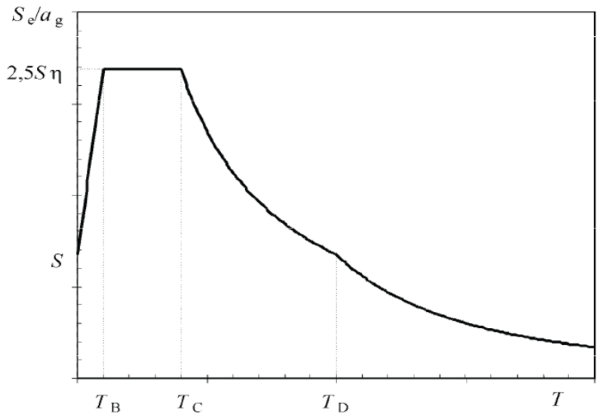


Figure 13. Elastic response spectrum [50, 51]

Materials and structures

For masonry buildings that are designed according to HRN EN 1998-1 [50] and HRN EN 1996 [53], the use of unreinforced masonry, bounded masonry and reinforced masonry is allowed. The normalized compressive strength of the wall elements is $f_{b,\,min} = 5$ N/mm^2 perpendicular on the horizontal joint and $f_{bh,\,min} = 2$ N/mm^2 parallel to the horizontal joint. In the Kaštela area where the design acceleration is equal to $a_g = 0.22g$, unbounded walls are not allowed.

Several types of masonry elements are used such as clay wall elements (bricks, clay blocks), lime-silicate blocks, concrete blocks or hollow concrete blocks.

Depending on the masonry type used for the seismic resistant elements, masonry buildings can be classified into the following types of constructions:

- Unreinforced masonry construction where the collapse of the walls occurs by shear. The walls are treated as fragile structural elements with limited energy absorption. This group includes walls made of older type of blocks and walls made of crushed or hewn stone.

- Confined masonry construction has walls bounded with RC vertical and horizontal confined elements (vertical columns and horizontal beams). Vertical and horizontal elements do not have the effect of frame construction, but together with the masonry contribute to the bearing capacity of the structure. The masonry carries vertical loads from the upper floors of the building, while the RC elements significantly contribute to the ductile behaviour of the masonry, taking over horizontal forces.

- Reinforced masonry construction where the horizontal and vertical reinforcement in the joints are used to improve seismic resistance of the masonry structures.

Typical values of flexural strength and shear strength for masonry walls according to [53] is shown in tables 6-9. For horizontal structures, semi prefabricated floors (Fert, Monta and similar systems) or reinforced concrete slab are mostly used. Fert system is shown in Figure 14. Semi prefabricated ceiling is compound of prefabricated beams made of reinforced concrete placed in a clay matrix. Beams are usually placed at a distance of about 50 cm. The gap between the beams is filled with brick blocks that facilitate the ceiling and serve as filling and sheet. The thin concrete slab is poured over the so-formed elements. Concrete fills the bar area and exceeds min 4 cm above the blocks to form a pressure plate. The pressure plate is reinforced, the stiffness of the ceiling in the other direction is ensured with the transverse beam.

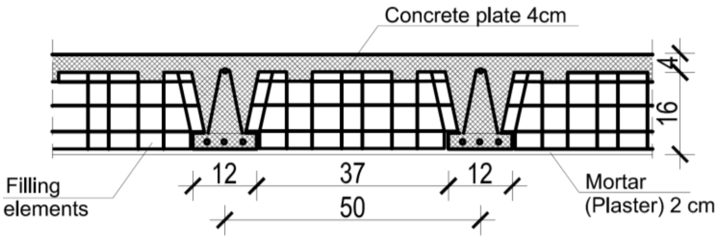


Figure 14. Section of „Fert“ structure

Table 6. Values of flexural strength for plane of failure parallel to bed joints [53]

Masonry Unit	f_{xk1} (MPa)			
	General purpose mortar		Thin layer mortar	Lightweight mortar
	$f_m<5$ (MPa)	$f_m>5$ (MPa)		
Clay	0,1	0,10	0,15	0,10
Calcium silicate	0,05	0,10	0,20	not used
Concrete	0,05	0,10	0,20	not used
Autoclaved Aerated Concrete	0,05	0,10	0,15	0,10
Manufactured stone	0,05	0,10	not used	not used
Dimensioned natural stone	0,05	0,10	0,15	not used

Table 7. Values of flexural strength for plane of failure perpendicular to bed joints [53]

Masonry Unit		f_{xk2} (MPa)			
		General purpose mortar		Thin layer mortar	Lightweight mortar
		$f_m<5$ (MPa)	$f_m>5$ (MPa)		
Clay		0,20	0,40	0,15	0,10
Calcium silicate		0,20	0,40	0,30	not used
Concrete		0,20	0,40	0,30	not used
Autoclaved aerated concrete	$\rho<400$ kg/m^3	0,20	0,40	0,20	0,15
	$\rho>400$ kg/m^3	0,20	0,40	0,30	0,15
Manufactured stone		0,20	0,40	not used	not used
Dimensioned natural stone		0,20	0,40	0,15	not used

Table 8. Values of the initial shear strength of masonry [53]

Masonry units	f_{vk0} (MPa)			
	General purpose mortar of the Strength Class given		Thin layer mortar (bed joint ≥0,5 mm and≤ 3 mm)	Lightweight mortar
Clay	M10-M20	0,30	0,30	0,15
	M2,5-M9	0,20		
	M1-M2	0,10		
Calcium silicate	M10-M20	0,20	0,40	0,15
	M2,5-M9	0,15		
	M1-M2	0,10		
Concrete	M10-M20	0,20	0,30	0,15
Autoclaved Aerated Concrete	M2,5 – M9	0,15		
Manufactured stone and Dimensioned natural stone	M1-M2	0,10		

Table 9. Mechanical properties of masonry made of different materials [53]

Properties	Material		
	Concrete	Clay	Autoclaved Aerated Concrete
Normalised mean compressive strength of a masonry unit	f_b=5,0·1,15=5,75 Mpa	f_b=10,0·1,15=11,5 Mpa	f_b=4,0·1,15=4,60 Mpa

Group of elements	2b (K=0,5)	2b (K=0,5)	2b (K=0,5)
Mortar	M10 (f_m=10,0Mpa)	M10 (f_m=10,0Mpa)	M10 (f_m=10,0Mpa)
Characteristic compressive strength of masonry	$f_k = K \cdot f_b^{0,65} \cdot f_m^{0,25} = 0,5 \cdot 5,75^{0,65} \cdot 5,0^{0,25} =$ 2,33Mpa	$f_k = K \cdot f_b^{0,65} \cdot f_m^{0,25} = 0,5 \cdot 11,5^{0,65} \cdot 10,0^{0,25} =$ 4,35Mpa	$f_k = K \cdot f_b^{0,65} \cdot f_m^{0,25} = 0,5 \cdot 4,6^{0,65} \cdot 4,0^{0,25} =$ 1,91 Mpa
Characteristic shear strength of masonry	$f_{vk} = 0,065 \cdot f_b = 0,65 \cdot 5,75 =$ 0,32MPa	$f_{vk} = 0,065 \cdot f_b = 0,65 \cdot 11,5 =$ 0,75MPa	$f_{vk} = 0,065 \cdot f_b = 0,65 \cdot 4,6 =$ 0,29 MPa

4 Calculation of the seismic vulnerability index according to vulnerability index method

4.1 Vulnerability index method

Activity 3.3. “Assessment of climate-unrelated hazards exposure in urban and coastal areas (seismic action)”, among other tasks, aims to develop seismic vulnerability maps for HR benchmarks test site and guidelines for estimation of seismic vulnerability of buildings based on vulnerability index. The methodology for large scale vulnerability assessment, based on calculation of vulnerability indexes, is developed and widely used in Italy but it hasn’t been regulated and applied in Croatia yet. In this project existing Italian methodology is applied on a Croatian test site Kaštela.

The first part of this study consists of the investigation of the procedure for calculation of the building vulnerability according to vulnerability index method. The study considers masonry buildings and can be applied both to historical buildings made of unregular or regular stone blocks, and more recent buildings made of concrete or clay blocks with mortar joints.

The adopted vulnerability index method was based on the original vulnerability index method for masonry structures developed by the Italian National Research Council and the Italian National Group for the Defense Against Earthquakes (GNDT) from 1984 onwards [18,19].

The method consists in filling in a survey with data relating to 11 geometrical and structural vulnerability parameters, the calculations of those parameters, and finally, the calculation of the vulnerability index for the building. The main parameters consider the type and organization of the resistant system, the quality of resistant system, the conventional resistance along two main horizontal directions of the building based on the estimation of the maximum resistant shear of the structure, the position of the building and foundations, the typology of floors, the planimetric and elevation configuration, the maximum distance among the walls, the typology and weight of the roof, the presence of non-structural elements, and the state of conservation. For each parameter, the surveyor must judge the condition among four possibilities, from “A”, corresponding to an optimal condition, to “D”, meaning a bad condition. For each judgment, the method provides a numerical score. Weight coefficients are then used, relating to each parameter to account for the relative importance of each parameter in the global definition of vulnerability. Finally, a vulnerability index I_v is calculated in the form:

$$I_V = \sum_i s_{vi} w_i \, , \tag{1}$$

where s_{vi} is the numerical score for each class and w_i is the weight of each parameter. This vulnerability index is then normalized in a 0–100% range. A low index means that the structure is not particularly vulnerable and has a high capacity under seismic action, whereas a high index shows that the structure is vulnerable and has low seismic capacity.

In this project the method was improved with the modifications proposed for the region of Tuscany in 2003 [54] to consider the possible substitution of the original light timber floors with heavier reinforced-concrete slabs. Field observations of the damage states of heritage buildings after earthquakes in the past 30 years has indeed shown that the replacement of timber floors with heavier concrete slabs, when performed on low quality masonry buildings, can substantially change the dynamic behaviour of the structures, because it adds a considerable mass on the top of the building, thus increasing the overall in-plane stiffness. These changes can cause the collapse of the structure under earthquake excitation.

Therefore, researchers on the Tuscan region corrected the vulnerability index method and updated the weights of parameters 1, 5, and 9. In particular:

- in parameter 1, the weight changes from 1 to 1.5;
- in parameter 5, the weight changes from a range 0.5–1 to the range 0.5–1.25, in particular, if heavier floors, such as concrete slabs, are supported by not-so-resistant masonry walls, the weight coefficient is 1.25;
- in parameter 9, the weight changes from the range 0.5–1 to 0.5–1.5, and the calculation of the weight is the same as the original interval (0.5–1.0), where the weight is 1.25 if a heavier roof, such as a concrete slab, is supported by weak masonry walls, and it is 1.5 if there is also a heavy floor just below the roof.

The present work uses the aforementioned modified weights proposed for the Tuscan region [54]. The weights of all the other parameters are assumed as in the original vulnerability index method.

The vulnerability parameters, their numerical score values and weight coefficients are shown in Table 1. The maximum value of the vulnerability index I_v is 438,75.

Table 10. Vulnerability parameters and their weights

Parameter	Score (s_{vi})				Weight (w_i)
	A	B	C	D	
Type and organization of the resistant system (P1)	0	5	20	45	1,50
Quality of the resistant system (P2)	0	5	25	45	0,25
Conventional resistance (P3)	0	5	25	45	1,50
Position of the building and foundation (P4)	0	5	25	45	0,75
Typology of floors (P5)	0	5	15	45	var.
Planimetric configuration (P6)	0	5	25	45	0,50
Elevation configuration (P7)	0	5	25	45	var.
Maximum distance among the walls (P8)	0	5	25	45	0,25
Roof (P9)	0	15	25	45	var.
Non-structural elements (P10)	0	0	25	45	0,25
State of conservation (P11)	0	5	25	45	1,00

Finally, in this document the current Croatian standards for the design of masonry structures HRN EN 1996-1 [53] and design of structures for earthquake resistance HRN EN 1998-1 [50], is adopted in definition of the classes for each parameter.

Based on the presented methodology, a vulnerability index form in Excel was created and used to compute the vulnerability indexes of masonry buildings in the test site.

4.2 General notes

The vulnerability index form is filled for one building only. The relevant floor which needs to be examined (Figure 1) is the one that is in the most unfavourable conditions from the standpoint of resistance to horizontal action, which is most commonly ground floor (Figure 15b). In cases as shown in Figure 15a, in which the basement can freely oscillate, this floor should also be considered as relevant. In the cases with strong discontinuities and significant variations in height, it is necessary to check that one of the floors is in the most unfavourable conditions (Figure 15c). The same applies in cases where there are variations in the type of construction (Figure 15d).

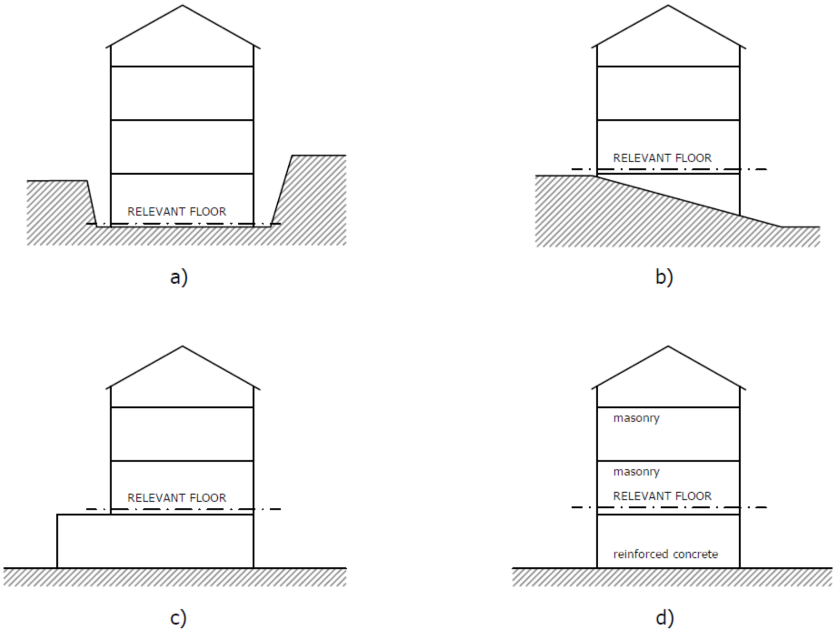


Figure 15. Examples of determining the relevant floor

For each data, the level of knowledge that has been provided before its assessment must be given. The following levels of knowledge are possible:

- **H - high level:** mostly direct information (measurements in situ, recorded details, reliable technical data, direct insight into information with a degree of reliability close to security);
- **M - medium level**: information obtained indirectly, from photographs, from non-performance drafts, reports of non-destructive testing, from experience of similar situations, from verbal information from persons with experience in tests, with medium confidence between high (H) and low (L);
- **L - low level**: predominantly presumed information (measures deducted from reasonable hypotheses such as those concerning the usual methods and the most frequent design choices,

verbal information different from the previous ones) with a degree of reliability of slightly more than a purely random choice of the class.

The vulnerability index form is divided into 5 parts provided for the following data:

- a list of 11 required parameters;
- assessment of the four possible classes (A, B, C, D) for each parameter;
- level of knowledge: H, M or L;
- elements of assessment: depending on whether they are requested or not;
- drawings and reminders of what is mentioned in the manual for some parameters.

4.3 Vulnerability parameters

4.3.1 Type and organization of the resistant system

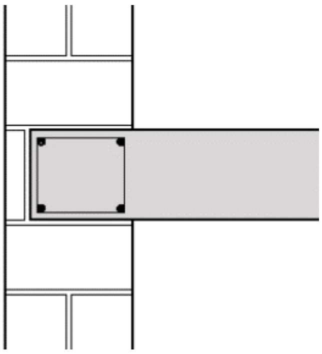
This parameter evaluates the degree of organization of vertical elements, not looking at the type of materials and characteristics of individual walls, but the presence and efficiency of the connection between the walls. Four classes are defined as follows:

Class A:

- Buildings built in accordance with the new seismic norms (HRN EN 1998-1) [50];
- Buildings strengthened or retrofitted in accordance with the norms for assessment and retrofitting of the buildings (HRN EN 1998-3) [55].

Class B:

- Buildings that present at all levels and on all free sides connectors in terms of confinement horizontal tie beams or chains and connected in a way they are able to bear seismic actions (Figure 16).


a)

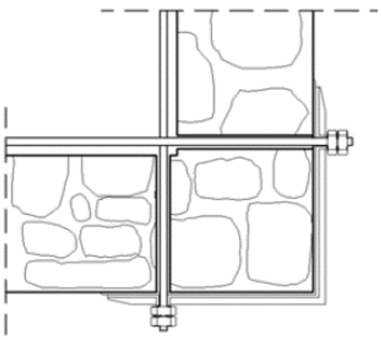

b)

Figure 16. Details of buildings with confinements

Class C:

- Buildings with well-connected walls but no confinements (Figure 17).

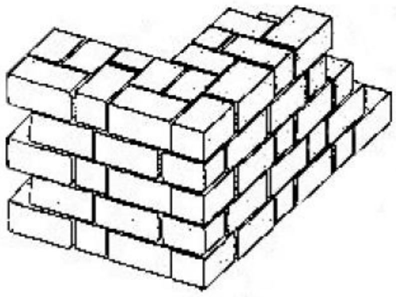


Figure 17. Well connected masonry walls without confinements

Class D:

- Buildings with poorly connected walls and no confinements (Figure 18).

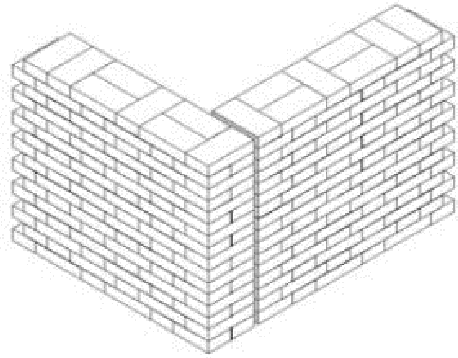


Figure 18. Poorly connected masonry walls

4.3.2 Quality of the resistant system

This parameter takes into account two main factors:

- quality and condition of preservation of blocks and mortar;
- texture and organization of the masonry.

The former refers to the quality of the materials of building blocks and mortar. If the mechanical properties of mortar are satisfying, even poorly organized and irregular masonry can have a sufficient strength. This does not apply in the case of hollow brick blocks with too many holes.

The second relates to the uniformity of the size and the regularity of the layout of the blocks. The horizontality of the rows and the brickwork rule must be checked, although this is not always a sure indicator of a well-made wall. It is possible that we are dealing with a hollow wall with a fill, so this possibility should be examined in more detail. For the hollow walls, the presence of transverse joint elements (connectors) between the two faces of the wall is essential. The importance of these elements can be seen by comparing the difference in the monolithicity of the cross-section of the wall that is correctly executed with connectors and the one without connectors. In the action of horizontal seismic force, the connectors prevent the slip between the two faces (Figure 19).

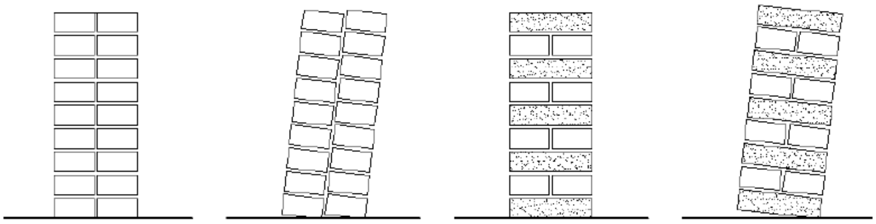


Figure 19. Role of connectors

Another important role of the connectors refers to the uniform distribution of vertical loads. When there are no connectors, the load from the floor structure can cause the instability of the individual walls (Figure 20).

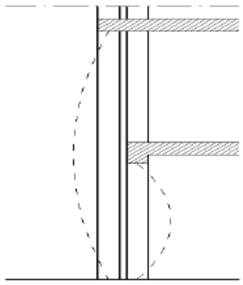


Figure 20. Instability of individual walls without connectors

The proposed classification describes different types of walls based on the value of the shear strength τ_k. These values are in accordance with the values specified in the seismic standards. Types of the walls relate to existing buildings, and class A includes masonry structures built in accordance with HRN EN 1998-1 [50] which are stronger than others, due to quality materials and building techniques. The classification procedure described above is summarized in the special schedules shown on the photos in Annex A, which indicate the class to which the building belongs. It is important to emphasize that the class determined in this way is only a reference indication for the assessor, who will from time to time have to assess the most accurate class according to the characteristics of the examined structure, according to his sensitivity and experience.

The four classes are defined as follows:

Class A:

- Homogeneous masonry with cut stone well square shaped and well organized. Mortar of good quality.
- It is made of low porosity sand stones with well-organized square blocks and with filled vertical and horizontal joints. Cement mortar has good quality.
- Hollow concrete blocks (a percentage of cavities between 15 and 45%) well laid out and filled with vertical and horizontal joints. Cement mortar has good quality.
- Made of full brick well laid out and with filled vertical and horizontal joints. Cement mortar has good quality.

- Strengthened in accordance with the new seismic standards (in the case of interventions that are not strictly performed according to these requirements it is necessary to determine the class that best suits the achieved quality).
- Reinforced according to HRN 1996-1 [53].

Class B:

- Masonry made of squared stone, inhomogeneous but well distributed and executed. Mort of good quality.
- Masonry made of rough stone with continuous layers of full brick throughout the thickness of the wall. Good organization of blocks and mortar of good quality.
- Masonry made of low porosity sandstone with well-organized square blocks and with filled vertical and horizontal joints. Cement mortar of good quality.
- Hollow concrete blocks or clay bricks (a percentage of holes between 15 and 45%) well organized but with only horizontal joints filled with good quality cement mortar.

Class C:

- Masonry made of roughly squared stone and generally well organized and constructed. Mortar of medium quality.
- Masonry with irregular or rounded stone with continuous layers of solid bricks or concrete over the entire thickness of the wall. Mainly good layout of blocks and mortar of medium quality.
- Facade wall of mostly square stone or brick on the outside and gravel or broken stone on the inside. Mainly good layout of blocks and mortar of medium quality.
- Hollow wall with stone or sand infill well organized and a large number of wall connectors (ties or thick layers). Mainly good layout of blocks and mortar of medium quality.
- Masonry made of regular square sandstone of medium porosity. Mainly good layout of blocks and mortar of medium quality.
- Masonry made of solid bricks. Poorly organized masonry. Poor quality mortar.

Class D:

- Masonry made of irregular stone or sandstone of medium to high porosity of approximately square form (rounded, river gravel, rough cast stone, irregularly shaped sandstone blocks, etc.). Poorly organized masonry without any layers that would affect the entire thickness of the wall. Poor quality mortar due to poor preservation.
- Masonry made of roughly shaped stone with continuous layers of solid bricks or concrete over the entire thickness of the wall. Poor layout of blocks and mortar of poor quality.
- Brick of poor quality with gravel parts. Poorly distributed masonry. Poor quality mortar.
- Hollow wall with filling of irregular stone or high porosity sand stones with incoherent fill or parts with holes. Lack or poor quality connection between the two faces. Poor quality mortar due to poor preservation.
- Hollow concrete blocks or clay bricks with a percentage of holes greater than 45%.

4.3.3 Conventional resistance

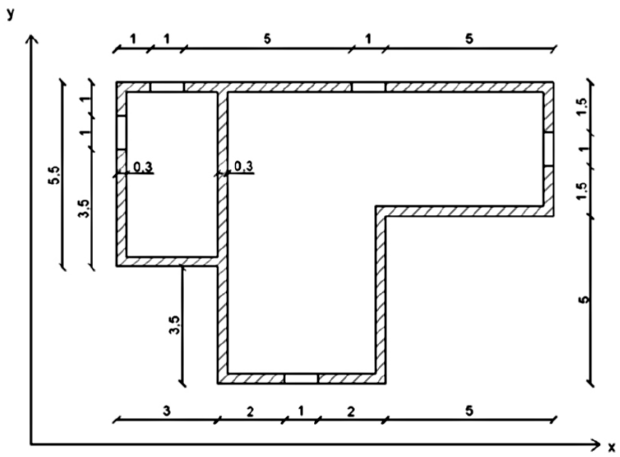
In the case of the perfect box behaviour the seismic resistance of masonry buildings can be determined quite reliably. The following procedure is necessary simplification and requires the collection of the information below relevant to the current floor:

- N - number of floors
- A_{uk} - total area of the building
- A_x, A_y - the total surface of the walls in two orthogonal directions. The length of the walls is measured from axis to axis. The wall surface with an inclination relative to the main line x or y is obtained multiplied by $\cos^2\alpha$. They are referred to as A (minimum value between A_x and A_y) and B (maximum value between A_x and A_y) so a_0 and g are defined as:

$$a_0 = \frac{A}{A_{uk}}, \ \gamma = \frac{B}{A} \qquad (2)$$

Furthermore, we define the size C as the ratio of the shear resistance at the level of the relevant floor and the total weight (which is the first estimate of the maximum bearing capacity of the building's equivalent one degree of freedom model expressed in the acceleration units):

$$C = \frac{V_{Rd}}{W} = \frac{a_0 \tau_k}{qN} \sqrt{1 + \frac{qN}{1{,}5 a_0 \tau_k (1+\gamma)}} \qquad (3)$$



A_x = [(1+5+5) + 5+3+(2+2)]x0,30 = 6,9m^2

A_y = [5,5+(7+1) + 5+(1,5+1,5)]x0,30 = 6,45m^2

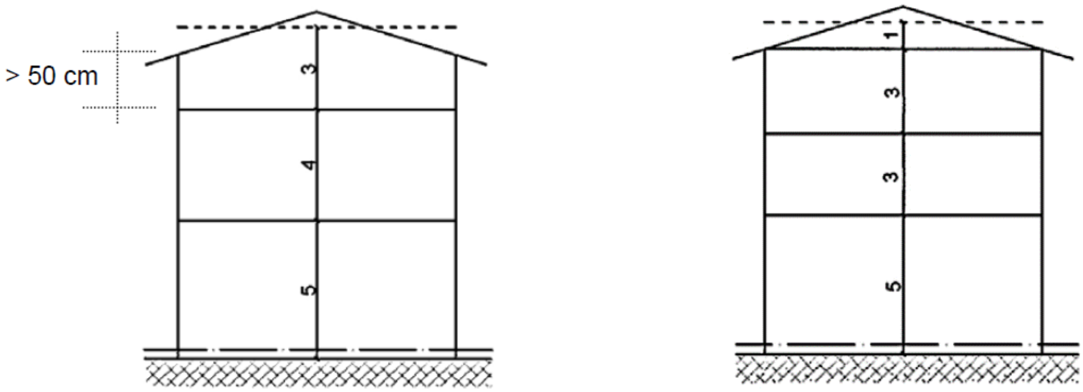
Figure 21. Example of calculation of walls' surface in two orthogonal direction

In the expression (3), the values of the τ_k resistance characteristic of the masonry type and the average weight of one floor per unit area q (sum of floor and wall weight) are displayed with already defined parameters. The average weight per unit of area q is calculated as the sum of the specific gravity weight multiplied by the average height of the floor and the total weight of the horizontal structure for the seismic combination which is presented as:

$$q = \frac{(A+B)h}{A_t} g_{walls} + (\sum G_j + \sum \psi_{Ei} Q_i) \tag{4}$$

The total weight of horizontal structures in seismic combination $(\sum G_j + \sum \psi_{Ei} Q_i)$ consists of a part related to permanent load $\sum G_j$ and the part caused by variable action $\sum \psi_{Ei} Q_i$. The combination coefficient for variable action ψ_{Ei} is calculated as $\psi_{Ei} = \varphi \cdot \psi_{2i}$, where ψ_{2i} is a combination coefficient for quasi-permanent value of variable actions defined in EN 1990, while φ is a coefficient depends on categories of the floor defined in EN 1991. Variable loads taken from Croatian standard HRN EN 1991-1 [56] is given in Table 1 of Annex C.

In the case of masonry buildings, the first addend is obviously dominant: accordingly, such an approximation in weight estimation does not involve significant errors. Although the expression (4) is derived from a hypothesis of uniform mass distribution by height of the building, according to the spirit of the seismic vulnerability estimation method of the building, the same can be used even when this hypothesis is not strictly confirmed. In this case, when estimating the average weight per unit area q, it should be kept in mind that the multiplication NqA must be equal to the total weight of the part of the building above the relevant floor section. If the in situ test does not exist, characteristic shear strength τ_k should be taken from the tables in Annex B which are taken from the technical regulations for masonry buildings from construction period of the building.



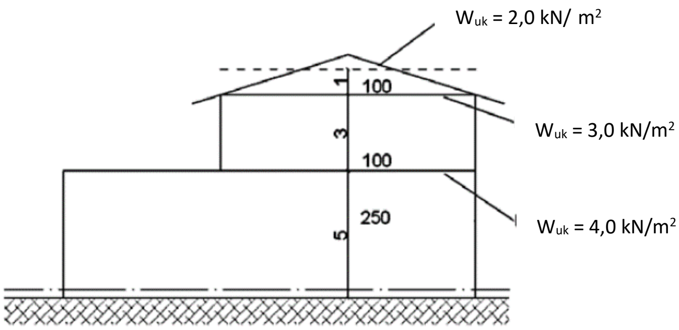
a)

b)

N = 3
h = (5 + 4 + 3)/3 = 4,0 m

N = 4
h = (5 + 3 + 3 + 1)/4 = 3,0 m

Figure 22. Examples of calculating the number of floors and average height of the floor



N = 3
$h = (5 + 3 + 1)/3 = 3{,}0m$
$A_{uk} = (250 + 100 + 100)/3 = 150m^2$
$W_{uk} = G + 0{,}3Q = (250*4{,}0 + 100*3{,}0 + 100*2{,}0)/(250 + 100 + 100) = 3{,}3\ kN/m^2$

Figure 23. Examples of calculating average weight of the floor

For brick walls, the value ranges from 0,06 to 0,12 MPa, and is higher for well-preserved walls. In the case of stone walls, the value of 0,02 MPa for the walls of irregular stone blocks is given, and for the regular blocks an interval of 0,07 – 0,09 MPa is recommended: τ_k = 0,07 Mpa for rough cast stone and 0,09 for cut stone in a homogeneous and high quality masonry. For sandstone masonry the proposed range is 0,02 to 0,10 MPa. For walls with filling τ_k = 0,02 MPa may be adopted if the mortar is of poor quality, poor performance and only horizontal joints are filled. It can be assumed that τ_k = 0,03 MPa for the walls of the same properties but without filling (poor quality, poor performance and only filled horizontal joints). The value τ_k = 0,04 MPa is adopted if the mortar is of poor quality but filled with vertical joints and is high quality, homogeneous and well-constructed.

Wall design can be extremely diverse, especially in subsequent reconstruction. It is not a rare case for walls of different materials. In such situations, the value τ_k as the weight average of the values given in the tables for different materials used should be determined. The assessment of the building class is based on the relation $\alpha = C/C'$ between the value C obtained as above and the reference values C' = 0.38g, which corresponds to the maximum peak ground acceleration measured at the rock soil in Croatia according to seismic hazard map for return period T=475 years [50]. Four classes are defined as follows:

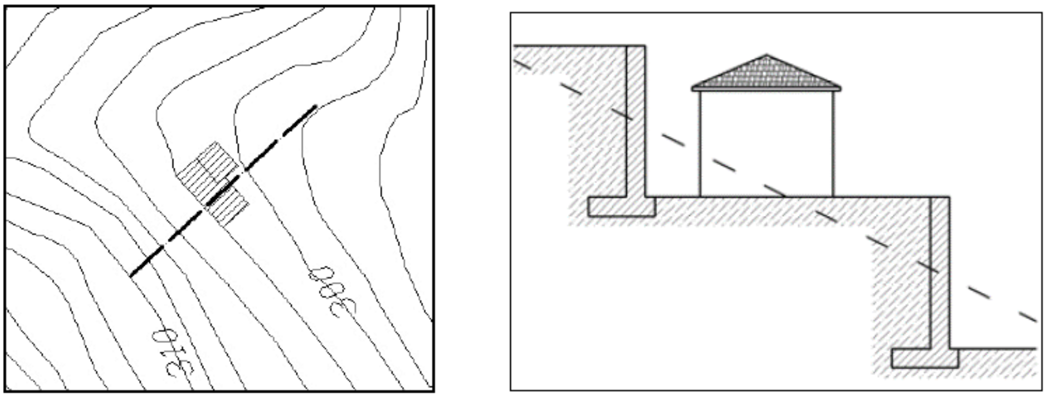
Class A: $\alpha \geq 1$;
Class B: $0{,}6 \leq \alpha < 1$;
Class C: $0{,}4 \leq \alpha < 0{,}6$;
Class D: $\alpha < 0{,}4$.

4.3.4 Position of the building and foundations

This parameter seeks, as far as possible, visually assessing the influence of soil and foundations. We therefore consider only some aspects: consistency of soil and slope of the terrain, eventual different elevation of foundation and soil coherence.

- Slope of the terrain: It is necessary to determine the section of terrain on which the building is located vertically to the isohipses.

- Soil consistency: This information can be obtained either from the Geotechnical Survey if it was part of the building project, or to take values from experience from the example of the surrounding terrain or ultimately by visual evaluation. It is necessary to determine whether the soil is rocky, even if dominated by partially broken surface deposits. Soft soil refers to the other cases, and here we differentiate the coherent and incoherent soil.
- Foundations: even foundation tie beams or extended masonry wall bases can be considered as footings.



a) *b)*

Figure 24. Determining the slope of the terrain

Four classes are defined as follows:

Class A:

- Buildings on a rock, with or without foundation, with a slope p ≤ 10% and an arbitrary difference of the foundation elevation Δh.
- Buildings on soft coherent soil inclined up to 10%, but no difference in foundation elevation.

Class B:

- Buildings on a rock, with or without any foundation, with a slope from 10% to 30%.
- Buildings on a soft, coherent soil with differences in the foundation elevation Δh not larger than 1m and with the following characteristics: with or without foundation with slope less than 10%; the building has a foundation, a terrain with an inclination from 10% to 30%; the building has no foundation and the terrain slope is from 10 to 20%;

Class C:

- Buildings with or without foundation, located on a rock with a slope from 30% to 50%;
- Buildings on a soft soil with a differences in foundation elevation less than 1m, which meet one of the following conditions: coherent soil, the building has a foundation and the ground has a slope from 30% to 50%; coherent soil, the building has no foundation and the slope of the

terrain is 20% to 30%; incoherent soil, the building has a foundation and the terrain has a slope of up to 50%; incoherent soil, the building has no foundation, and the ground has a slope less than 30%;

Class D:

- Buildings located on soft soil or rock with slope more than 50%
- Buildings on a soft soil with difference of foundation elevation greater than 1 m
- No foundation-based buildings with a slope of more than 30%.

For the purpose of determining the class, the most unfavourable condition is considered. In the case of a building on a rock, no possible differences in the elevation of the foundation is considered. When it is found that a rock-based building has foundation tie beams or an expanded masonry foundation, slope borders are taken for soft ground instead of rock. Related to this point it should be emphasized that this division does not involve the presence of landslides or liquefaction: in these cases, in reality, a ban on land use was foreseen for construction purposes except with particularly severe and delicate interventions. Furthermore, for cases of liquefaction, more detailed studies are required that go beyond the level of assessment in question. It is clear that the difference between stable and unstable soils, in this context, is essentially related to the possibility that a seismic event causes differential settlements.

4.3.5 Typology of floors

The quality of the horizontal structure is of great importance in ensuring the favourable performance of vertical elements. On the other hand, it is not an unusual case of a building where only horizontal structures, though well-constructed, have collapsed. When dividing buildings in different classes, it is especially important to check the following requirements for each floor:

- rigid diaphragm behaviour;
- effectively connected to vertical elements.

Four classes are defined as follows:

Class A: buildings with floor structures of any kind provided they meet the following three conditions:

- negligible deformability;
- effective connections between the floor structure and the wall;
- no staggered floors;

Class B: buildings with floor structures of class A but staggered;

Class C: buildings with horizontally deformable floor structures provided they are well-connected to the walls;

Class D: buildings with floor structures poorly connected to the walls.

When there are more types of floor structure in the same building, the class is defined for the worst of them, unless its surface is negligible.

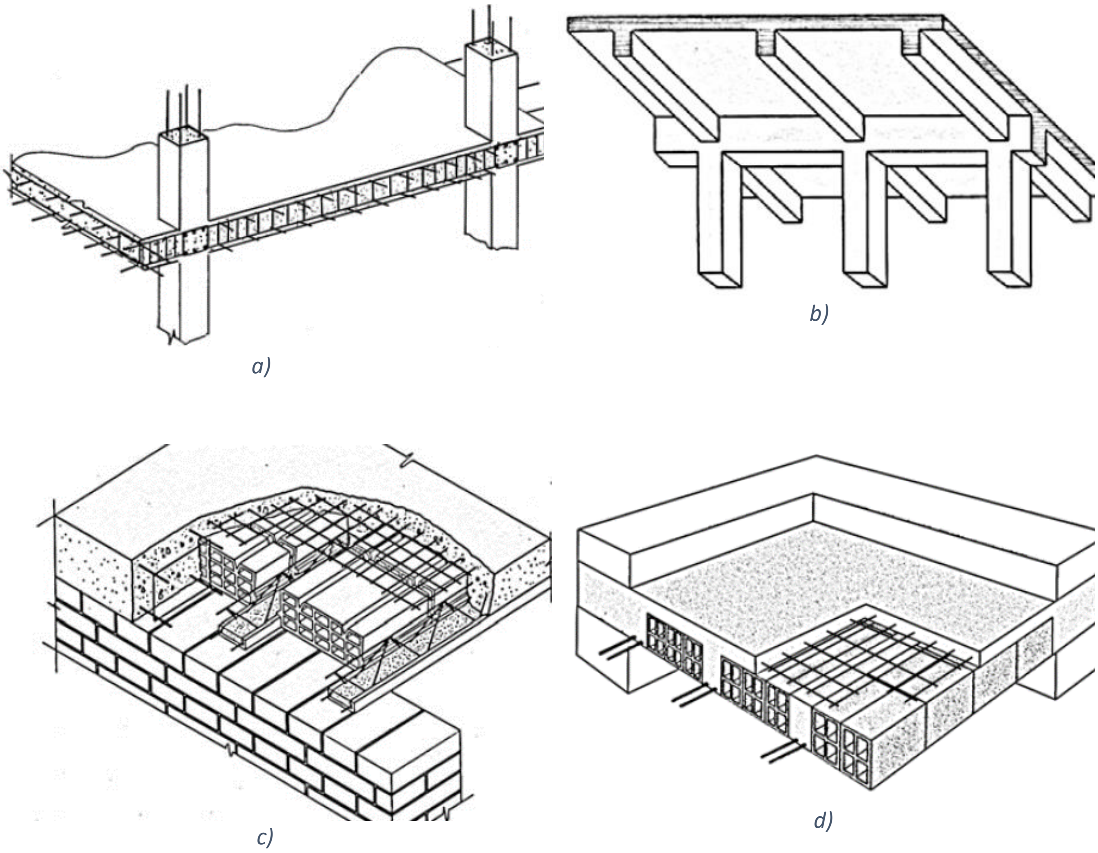


Figure 25. Rigid diaphragms

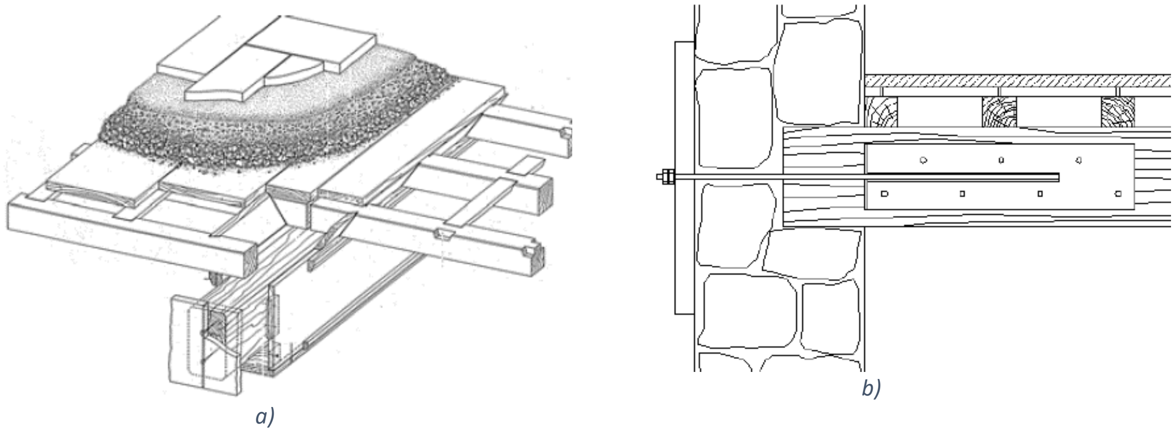


Figure 26. Deformable floors

4.3.6 Planimetric configuration

The seismic response of the building depends on its ground plan itself as well the other factors. For rectangular buildings, the ratio of shorter and longer sides is presented with $\beta_1 = a/l \cdot 100$ (%) (Figure 27). In the case of a buildings that deviates from the rectangular shape, except for this main ratio, the size of this deviation must also be taken into account: this can be done using the parameter β_2.

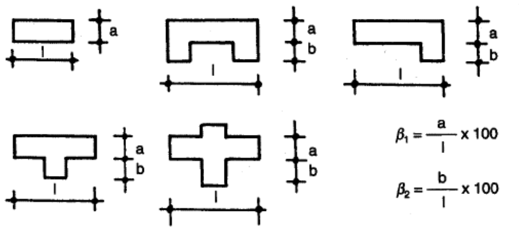


Figure 27. Examples of calculating β_1 and β_2

The assignment of a building to the various classes takes place based on the most unfavourable conditions in the verification plan set by the parameters β_1 and β_2 as follows:

Class A: $\beta_1 \geq 80$ and $\beta_2 < 10$;
Class B: $60 \leq \beta_1 < 80$ and $10 \leq \beta_2 < 20$;
Class C: $40 \leq \beta_1 < 60$ and $20 \leq \beta_2 < 30$;
Class D: $\beta_1 < 40$ or $\beta_2 \geq 30$.

4.3.7 Elevation configuration

In the case of masonry structures, especially older, the main cause of irregularity is the porches, loggia and roof terraces. The presence of the porches is entered as the percentage ratio of the area of the porch (with columns) and the total floor area.

The second element to be evaluated for the purpose of irregularity is the presence of towers of significant height and mass in relation to the remaining part of the building (the ratio between the height of the T tower and the total height of the H building in percentage); smaller towers (chimneys, etc.) are not taken into account.

The ratio of the weight variation $\pm\Delta M/M$ is taken into account where ΔM is mass variation between two adjacent floors, and M is the mass of the lower one.

The ratio $\pm\Delta M/M$ can be replaced with $\pm\Delta A/A$ where A and ΔA are area of the floor and area variation. The guiding criteria for class assignment is the one related to the worst condition. Four classes are defined as follows:

Class A:

- buildings with an even distribution of mass and bearing elements by height;
- buildings in which the mass and the structural elements are continuously reduced by height;

- buildings with retracted parts not exceeding 10% of the ground floor.

Class B:

- buildings with porches and loggia of modest dimensions up to 10% of the floor area;
- buildings with retracted parts from 10% to 20% of the floor area;
- buildings with towers up to 10% of the building height.

Class C:

- buildings with porches and loggia from 10% to 20% of the floor area;
- buildings with retracted parts larger than 20% of the floor area;
- buildings with tower height from 10 to 40% of the height.

Class D:

- buildings with porches and loggia exceeding 20% of the floor area;
- buildings with towers exceeding 40% of the height.

For walls made of different materials at different levels it will be considered in such a way that:

- buildings belonging to the class A or B according to geometry are classified as class C;
- buildings that belong to class C according to geometry are classified as class D.

4.3.8 Maximum distance among the walls

This parameter takes into account the spacing of the main walls and the transverse walls. The classes are defined in relation to the ratio between the transverse walls and the thickness of the main wall:

Class A: buildings with distance/thickness ratio less than 15;

Class B: buildings with distance/thickness ratio from 15 to 18;

Class C: buildings with distance/thickness ratio from 18 to 25;

Class D: buildings with distance/thickness ratio exceeding 25.

4.3.9 Roof structure

The roof characteristics that affect the seismic response of the building are type and mass. The type defines the class, and the mass determines the weight that belongs to this parameter. The necessary elements of assessment are:

- type: with horizontal reactions, with reduced horizontal reactions, and without horizontal reactions (Figure 28);
- the presence of horizontal tie beams;
- the presence of braces;
- roof dead load (G + 0,3Q);

- the length of the roof supports and the extent of the roof.

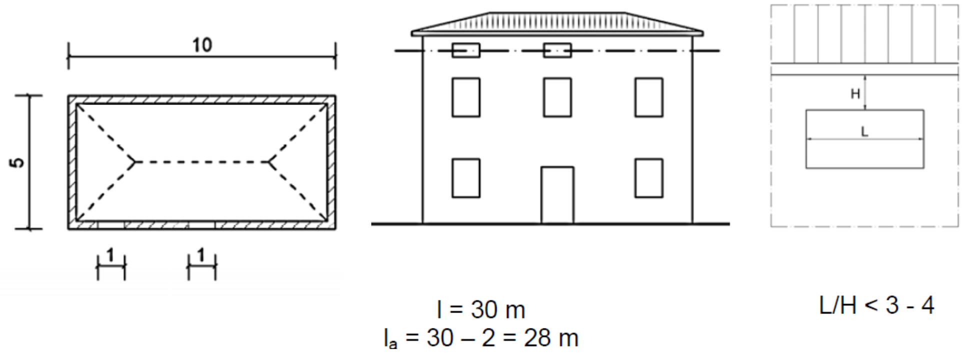


Figure 28. Elements of roof evaluation

It should be noted that the length of the roof support should not be taken into account, unless the span and height ratio is less than 4.

Classes are defined as follows:

Class A:

- Buildings without roof horizontal reactions and with tie beams and/or braces.

Class B:

- Buildings without roof horizontal reactions but with no tie beams and/or braces;
- Buildings with reduced roof horizontal reactions and with tie beams and/or braces.

Class C:

- Buildings with reduced roof horizontal reactions but without tie beams and/or braces;
- Buildings with roof horizontal reactions and with tie beams and/or braces.

Class D

- Buildings with roof horizontal reactions but without tie beams and/or braces.

4.3.10 Non-structural elements

This parameter takes into account the shutters, appendices and projections that can cause material loss or the consequences of life and human health in the event of their demolition. It is a secondary element for the purpose of assessing vulnerability for which there is no sense to distinguish between the first two classes. The classes are defined as follows:

Classes A and B:

- buildings without shutters, appendices, projections or suspended ceilings;
- buildings with shutters well connected to the walls, smaller chimneys or well-connected suspended ceilings;
- buildings with balconies well connected to the floor structure.

Class C:

- buildings with shutters or smaller signs poorly connected to the walls and with poorly connected suspended ceilings smaller size or larger ones but well connected.

Class D:

- buildings which have: chimneys or other appendices poorly connected to the structure, parapet walls or other elements with significant weight which can fall down in the case of earthquake;
- buildings with balconies or other projections which were added after the main structure was built and poorly connected to it;
- buildings with large and poorly connected suspended ceilings.

4.3.11 State of conservation

This parameter takes into account the state of preservation. Four classes are defined as follows:

Class A: Masonry in a good condition without any visible damage.

Class B: Building with cracks that are not caused by an earthquake, capillary size and slightly widely spread.

Class C:

- Medium sized cracks (up to 2-3 mm) that are not of seismic origin, or with capillary cracks of seismic origin;
- Buildings without cracks but with significantly reduced wall strength.

Class D:

- Floors with a vertical deviation due to tilting or having larger cracks that do not have to be widely spread;
- Buildings characterized by serious material decay;
- Buildings without cracks but with seriously reduced bearing capacity.

5 Development of a damage-vulnerability-peak ground acceleration vulnerability curves

5.1 Vulnerability model

The vulnerability index is not a relevant indicator of seismic risk because it does not give information about the behaviour of the building under a specific seismic action. The seismic risk of buildings is often expressed by the damage caused by an earthquake of a certain intensity. Alternatively, it can be formulated in terms of an index of seismic risk which represents seismic safety of the building. It is defined as a ratio between the peak ground acceleration corresponding to the collapse of the structure and the demand ground acceleration. In this project, both approaches have been applied.

Several studies established correlations between the vulnerability index, peak ground acceleration, or macro-seismic intensity, and the damage index. They present a cause-effect relation, where the earthquake is the cause and the damage is the effect. Therefore, vulnerability relates the ground acceleration to a certain level of damage. Two limit levels of acceleration are important for the damage analysis: the acceleration corresponding to the beginning of the damage to a structure, and the acceleration corresponding to the collapse. The level of damage varies in the [0, 1] space.

This study uses the approach developed by Guagenti and Petrini [23], who derived a relation between damage, acceleration, and the vulnerability index by observing the damage to masonry buildings under real earthquakes. They studied a set of damaged masonry buildings in the historical city centre of the towns of Venzone (Udine, Intensity IX MCS, May 1976 earthquake), Tarcento, and San Daniele (Udine, Intensity VIII MCS, May 1976 earthquake); they also considered some other buildings from the 1984 Parco d'Abruzzo earthquake (Intensity VII MCS). The level of damage to each building, as well as the level of the ground acceleration, were accounted for. Corresponding acceleration/damage laws can be represented with smooth vulnerability curves, such as the one shown in Figure 29.

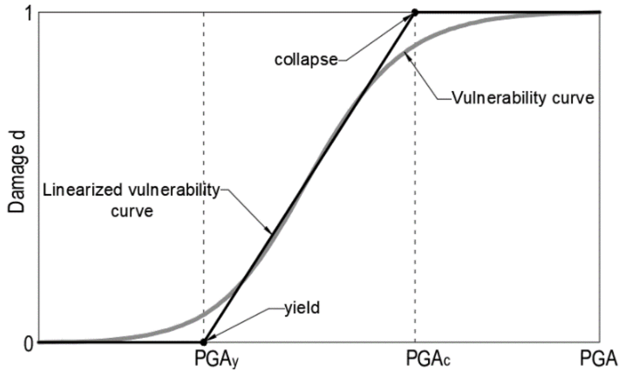


Figure 29. Vulnerability curve and its idealization

For simplicity, Guagenti and Petrini substituted the vulnerability curve with a tri-linear law defined by the values of the peak ground acceleration corresponding to early damage, PGA_i, and to the collapse, PGA_c, as follows:

$$d = \begin{cases} 0 & PGA < PGA_i \\ (PGA - PGA_i)/(PGA_c - PGA_i) & PGA_i \leq PGA \leq PGA_c \\ 1 & PGA \rangle PGA_c \end{cases} , \quad (5)$$

By means of this equation, the definition of the beginning of the damage and the collapse of buildings are related to the two values of acceleration.

In this work, instead of a field observation of the damage caused by past earthquakes, a static non-linear pushover analysis was performed for the buildings at the test site; thus, the yield and collapse acceleration were determined. Then, using the previously computed vulnerability indexes and yield and collapse accelerations, new damage-vulnerability-peak ground acceleration relationships were derived. The damage index is expressed in the [0–1] space via a tri-linear law, shown in Figure 30, defined by two points: yield acceleration PGA_y, which represents the beginning of the damage (d = 0), and acceleration for the collapse of the building PGA_c (d = 1).

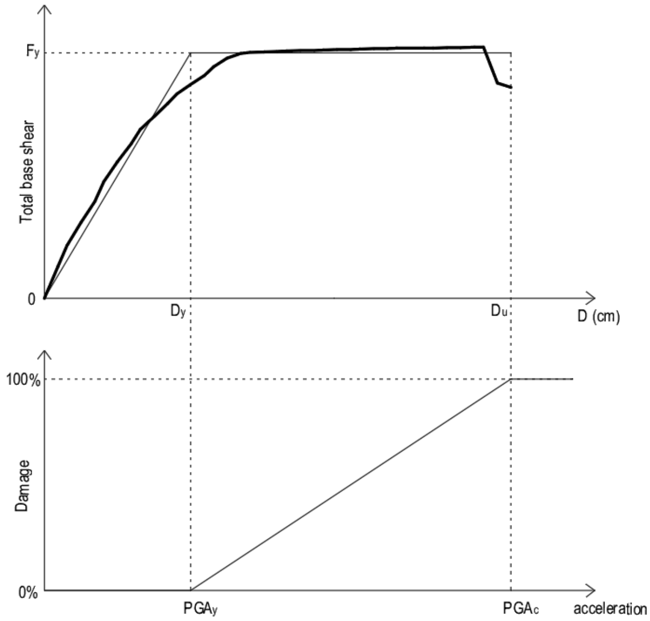


Figure 30. Pushover curve and tri-linear acceleration/damage law

5.2 Static-Nonlinear Pushover Analysis of Representative Buildings

Peak ground accelerations at the yield and the collapse of the structure were evaluated through static non-linear pushover analysis, according to Eurocode 8 [51] and the corresponding Croatian standards [50,55].

The analysis was carried out with TREMURI software [57,58], in which the building is modelled as a spatial structure where the walls resist to both vertical and horizontal loads. Walls are modelled by means of non-linear two-node macro-elements, representing whole masonry panels and piers. The macro-element considers both the shear-sliding damage failure mode and its evolution. It accounts for strength and stiffness degradation, and rocking mechanisms, whereas the toe crushing effect is modelled by means of a phenomenological non-linear constitutive law with stiffness degradation under compression.

The horizontal structures, such as floors, vaults and ceilings, transfer their vertical loads to the walls and transmit the horizontal actions to the walls. In this way, the structure is modelled by assembling the walls and the horizontal structures, both lacking bending rigidity outside of the level. Horizontal structures can have membrane stiffness or can be regarded as rigid. For each slab's typology, the connection of the structural components should be defined. A good connection to the masonry contributes to the resistance of the global system. In addition, the floor can divide its mass in a single direction or along two directions of the floor. If the floors possess bi-directional stiffness, it is necessary to indicate the vertical load percentage for the principal direction. The floors are modelled as orthotropic membrane three-node elements, with two degrees of freedom per node (displacements u_x and u_y), which are associated with a warping direction. This enables us to model both flexible and rigid floors.

The same rules for floors also apply to roofs. A roof can be modelled as a part of a bearing system or just as a load-distributing frame.

The response of the structure is investigated along the two geometrical orthogonal axes, in both the positive and negative directions. Non-regular distribution of the masses inside the structure is considered by means of the assumption of an eccentricity of the lateral loads, equal to ±5% of the maximum floor dimension at each level. Three lateral load distributions - uniform, linear and modal distribution considering positive and negative eccentricities led to a total of 36 analyses.

Each pushover analysis resulted in an MDOF capacity curve, which represents the relation between base shear force and the displacement of a control node placed at the top of the building. The pushover curve was scaled according to the N2 method described in Annex B of Eurocode 8 [50] using the transformation factor $\Gamma = \Sigma m_i\Phi_i/\Sigma m_i\Phi_i^2$, where Φ_i is the i-th component of the eigenvector and m_i is the mass of the node i. For the actual base shear force F and the corresponding top displacement of the structure d of the MDOF system, the values $F^* = F/\Gamma$ and $d^* = d/\Gamma$ represent the base shear force and the displacement of the equivalent SDOF system, respectively.

After the transformation of the MDOF curve in the SDOF one, a bilinear force-displacement diagram was obtained. The yield force F_y^*, representing the actual strength of an idealized system, is equal to the base-shear force at the formation of the plastic mechanism. The initial stiffness was determined assuming an area equivalence between the equivalent and the bilinear system. The yield displacement of the bilinear SDOF system $d_y^* = 2(d_m^* - E_m^*/F_y^*)$ was obtained from the deformation energy E_m^* up to the formation of the plastic mechanism. The mass m^*, the stiffness k^*, and the period T^* of the equivalent SDOF system can be obtained as follows:

$$m^* = \sum_{i=1}^{n} \Phi_i^* m_i \; ; \quad k^* = \frac{F_y^*}{d_y^*} \; ; \quad T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \; , \tag{6}$$

The spectral yielding acceleration S_{ay} and the elastic spectral acceleration S_{ae} of an elastic SDOF with period T^* are calculated as:

$$S_{ay} = \frac{F_y^*}{m^*} \; ; \quad S_{ae} = S_{ae}(T^*) \; , \tag{7}$$

Reduction factor R_μ is expressed as:

$$R_\mu = \frac{S_{ae}}{S_{ay}} \; , \tag{8}$$

and used to calculate the displacement ductility factor:

$$\begin{aligned} \mu_r &= (R_\mu - 1)\frac{T_c}{T^*} + 1 \; ; \quad T^* < T_c \\ \mu_r &= R_\mu \qquad\qquad\quad ; \quad T^* \geq T_c \; , \end{aligned} \tag{9}$$

Spectral inelastic demand acceleration S_{ai} and displacement S_{di} were derived as follows:

$$S_{ai} = \frac{S_{ae}}{R_\mu(\mu_r, T)} \quad ; \quad S_{di} = \frac{\mu_r}{R_\mu(\mu_r, T)} S_{de} = \mu_r \frac{T^2}{4\pi^2} S_{ai} \; , \tag{10}$$

The values of peak ground acceleration PGA_y and collapse acceleration PGA_c are calculated from the corresponding displacements according to the following procedure.

The displacement demand d_r^* was cast as a function of the spectral elastic displacement $S_{de}(T^*)$ using the following analytical relationship:

$$\begin{aligned} d_r^* &= \left[1 + (R_\mu - 1)\frac{T_c}{T^*}\right]\frac{S_{de}(T^*)}{R_\mu} \; ; \quad T^* < T_c \\ d_r^* &= S_{de}(T^*) \qquad\qquad\qquad\quad ; \quad T^* \geq T_c \; , \end{aligned} \tag{11}$$

The reduction factor can be calculated again as a function of the actual ductility μ of SDOF system in the form:

$$\overline{R}_\mu = 1 + (\mu - 1)\frac{T^*}{T_c} \quad ; \quad T^* < T_c$$
$$\overline{R}_\mu = \mu \qquad ; \quad T^* \geq T_c \,, \tag{12}$$

Given the yielding displacement d_y^* is associated with the early damage state and ultimate displacement d_u^* with the collapse; the early damage ductility μ_y and collapse ductility μ_c are expressed as:

$$\mu_y = 1 \quad ; \quad \mu_c = \mu_u = \frac{d_u^*}{d_y^*} \,, \tag{13}$$

The associated spectral displacements can be calculated from Equation (11):

$$S_{de,y}\left(T^*\right) = \frac{d_y^* \overline{R}_\mu\left(\mu_y\right)}{\left[\overline{R}_\mu\left(\mu_y\right) - 1\right]\frac{T_c}{T^*} + 1} \quad ; \quad S_{de,c}\left(T^*\right) = \frac{d_u^* \overline{R}_\mu\left(\mu_c\right)}{\left[\overline{R}_\mu\left(\mu_c\right) - 1\right]\frac{T_c}{T^*} + 1} \,, \tag{14}$$

The spectral accelerations are given as:

$$S_{ae,y}\left(T^*\right) = \frac{4\pi}{T^{*2}} S_{de,y}\left(T^*\right) \quad ; \quad S_{ae,c}\left(T^*\right) = \frac{4\pi}{T^{*2}} S_{de,c}\left(T^*\right) \,, \tag{15}$$

According to Eurocode 8, depending on the period, the elastic spectral acceleration is defined by the following expressions:

$$S_{ae}(T) = a_g S\left[1 + T/T_B(\eta \cdot 2{,}5 - 1)\right], \quad 0 \leq T \leq T_B$$
$$S_{ae}(T) = a_g S\eta \cdot 2{,}5, \quad T_B \leq T \leq T_C$$
$$S_{ae}(T) = a_g S\eta \cdot 2{,}5 T_C / T, \quad T_C \leq T \leq T_D$$
$$S_{ae}(T) = a_g S\eta \cdot 2{,}5\left(T_C T_D / T^2\right), \quad T_D \leq T \leq 4s \,, \tag{16}$$

Generally, Equations (16) can be written in the form:

$$S_{ae}(T) = PGA \cdot f_i(T) \,, \tag{17}$$

Here, PGA=a_g is peak ground acceleration, whereas f_i (i = 1, …, 4) represents the function which defines four different branches of the elastic response spectrum and depends on the period T, soil factor S, and damping correction factor η, and the characteristics periods T_B, T_C, and T_D represent the lower and upper limits of each spectral acceleration branch. The peak ground accelerations PGA_y and PGA_c, corresponding to the yield displacement and to the ultimate displacement, respectively, can now be calculated in the form:

$$PGA_y = \frac{S_{ae,y}\left(T^*\right)}{f_i\left(T\right)} \quad ; \quad PGA_c = \frac{S_{ae,c}\left(T^*\right)}{f_i\left(T\right)} \quad , \tag{18}$$

5.3 Development of damage-vulnerability-peak ground acceleration curves

For representative buildings at the test site, evaluation of the yield and the collapse accelerations has been performed according the procedure presented in Chapter 5.2. Finally, the relationships between vulnerability index and yield acceleration and vulnerability index and collapse acceleration has established and presented by trend lines I_v–PGA_y and I_v–PGA_c. It is important to note that the obtained relationships between the vulnerability index and the accelerations have been obtained on the basis of a number of typical buildings that were analysed in detail by the pushover method. The vulnerability indexes for the buildings at the test site and the obtained trend lines have used to calculate the peak ground accelerations for early damage and collapse state for all buildings at the test site.

The next step is the definition of the vulnerability curves for the test site. Tri-linear vulnerability curves, shown in Figures 29 and 30, are determined using yield and collapse peak ground accelerations, PGA_y and PGA_c. As PGA_y and PGA_c are functions of the vulnerability index I_v, the values of PGA_y, corresponding to damage d = 0, and PGA_c, corresponding to damage d = 1, can be computed for each value of I_v. The obtained vulnerability curves were exploited to define the damage index based on the vulnerability indexes. Damage index represents a measure of seismic risk of the test site.

Developed damage-vulnerability-peak ground acceleration curves are used to estimate the damage of the buildings under specific seismic action. Therefore, they give information about seismic risk of the buildings at the test site in terms of seismic damage.

This deliverable presents the methodology that allows the damage assessment of the building stock on a territorial scale for a certain seismic action expressed in terms of peak ground acceleration. The result is achieved by combining an expeditious empirical method based on vulnerability index calculation and detailed non-linear pushover analyses.

The evaluation of seismic vulnerability represented by vulnerability indexes and seismic risk in terms of the damage for the buildings at the test site Kaštel Kambelovac, as well as corresponding vulnerability and damage index maps are given in deliverables 3.3.3 and 3.3.4.

6 Seismic risk assessment of the test site

6.1 Description of the methodology

Seismic risk assessment of existing urban area provides an important information to take action of seismic risk reduction in different phases of planning and emergency management. Between different large-scale assessment approaches, vulnerability index method is often used for the first screening of the buildings and vulnerability classification, but it does not give information about the behaviour of the building under a specific seismic action.

In this chapter methodology for assessment of seismic risk at the test site has been proposed. It combines seismic vulnerability indexes obtained by vulnerability index method with critical peak ground accelerations for different limit states of the buildings, computed by non-linear static (pushover) analysis. The procedure has been applied at the test site Kaštel Kambelovac which consists of the historical core built between the 15th and 19th centuries and the parts outside of historical core with buildings dating from the beginning of the 20st century to nowadays.

The vulnerability assessment is based on the modified vulnerability index method [59] presented in Chapter 4 of this Deliverable. The vulnerability indexes for 111 buildings with known geometrical, structural and material characteristics (75 in old city centre and 35 outside of the centre) were calculated. To improve the interpretation of the results, vulnerability indexes, as well as other important input information, were integrated into a geographical information system (GIS) tool. Vulnerability of other buildings at the test site that had not an available technical documentations was determined based on the estimated geometric and structural characteristics of the buildings using a geodetic survey of the area, a street view map and a visual inspection of the area.

Static non-linear pushover method [1,2] are used to evaluate seismic behaviour and capacity of the building for three limit state (LS) conditions that have been taken into account according to Eurocode 8, part 3 [55] as follows:

- Near collapse NC: global capacity of the building is taken equal to the ultimate displacement capacity;
- Significant damage SD: global capacity of the building is taken equal to ¾ of the ultimate displacement capacity;
- Damage limitation DL: the capacity for global assessment is defined as the yield point of the idealized elastic-perfectly plastic force-displacement relationship of equivalent SDOF.

Static non-linear (pushover) analysis have been applied for 19 buildings at the test site. The sample includes stone masonry buildings in the old centre as well as buildings built of concrete and brick blocks with and without horizontal and vertical confinement. The peak ground accelerations associated with the DL, SD and NC limit states were computed for x and y directions. The lowest PGA values were identified for each building and limit state.

Vulnerability indexes for analysed buildings were linked with the critical PGA results. The vulnerability index – peak ground acceleration relations for early damage, significant damage and near collapse states were establish. Derived relations are used to estimate the critical peak ground accelerations using the vulnerability index parameters for the buildings which are not analysed in detail by pushover analysis.

Finally, the methodology is used to estimate seismic risk in terms of an index of seismic risk which validate safety of the structures for selected return periods. This is a base for the seismic risk management actions. The index of seismic risk is defined as a ratio between the peak ground acceleration corresponding to the near collapse, significant damage or damage limitation states, respectively, and the demand ground acceleration. The index of seismic risk for three return periods are presented in the web maps of the test site.

6.2 Calculation of critical peak ground accelerations

The values of critical peak ground accelerations for DL, SD and NC limit states are calculated from the corresponding displacements by non-linear (pushover) analysis according to the following procedure.

The procedure is based on the calculation of ductility for DL, SD and NC limit states expressed as:

$$\mu_{DL} = \mu_y = 1 \quad ; \quad \mu_{SD} = \frac{\frac{3}{4} d_u^*}{d_y^*} \quad ; \quad \mu_{NC} = \mu_u = \frac{d_u^*}{d_y^*} , \tag{13}$$

The associated spectral displacements can be calculated from Equation (13):

$$S_{de,DL}\left(T^*\right) = \frac{d_y^* \bar{R}_\mu\left(\mu_{DL}\right)}{\left[\bar{R}_\mu\left(\mu_{DL}\right) - 1\right]\frac{T_c}{T^*} + 1}$$

$$S_{de,SD}\left(T^*\right) = \frac{\frac{3}{4} d_u^* \bar{R}_\mu\left(\mu_{SD}\right)}{\left[\bar{R}_\mu\left(\mu_{SD}\right) - 1\right]\frac{T_c}{T^*} + 1}$$

$$S_{de,NC}\left(T^*\right) = \frac{d_u^* \bar{R}_\mu\left(\mu_{NC}\right)}{\left[\bar{R}_\mu\left(\mu_{NC}\right) - 1\right]\frac{T_c}{T^*} + 1} , \tag{14}$$

The spectral accelerations are given as:

$$S_{ae,DL}\left(T^*\right)=\frac{4\pi}{T^{*2}}S_{de,DL}\left(T^*\right)$$
$$S_{ae,SD}\left(T^*\right)=\frac{4\pi}{T^{*2}}S_{de,SD}\left(T^*\right)$$
$$S_{ae,NC}\left(T^*\right)=\frac{4\pi}{T^{*2}}S_{de,NC}\left(T^*\right), \qquad (15)$$

According to Eurocode 8, depending on the period, the elastic spectral acceleration is defined by the following expressions:

$$S_{ae}\left(T\right)=a_g S\left[1+T/T_B\left(\eta\cdot 2{,}5-1\right)\right], \quad 0\le T\le T_B$$
$$S_{ae}\left(T\right)=a_g S\eta\cdot 2{,}5, \quad T_B\le T\le T_C$$
$$S_{ae}\left(T\right)=a_g S\eta\cdot 2{,}5T_C/T, \quad T_C\le T\le T_D$$
$$S_{ae}\left(T\right)=a_g S\eta\cdot 2{,}5\left(T_C T_D/T^2\right), \quad T_D\le T\le 4s, \qquad (16)$$

Generally, Equations (16) can be written in the form:

$$S_{ae}\left(T\right)=PGA\cdot f_i\left(T\right), \qquad (17)$$

Here, PGA=a_g is peak ground acceleration, whereas f_i (i = 1, …, 4) represents the function which defines four different branches of the elastic response spectrum and depends on the period T, soil factor S, and damping correction factor η, and the characteristics periods T_B, T_C, and T_D represent the lower and upper limits of each spectral acceleration branch. The peak ground accelerations PGA_{DL}, PGA_{SD} and PGA_{NC}, corresponding to the displacements for three limit states, can now be calculated in the form:

$$PGA_{DL}=\frac{S_{ae,DL}\left(T^*\right)}{f_i\left(T\right)}$$
$$PGA_{SD}=\frac{S_{ae,SD}\left(T^*\right)}{f_i\left(T\right)}$$
$$PGA_{NC}=\frac{S_{ae,NC}\left(T^*\right)}{f_i\left(T\right)} \qquad (18)$$

6.3 Definition of index of seismic risk

Seismic risk is expressed in terms of an index of seismic risk for selected return period. According to EC-8 seismic hazard is defined with the following parameters:

- a_g - peak ground horizontal acceleration on type A soil, $a_g = \gamma_I a_{gR}$, where γ_I depends on the importance of the building;
- S - soil parameter.

Demand ground acceleration is given as $PGA_D = a_g S$.

Index of seismic risk is defined as a ratio between the peak ground acceleration corresponding to the near collapse, significant damage or damage limitation states, respectively, and the demand ground acceleration. The indexes of seismic risk for damage DL, SD and NC collapse states are expressed in the form:

$$\alpha_{PGA,DL} = \frac{PGA_{DL}}{PGA_D}$$
$$\alpha_{PGA,SD} = \frac{PGA_{SD}}{PGA_D}$$
$$\alpha_{PGA,NC} = \frac{PGA_{NC}}{PGA_D} \tag{17}$$

where PGA_D represents demand peak ground acceleration for selected return period.

This indexes are used to validate safety of the structure. The values $\alpha_{PGA}>1$ are related to safe structures, while the values $\alpha_{PGA}<1$ are related to non-safe structures.

The indexes od seismic risk for three return periods are presented in the web maps of the test site.

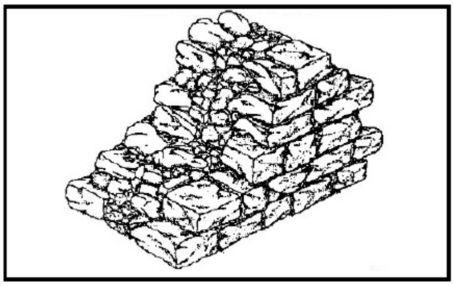
7 Conclusion

This Deliverable presents a procedure for assessing the seismic vulnerability of the historical towns and settlements located along the Croatian side of Adriatic coast. The selected test site for representation and validation of the methodology, Kaštel Kambelovac, has a typical configuration and buildings that are representative for other historical centres in Dalmatia, not only along the coast, but also inland. The proposed approach is based on calculation of seismic vulnerability indexes by seismic vulnerability method derived from the Italian GNDT approach, with some modifications resulting from the specificity of the buildings in the investigated area. The vulnerability indexes are combined by the results of numerical investigations of the behaviour of typical buildings with non-linear pushover analysis. A new vulnerability-peak ground acceleration relations and damage-vulnerability-peak ground acceleration relations has been developed using the vulnerability indexes and the yield, significant damage and collapse accelerations of buildings obtained through non-linear static analysis. Developed procedure presents a basis for seismic risk assessment based on calculation of the seismic vulnerability index, damage index, critical accelerations and index of seismic risk for the yield, significant damage and collapse states of the buildings at the test site.

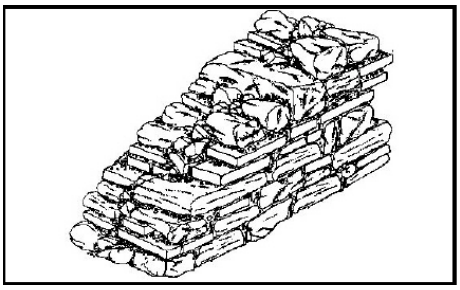
Appendix A: Masonry typology

Appendix A presents the typology of masonry and the corresponding classification for organized and disorganized masonry and the appropriate quality of the mortar. The typology and classification have been adopted according to GNDT manual [54].

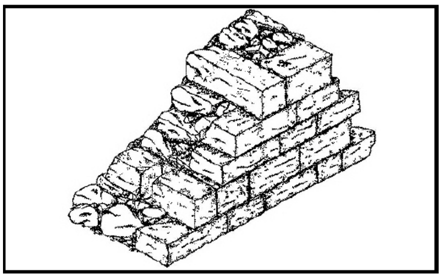
1	Hollow wall formed by stones of various sizes, disorganized and without connection between the two faces. CLASS D



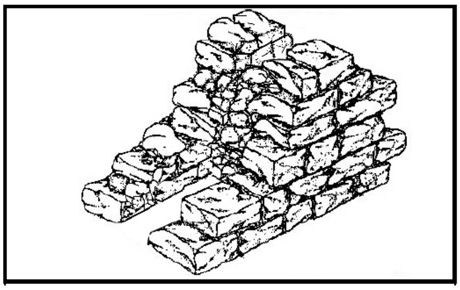
DESCRIPTION: Hollow wall with incoherent infill without wall connectors (ties) between two faces.



DESCRIPTION: Hollow wall with layers of squared stone or solid bricks that do not extend over the entire wall width.



DESCRIPTION: Hollow wall with non-degraded infill with small number of wall connectors (ties) between two faces.



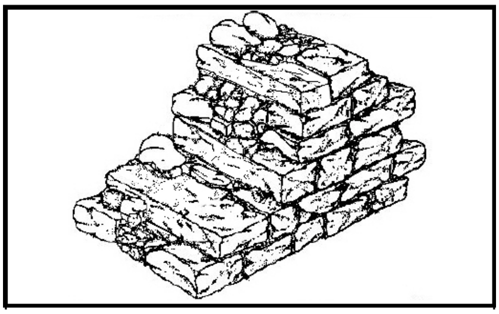
DESCRIPTION: Hollow wall with partially empty or strongly degraded infill.

DESCRIPTION: Hollow wall with roughly hewn stone of various sizes.

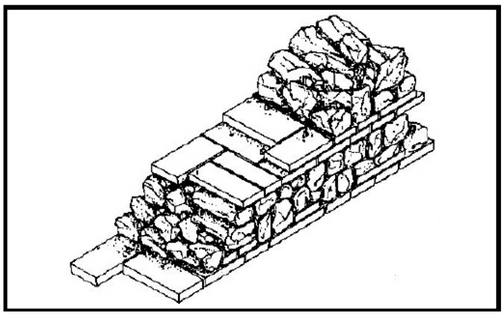
DESCRIPTION: Hollow wall with non-squared disorganized stones.

Well organized masonry		Disorganized masonry	
D		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
D	D	D	D

2	Hollow wall formed by stones of mostly uniform sizes, well organized and without connection between the two faces, with edges, door frames and/or layers in squared stone or solid bricks. CLASS C or D



DESCRIPTION: Wall with large number of connecting elements (ties) between the two faces.



DESCRIPTION: Wall with layers of squared stone or solid bricks that do extend over the entire wall width.

DESCRIPTION: Hollow wall with squared stone of uniform sizes with ties between the two faces.

DESCRIPTION: Hollow wall well organized and executed in all directions.

Well organized masonry		Disorganized masonry	
C		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
C	D	D	D

3	Roughly hewn stone masonry in the presence of irregularities. CLASS C or D

DESCRIPTION: Made of unworked stone elements of various sizes obtained from pieces of stone from quarry.

DESCRIPTION: Composed of elements of various types and sizes of rough stone, with inserts of bricks.

DESCRIPTION: Composed of non-squared stone elements of various sizes arranged in a chaotic manner, in the absence of horizontal rows.

DESCRIPTION: Composed of tufa elements, very porous, of irregular size and disorganized.

DESCRIPTION: Composed of half worked elements mostly flat and poorly organized.

DESCRIPTION: Composed of half worked elements mostly flat and organized in layers.

Well organized masonry		Disorganized masonry	
C		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
C	D	C	D

4	Roughly hewn stone masonry in the presence of edges, door frames and/or layers made of solid bricks or squared stone. CLASS B or C

DESCRIPTION: Presence of continuous or discontinuous layers in solid bricks and with discrete squared stones.

DESCRIPTION: Presence of layers of cement conglomerate.

DESCRIPTION: Wall composed of non-squared or roughly squared stones with layers of solid bricks.

DESCRIPTION: Wall with layers of non-degraded cement conglomerate.

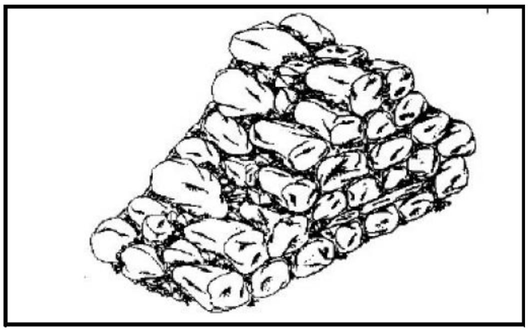
DESCRIPTION: Wall composed of roughly hewn stone with frames in solid bricks.

DESCRIPTION: Wall composed of roughly hewn stone with edges in solid bricks.

DESCRIPTION: Wall composed of sand stone with well-made corners.

Well organized masonry		Disorganized masonry	
B		C	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
B	B	B/C	D

5	Rounded stone masonry or river pebbles of various sizes without frames and/or layers in solid brick and/or squared stone. CLASS D



DESCRIPTION: Wall composed of river pebbles of small and middle sizes without layers.

DESCRIPTION: Wall composed of middle size river pebbles roughly worked.

DESCRIPTION: Wall composed of rounded stone with polished surface and without layers with inserts of bricks.

DESCRIPTION: Wall composed of various sizes river pebbles mostly organized in layers.

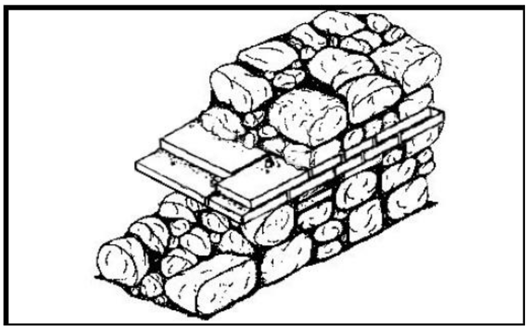
DESCRIPTION: Wall composed of various sizes river pebbles without layers.

DESCRIPTION: Wall composed of disorganized rounded stone of various sizes.

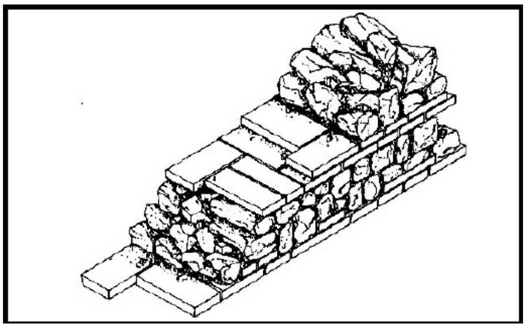
Well organized masonry		Disorganized masonry	
D		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
D	D	D	D

6 Rounded stone masonry or river pebbles of various sizes with frames and/or layers in solid brick and/or squared stone.

CLASS C or D



DESCRIPTION: Wall composed of rounded stone with polished surface and with layers of full wall width.



DESCRIPTION: Wall composed of small and middle size river pebbles with layers of full wall width.

DESCRIPTION: Wall composed of rounded stone with polished surface and with layers of solid bricks.

Well organized masonry		Disorganized masonry	
C		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
C	D	D	D

7	Walls composed of cut stones of constant sizes. CLASS A or B

DESCRIPTION: Wall made of natural well worked homogenous squared stones accurately executed.

DESCRIPTION: Wall made of natural well worked homogenous squared stones accurately executed.

DESCRIPTION: Wall composed of standard size well organized sand blocks.

DESCRIPTION: Wall composed of squared sand stones adequately textured.

DESCRIPTION: Wall composed of disorganized roughly squared sand stones.

DESCRIPTION: Wall composed of well-organized antique sand blocks.

Well organized masonry		Disorganized masonry	
A		B	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
A	B	B	C

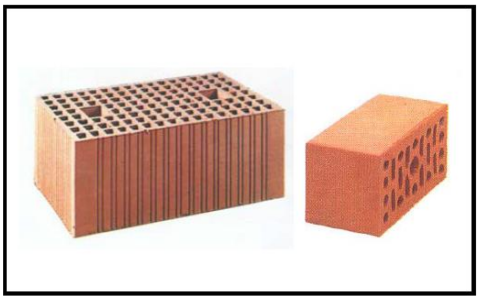
8	Masonry in prefabricated concrete blocks, with ordinary or light aggregates (expanded clay), homogeneous in all of its extents. CLASS A or B

DESCRIPTION: Composed of artificial elements in normal or lightweight concrete of standard dimensions arranged regularly. In the presence of only horizontal mortar beds assign a lower class.

Well organized masonry		Disorganized masonry	
A		A/B	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
A	B	A/B	B

9	Masonry composed of solid or hollow bricks (% of holes less than 45%). CLASS A, B or C

DESCRIPTION: Masonry in solid bricks with constant dimensions.



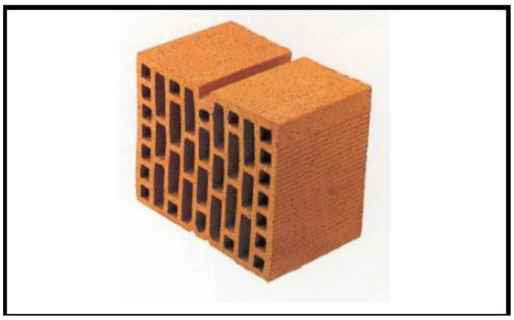
DESCRIPTION: Standard size hollow clay block and hollow brick.

DESCRIPTION: Masonry in hollow clay blocks with constant dimensions and poor quality mortar.

Well organized masonry		Disorganized masonry	
A		B	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
A	B	B	C

10	Masonry of hollow clay blocks with the percentage of holes more than 45%. CLASS D

DESCRIPTION: Wall composed of standard size artificial clay blocks.

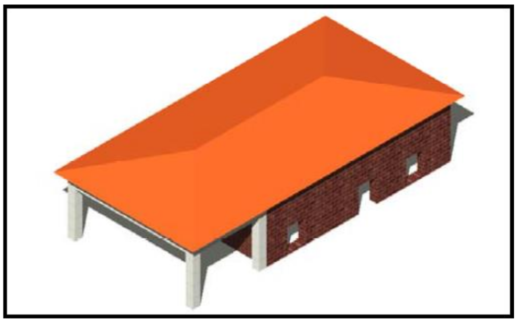


DESCRIPTION: Standard size artificial hollow clay block.

DESCRIPTION: An old type of constant size artificial blocks.

Well organized masonry		Disorganized masonry	
D		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
D	D	D	D

11	Mixed structure, meaning a combination (in the same plane) of one (or more) of the wall types in ratio 1 ÷ 10 with a reinforced concrete frame type. CLASS C or D

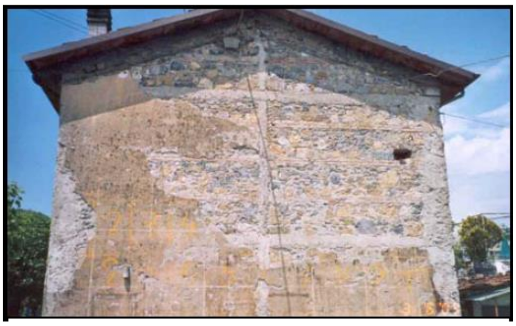


DESCRIPTION: Masonry and reinforced concrete structure combined on the same floor.

Well organized masonry		Disorganized masonry	
C		D	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
C	D	D	D

12	Confined masonry. CLASS A, B or C

DESCRIPTION: Masonry walls confined with vertical and horizontal tie beams and columns.



DESCRIPTION: Masonry walls confined with vertical and horizontal tie beams and columns.

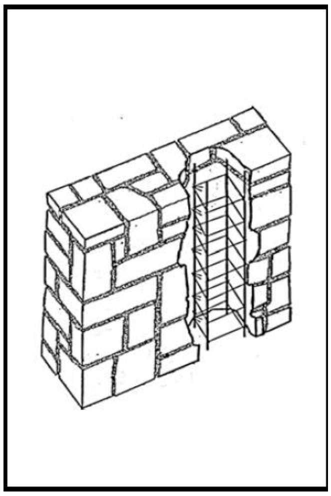
Well organized masonry		Disorganized masonry	
A		B/C	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
A	B	B	C

13	Reinforced masonry. CLASS A or B

DESCRIPTION: Masonry wall reinforced with steel bars and poured concrete.

Well organized masonry		Disorganized masonry	
A		A	
Good quality mortar	Poor quality mortar	Good quality mortar	Poor quality mortar
A	B	A	B

<table>
<tr><td>14</td><td colspan="2">Strengthened in accordance with the new seismic standards. In the case of poor quality, a lower class needs to be assigned!
CLASS A</td></tr>
<tr><td>
DESCRIPTION: Masonry reinforced with injections of binding mixtures.</td><td>
DESCRIPTION: Strengthening with reinforcement mesh and casting of concrete with a thickness of at least 5 cm on both sides connected by the bars (about 6 pcs / m2). Overlaps of at least two squares and stirrups at the corners.</td><td rowspan="2">Masonry strengthened in accordance with the new seismic norms.

In the case of poorly executed intervention, consider the class of more similar vulnerabilities due to achieved strength.
Sometimes it is not possible investigate the success of the intervention carried out, such as in the case of injections of binding mixtures. In these cases, the type of performed intervention should be found in the project data.
In the case of reinforced concrete, it is possible with essays to establish the quality of the connections between the two walls (bars must be bent to hook up the mesh) and secondly the quality of the concrete used, making sure it is not a simple plaster. The intervention of the columns must be such as not to weaken the masonry where it is put in. Problem that must be taken care of is the connection between the small columns and horizontal ties, so to create an effect of confined masonry.
The application of horizontal and/or vertical prestressed metal bars gives the wall a state of diffuse stress. This intervention is combined with strengthening of the wall through the application of a reinforced concrete.</td></tr>
<tr><td>
DESCRIPTION: Strengthening by inserting confinement tie columns or steel profiles.</td><td>
DESCRIPTION: Masonry reinforced with horizontal and/or vertical prestressed tie-rods covered in reinforced concrete.</td></tr>
</table>

Appendix B: Characteristic shear strengths of masonry walls

Two values of characteristic shear strength are given in this appendix.

The first value τ_k is taken from the original user manual [19].

The value τ_k* is taken from past building standards that were applied in the Croatian test site through different period of construction as follows:

- the values from The Temporary Technical Regulations on Brick Walls from 1949 [43];
- the values from Temporary technical regulations for construction in seismic areas (1964) [44];
- the values from The regulation on technical standards for the design and construction of prefabricated structures of unreinforced and reinforced cellular concrete (1981) [47];
- the values from EC6 [50].

For the following masonry type, the values τ_k and τ_k* are given as follows:

	τ_k (MPa)	τ_k* (MPa)
a) non-reinforced masonry, without damage		
- solid bricks with gauged mortar	0,06 – 0,12	0,03 - 0,22
- hollow clay blocks with gauged mortar	0,08	0,03 - 0,13
- normal or lightweight concrete blocks with gauged mortar	0,18	0,03 - 0,13
Stone masonry (in the presence of layers of solid bricks τ_k can be assumed 30% higher):		
- disorganized stones	0,02	/
- squared and well organized stones	0,07-0,09	/
- well-organized hollow walls	0,04	/
- sand stones	0,02-0,010	/
b) New masonry		
- solid bricks with circle holes with cement mortar and compressive strength of mortar not less than 14,5 MPa	0,20	0,20-0,40
- hollow blocks with % of holes less than 40 % with cement mortar and compressive strength of mortar not less than 14,5 MPa	0,18	0,20-0,40
c) Strengthened masonry		
- solid bricks or well organized stones strengthened with reinforced concrete with minimum thickness 3cm on both sides	0,11	/
- Stone masonry with injected binding mixtures and torcreted	0,11	/

The Temporary Technical Regulations on Brick Walls from SFRJ (1949) [43] gives compressive strength for brick walls. Shear strength is than calculated as 1/10 of the compressive strength if cement mortar is applied or 1/12 if gauged mortar is applied. The compressive stress for brick masonry wall is given in the Table 1 of this Appendix.

Table 1. The compressive stress for brick masonry wall [43]

Brick class (mean strength) (kg/cm^2)		Compressive stress for brick masonry wall (MPa)			
		In cement mortar	In general purpose mortar	In limestone	In clay mortar
	200	2,2	1,8	-	-
	150	1,6	1,4	1,0	-
	110	-	1,1	0,8	-
	70	-	0,8	0,6	0,5
(Exceptional Approval)	40	-	-	0,4	0,4
(porous)	30	-	-	0,3	0,3

For the buildings built after 1964 Temporary technical regulations for construction in seismic areas [44] has defined the compressive stress for masonry wall depending on the mark of wall elements, mark of mortar and wall slenderness (Table 2). Shear strength is calculated as 8% of compressive strength.

Table 2. The compressive stress for masonry wall [44]

Mark of wall elements	Mark of mortar	Wall slenderness h/d					
		10	12	14	16	18	20
75	5	0,40	0,35	-	-	-	-
	25	0,50	0,45	0,35	-	-	-
100	5	0,50	0,45	0,35	-	-	-
	25	0,60	0,50	0,45	0,35	-	-
	50	0,70	0,60	0,50	0,40	-	-
150	25	0,90	0,80	0,65	0,55	0,40	-
	50	1,10	0,95	0,80	0,65	0,50	-
	100	1,30	1,15	0,95	0,80	0,60	-
200	25	1,00	0,85	0,70	0,60	0,45	-
	50	1,30	1,15	0,95	0,80	0,60	-
	100	1,60	1,40	1,20	0,95	0,75	0,50

For the buildings built after 1981, The regulation on technical standards for the design and construction of prefabricated structures of unreinforced and reinforced cellular concrete [47] were applied and the compressive strength for different wall thickness is shown in Table 3.

Table 3. The compressive strength for different wall thickness [47]

Cellular concrete class	Compressive strength in MPa for different wall thickness			
	15 cm	20 cm	25 cm	30 cm
25	0,25	0,30	0,35	0,40
30	0,27	0,32	0,37	0,42
35	0,30	0,35	0,40	0,45

40	0,32	0,37	0,42	0,47
45	0,35	0,40	0,45	0,50
50	0,35	0,40	0,45	0,50
55	0,35	0,40	0,45	0,50
60	0,35	0,40	0,45	0,50

Finally, characteristic shear strengths according to EC6 [50] is given in Table 4.

Table 4. The characteristic shear strengths according to EC6 [50]

Masonry units	f_{vk0} (MPa)			
	General purpose mortar of the Strength Class given		Thin layer mortar (bed joint ≥0,5 mm and≤ 3 mm)	Lightweight mortar
Clay	M10-M20	0,30	0,30	0,15
	M2,5-M9	0,20		
	M1-M2	0,10		
Calcium silicate	M10-M20	0,20	0,40	0,15
	M2,5-M9	0,15		
	M1-M2	0,10		
Concrete	M10-M20	0,20	0,30	0,15
Autoclaved Aerated Concrete	M2,5 – M9	0,15		
Manufactured stone and Dimensioned natural stone	M1-M2	0,10		

Appendix C: Self weights and variable loads

Self-weights for masonry from the original manual [19]: (kN/m^3)

- solid clay bricks (% of holes < 15) 18,0
- hollow clay blocks (% of holes from 15 to 45) 16,0
- hollow clay blocks (% of holes > 45) 11,0
- stone masonry 22,0
- stone masonry with brick layers 21,0
- cement blocks 12,0

Variable loads taken from Croatian standard HRN EN 1991-1 [56] is given in Table 1 of this Annex.

Table 1. Variable loads, Croatian standard HRN EN 1991-1 [56]

Category	Specific use	Example	Loads [kN/m^2]
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.	1,5 to 2,0 (floors) 3,0 (stairs) 4,0 (balconies)
B	Office and working areas	Companies offices, medical offices without heavy equipment, hospitals waiting rooms	2,0
		Kitchens in hospitals, hotels etc., areas for medical treatments in hospitals, basements in domestic buildings	3,0
		As above but heavy equipment	5,0
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D)	C1: Areas with tables, etc. e.g. areas in schools, cafes, restaurants, dining halls, reading rooms, receptions.	3,0
		C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms.	4,0
		C3: Areas without obstacles for moving people, areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts.	5,0
		C4: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages.	5,0
		C5: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.	7,5
D	Shopping areas	D1: Areas in general retail shops	2,0
		D2: Areas in department stores	5,0

Appendix D: Class assessment criteria tables

The following tables summarize the criteria set out in the class assignment manual for parameters 3, 4, 5, 6, 7, 8 and 9.

Assessment of the parameter no. 3
CONVENTIONAL RESISTANCE

Ratio α = C/C'	CLASS
$\alpha \geq 1$	A
$0,6 \leq \alpha < 1$	B
$0,4 \leq \alpha < 0,6$	C
$\alpha < 0,4$	D

Assessment of the parameter no. 6
PLANIMETRIC CONFIGURATION

$\beta 1 = a/l$ (%)	$\beta 2 = b/l$ (%)	CLASS
≥ 80	≤ 10	A
≥ 60	≤ 20	B
≥ 40	≤ 30	C
< 40	> 30	D

Assessment of the parameter no. 4
POSITION OF THE BUILDING AND FOUNDATIONS

SOIL TYPE AND PRESENCE OF FOUNDATIONS	SLOPE OF THE TERRAIN p (%)	DIFF. FOUND. ELEV. Δh (m)	CLASS
1 Soil type: Rock Building with foundations	≤ 10	-	A
	≤ 30	-	B
	≤ 50	-	C
	> 50	-	D
2 Soil type: Rock Building without foundations	≤ 10	-	A
	≤ 30	-	B
	≤ 50	-	C
	> 50	-	D
3 Soil type: coherent soil Building with foundations	≤ 10	0	A
	≤ 10	< 1	B
	≤ 30	≤ 1	B
	≤ 50	≤ 1	C
	> 50	-	D
	-	> 1	D
4 Soil type: coherent soil Building without foundations	≤ 10	0	A
	≤ 10	< 1	B
	≤ 20	≤ 1	B
	≤ 30	≤ 1	C
	> 30	-	D
	-	> 1	D
5 Soil type: incoherent soil Building with found.	≤ 50	≤ 1	C
	> 50	-	D
	-	> 1	D
6 Soil type: incoherent soil Building without found.	≤ 30	≤ 1	C
	> 30	-	D
	-	> 1	D

Assessment of the parameter no. 7
ELEVATION CONFIGURATION

AREA VARIATION ΔA (%)	RATIO T/H (%)	LOGGIA AND PORCHES (%)	CLASS
≤ 10	≤ 10	0	A
≤ 20	≤ 10	≤ 10	B
> 20	≤ 40	≤ 20	C
	> 40	> 20	D

ΔA - floor area variation

For walls made of different materials at different levels it will be considered in such a way that:
- buildings belonging to the class A or B according to geometry are classified as class C;
- buildings that belong to class C according to geometry are classified as class D.

Assessm. of the param. 8
MAX DIST. AMONG THE WALLS

Ratio l/s	CLASS
≤ 15	A
≤ 18	B
≤ 25	C
> 25	D

Assessment of the parameter no. 5
TYPOLOGY OF FLOORS

Type	STAGGERED FLOORS	CLASS
1 Rigid and well connected	Ne	A
	Da	B
2 Deform. and well conn.	Ne	C
	Da	C
3 Rigid and poorly conn.	Ne	D
	Da	D
4 Deform. and poorly conn.	Ne	D
	Da	D

Assessment of the parameter no. 9
ROOF

PUSHING FORCES	TIE BEAMS	BRACES	CLASS
No	Yes	Yes or no	A
No	Yes or no	Yes	A
No	No	No	B
Reduced	Yes	Yes or no	B
Reduced	Yes or no	Yes	B
Reduced	No	No	C
Yes	Yes	Yes or no	C
Yes	Yes or no	Yes	C
Yes	No	No	D

Appendix E: Vulnerability index form

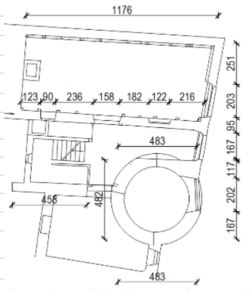
The structure of vulnerability index form is given as follows.

Building name:	Cambi tower
Building type:	Public
Structure type:	stone walls, wooden floors, wooden roof
Construction period:	XV century
Changes of the structure:	Residential part on the north side added in the XVIII century
Period of changes:	XVIII
Data collection methods:	Visual inspection, architectural measurments, standards at the time of construction, iphotos, google maps, geoportal.dgu
Location:	c.p. *32; c.m. Kaštel Kambelovac

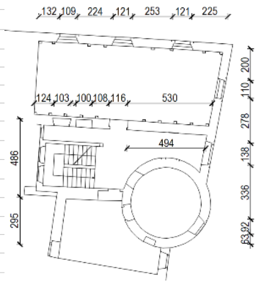
NO.	PARAMETER:			CLASS
1	TYPE AND ORGANISATION OF THE RESISTANT SYSTEM			D
	Built in accordance with HRN EN 1998-1	No		
	Retrofitted in accordance with HRN EN 1998-3	No		
	Confinement hor. tie beams on every floor	No		
	Confinement ver. tie beams on every floor	No		
	Walls connection quality:	Poor		
2	QUALITY OF THE RESISTANT SYSTEM			D
	Material:	Limestone		
	Element type:	Roughly shaped blocks		
	Homogenity:	Homogenous wall with two faces		
	Organization of masonry:	Poorly organized		
	Quality of mortar:	Poor		
3	CONVENTIONAL RESISTANCE			D
	Number of floors above the level of evaluation:	5		
	Total area A_{tot} :	152.3	m^2	
	Walls area in direction x A_x:	19.33	m^2	
	Walls area in direction y A_y:	7.35	m^2	
	Characteristic shear strength τ_k:	0.07	Mpa	
	Level of knowledge	KL1		
	Confidence factor FC:	1.35		
	Design shear strength	0.052	MPa	
	Mean story height **h**:	3.16	m	
	Masonry specific weight g_z:	22.0	kN/m^3	
	Floor self and imposed loads Q_{uk}:	3.24	kN/m^2	
	Min. value between A_x & A_y A =	7.4	(m^2)	
	Max. value between A_x & A_y B =	19.33	(m^2)	
	Coef. $a_0 = A/A_{Uk}$ =	0.05		
	Coef. $\gamma = B/A$ =	2.63		
	$q = (A + B)*h*g_z/A_{uk} + \Delta g$ =	15.42	kN/m^2	
	$C = a_0\tau_k/(qN)*[1+(qN)/1.5a_0\tau_k(1+\gamma)]^{1/2}$	0.08	g	
	$\alpha = C/0.38$	0.22		
4	POSITION OF THE BUILDING AND FOUNDATIONS			A
	Slope of the terrain: p =	5	%	
	Soil type:	Rock		
	Existance of foundations:	No		
	Max. diference in foundation elevation Δh:	0.50	m	
5	TYPOLOGY OF FLOORS			D
	Staggered floors:	No		
	Floor type:	Flexible system		
	Connection with walls:	Poor		
	Percentage of rigid and connected diaphragms:	10.0	%	

DRAWINGS AND NOTES

Ground floor plan:



First floor plan:



Load analysis - floors:

- final layers	0.15	(kN/m^2)
- planking	0.15	(kN/m^2)
- crushed stone	1.60	(kN/m^2)
- beams	0.25	(kN/m^2)
- ceiling	0.40	(kN/m^2)
G =	2.55	(kN/m^2)
Q =	2.30	(kN/m^2)
1,0G + 0,3Q =	3.24	(kN/m^2)

Weight for parameter 5:

W5 = 0.50*(100/a0) = 5.00

α_0 - percentage of rigid and connected diaphragms:

If W5 > 1, value of 1 is taken

In the case of concrete floors on weak walls; W5 = 1,25

6	PLANIMETRIC CONFIGURATION		D
Planimetric ratio:	β_1 = a/l =	41.7	%
Planimetric ratio:	β_2 = b/l =	40.0	%

7	ELEVATION CONFIGURATION		D
Percentage of mass variation:	$\Delta A/A_{uk}$ =	81.6	%
T/H ratio:	T/H =	65.4	%
Percentage of porch in total area:		0.0	%
Presence of porch in the first floor:		No	
Different materials on different floors:		No	

8	MAX. DISTANCE AMONG THE WALLS		A
Max. distance among the walls - **l**:		5.3	m
Wall thickness - **s**:		0.55	m
Ratio l/s =		9.64	

9	ROOF		D
Roof structure type:	With spreading		
Roof with horizontal ties or braces:		No	
Roof weight $\mathbf{Q_{uk}}$:		1.44	kN/m^2
Roof perimeter - **l**:		50.0	m
Roof support length $\mathbf{l_s}$:		44.0	m

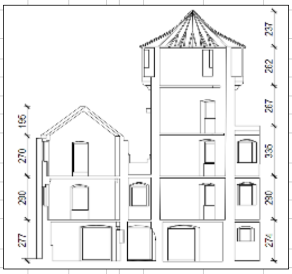
10	NON STRUCTURAL ELEMENTS		D
Type of appendices:	With appendices poorly connected		
Type of susp. ceilings:	No suspended ceilings		
Chimneys:	With no chimneys		
Balconies:	Poorly connected balconies		

11	STATE OF CONSERVATION		D
Evidence of damage:		Yes	
Damage origin:	Not of seismic origin		
Size of the cracks:		> 3	mm
Wall strenght:	Serious decay		
Vertical deviation:		Yes	

Weight for parameter 7: W7 = 0.50 for 1st floors with porches

W7 = 1.00 for all other cases

Section cut:



Load analysis - roof:		
- roofing tiles	0.45	(kN/m^2)
- battens	0.10	(kN/m^2)
- planking	0.15	(kN/m^2)
- beams	0.25	(kN/m^2)
- ceiling	0.40	(kN/m^2)
G =	1.35	(kN/m^2)
Q =	0.30	(kN/m^2)
1,0G + 0,3Q =	1.44	(kN/m^2)

Weight for parameter 9:

W9 = 0.50 + a1 + a2

α_1 = 0.00

α_2 = 0.00

α_1 = 0.25 for roof with concrete slabs or generally havier than 2.0 kN/m^2

α_1 = 0.00 for all other cases

α_2 = 0.25 if the ratio between the roof perimeter and support lenght ≥ 2,0

α_2 = 0.00 for all other cases

For the case of concrete roof on weak walls; W9 = 1.25; Moreover if the last floor is also concrete; W9 = 1.50

	PARAMETER	POINTS (P) A	B	C	D	WEIGHT (W)
1	TYPE AND ORGANISATION OF THE RESISTANT SYSTEM	0	5	20	45	1.50
2	QUALITY OF THE RESISTANT SYSTEM	0	5	25	45	0.25
3	CONVENTIONAL RESISTANCE	0	5	25	45	1.50
4	POSITION OF THE BUILDING AND FOUNDATIONS	0	5	25	45	0.75
5	TYPOLOGY OF FLOORS	0	5	15	45	1.00
6	PLANIMETRIC CONFIGURATION	0	5	25	45	0.50
7	ELEVATION CONFIGURATION	0	5	25	45	1.00
8	MAXIMUM DISTANCE AMONG THE WALLS	0	5	25	45	0.25
9	ROOF	0	15	25	45	0.50
10	NON STRUCTURAL ELEMENTS	0	0	25	45	0.25
11	STATE OF CONSERVATION	0	5	25	45	1.00

Vulnerability Index: $I_{v11} = \sum_{i=1}^{11} P_i W_i$ = 76.9

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