3.3.3 Determination of seismic vulnerability indexes for masonry historical buildings located in the HR test site

Final Version

Deliverable Number 3.3.3

Università degli Studi di Ferrara

SVEUČILIŠTE U SPLITU FAKULTET GRAĐEVINARSTVA, ARHITEKTURE I GEODEZIJE

COMUNE DI FERRARA
Città Patrimonio dell'Umanità

Project Acronym	PMO-GATE
Project ID Number	10046122
Project Title	Preventing, Managing and Overcoming natural-hazards risk to mitiGATE economic and social impact
Priority Axis	2: Safety and Resilience
Specific objective	2.2: Increase the safety of the Programme area from natural and man-made disaster
Work Package Number	3
Work Package Title	Assessment of single-Hazard exposure in coastal and urban areas
Activity Number	3
Activity Title	Assessment of climate-unrelated hazards exposure in urban and coastal areas (seismic action)
Partner in Charge	UNIVERSITY OF SPLIT, FACULTY OF CIVIL ENGINEERING, ARCHITECTURE AND GEODESY
Partners involved	UNIVERSITY OF SPLIT, FACULTY OF CIVIL ENGINEERING, ARCHITECTURE AND GEODESY PUBLIC INSTITUTION RERA SD FOR COORDINATION AND DEVELOPMENT OF SPLIT DALMATIA COUNTY NATIONAL INSTITUTE OF OCEANOGRAPHY AND EXPERIMENTAL GEOPHYSICS TRIESTE CITY OF KAŠTELA
Status	Final
Distribution	Public

Contents

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 Deliverable, the seismic vulnerability of the buildings in the Kaštel Kambelovac, a Croatian settlement located along the Adriatic coast, has been assessed 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. Developed methodology for seismic vulnerability assessment, shown in Deliverable 3.3.1, are used to calculate seismic vulnerability index, damage index, critical accelerations for the yield, significant damage and collapse states of the buildings, and finally index of seismic risk for three return periods for the buildings at the test site.

1 Introduction

One of the purpose of the PMO-GATE 3.3 Activity “Assessment of climate-unrelated hazards exposure in urban and coastal areas (seismic action)” is the determination of seismic vulnerability indexes for masonry historical buildings located in the HR test site, which was selected within the City of Kaštela.

Calculation of the seismic vulnerability indexes in this project is based on the methodology for vulnerability assessment on large scales, developed by the Italian National Research Council and the Italian National Group for the Defense Against Earthquake from 1984 onwards [1, 2]. It consists in filling in a survey form composed of 11 parameters, calculations of those parameters and finally, calculation of the vulnerability index for the building. In this project an update from the Tuscany Region [3] has been used because of the modification of some qualitative and quantitative aspects.

The application of the methodology and calculation of vulnerability indexes, demands information about the geometry and material characteristics of a buildings. Therefore, the main characteristics of the buildings in the Kaštela City were analysed, considering that the building codes in different construction periods have influenced to the building characteristics, used construction materials and their mechanical characteristics, as well as level of the details. Furthermore, the classification of the buildings according to their construction period, considering 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.

In this deliverable, after the description of the HR test site, the structural and material characteristics of the buildings are discussed as well as a methodology for identification and collection of information about the buildings. Subsequently, a short description of the vulnerability index method, used for seismic vulnerability assessment of the buildings in the HR test site, is given. Vulnerability indexes for the buildings are presented. Complete vulnerability index form with building details and necessary calculations is shown. Short description of the soil seismic characteristics and expectations from geophysical survey are presented. An application of static non-linear analysis in determining of seismic capacity of the buildings is demonstrated. Finally, the results of non-linear static analysis are combined with vulnerability indexes and used to calculate seismic risk in terms of damage and index of seismic risk for the buildings at the test site.

2 Description of the HR test site

The City of Kaštela is formed from seven settlements developed from the 15th century until today (Fig. 1).

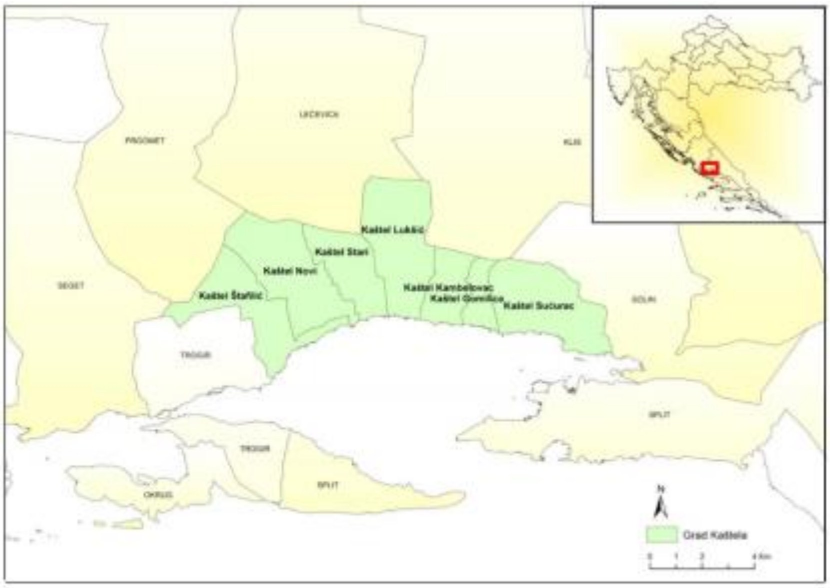


Fig. 1. Settlements and geographic position of Kaštela City [4]

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 (Fig. 2).

Fig. 2. Protected historical centres of Kaštela City

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. Since the area of an 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. 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 can cover characteristics of building history of whole Kaštela area from the 15th century onwards. After considering all the arguments in choosing the area to be observed and also considering possibilities of extrapolation of results, Kaštel Kambelovac was chosen as a test site.

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", i.e. the Road of Dr. Franjo Tuđman. The observed area covers around 45000 square meters and includes more than 400 objects. The benefit of the chosen area is diversity of objects considering construction, architecture and material, built from the 15th century until today. According to [5], 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. Within the observed area, a historical centre founded in the 16th century around the tower of Cambi, as well as the church of St. Mihovil from the 19th century with a bell tower from 1860, are placed. Area is the mixture of private and public facilities, mostly built as masonry and concrete buildings. Plan view of the selected area is shown in Fig. 3, 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.

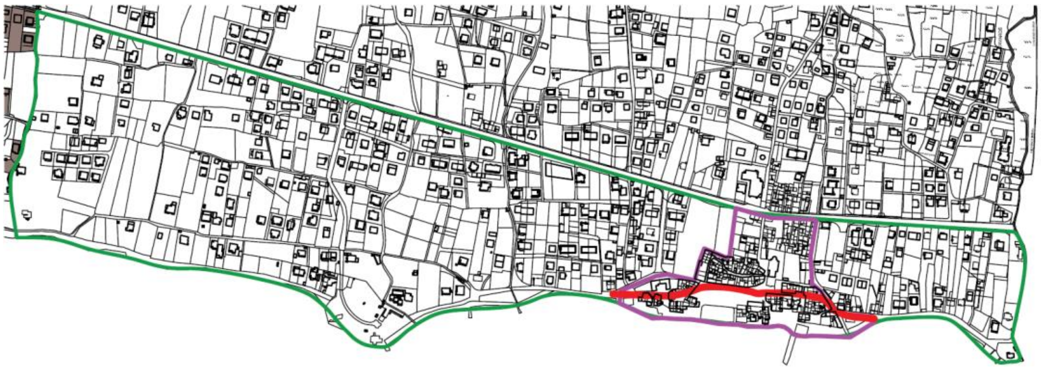


Fig. 3. Plan view of the selected area (green line) with the mark of the natural coastline (red line) and the historical centre (purple line)

Within the historical centre (Fig. 3 - 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 [5] the line of the existing coast was changing throughout the history. Original coastline (Fig. 3 - 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 (Figs. 4 and 5).

Fig. 4. The view on a historical centre, Kaštel Kambelovac

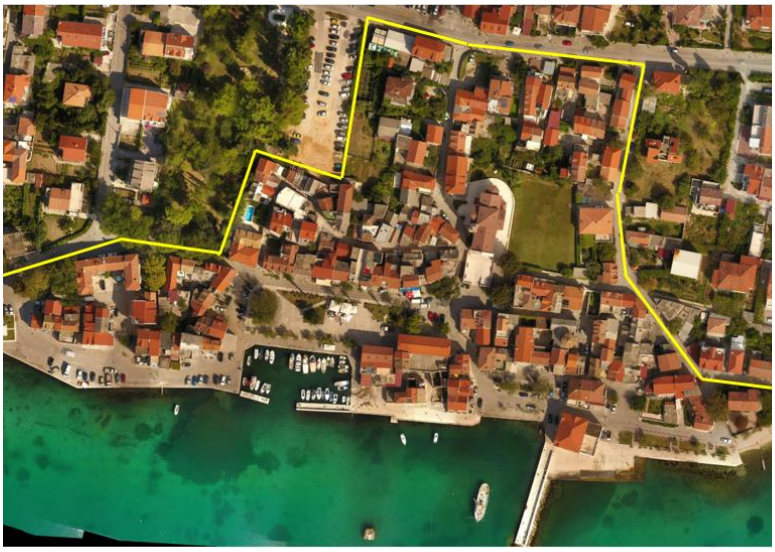


Fig. 5. Plan of a historical centre of Kaštel Kambelovac

3 Structural and material characteristics of the buildings

The 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.

Structural and material characteristics of the buildings in the Kaštel Kambelovac depend on construction period. They are presented in Deliverable 3.3.1. Guidelines of the assessment procedure for earthquake vulnerability in HR test site [6]. The most basic features are listed here.

The buildings in the historical centre (Fig. 6 and 7), built between the 15th and 19th centuries, are mostly made of stone blocks with mortar joints. The walls have a thickness between 45 and 75 cm. Namely, 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 per 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. Original floors were made of wood. Recently, some of those buildings were reconstructed and wooden floors were often replaced with monolith reinforced concrete plates. Some original stone buildings facade covered with plaster (Fig. 6).

The buildings built at the end of 19th and beginning of the 20th century are mostly made of stone or brick-mortar walls, with wooden floor structure (beams + floor covering). Since 1925, the reinforced concrete (RC) ceilings with very low thickness, minimal protective layers and segregation problems (low concrete quality) were used. In later periods prefabricated ceilings were used. The buildings were constructed without concrete boundary elements (beam and columns). Generally, these buildings were not designed to cope with strong horizontal motions (earthquake).

Fig. 6. Historical urban area of Kaštel Kambelovac, view on Drago Britvić Street

Fig. 7. Historical urban area of Kaštel Kambelovac, Pučki kaštel and Dudan’s palace

The most important historical buildings are Tower and Mansion Cambi, St. Mihovil and Martin Church and Former Ballet School (Music School today) which was recently reconstructed (Fig. 8).

(a)

(b)

(c)

Fig. 8. The main historical buildings in Kaštel Kambelovac: (a) Tower and Mansion Cambi, (b) St. Mihovil and Martin Church, (c) Former Ballet School

The main public buildings in historical centre, adjacent to the aforementioned Ballet School, are City Library, Kindergarten and Rowing Club (Fig. 9).

(a)

(b)

(c)

Fig. 9. The main public buildings in Kaštel Kambelovac: (a) City Library, (b) Kindergarten, (c) Rowing Club

Some buildings are in very poor condition, quite damaged, with joints between the stone blocks from which the mortar fell (Fig. 10), damaged walls, deformed, and in some cases destroyed roofs (Fig. 11).

Fig. 10. Damaged buildings in the historical centre

In addition, there are several completely demolished buildings (Fig. 11). Also, some residential buildings have significant cracks in the load-bearing walls so their safety is low (Fig. 12 and 13).

Fig. 11. Damaged walls and destroyed roofs of the buildings

Fig. 12. Uninhabited building in the centre with significant cracks

Fig. 13. Building with significant cracks

Given the very poor condition of the buildings in the centre, in the event of an earthquake, some of those buildings could be danger to the safety of occupants, neighbours or passersby.

Outside of the historical centre, the buildings were mostly made as masonry structures consist of stone, concrete or clay blocks without or with concrete boundary elements (beam or/and columns) depending on the construction period.

From 1945 to 1964, monolithic reinforced concrete floors were predominantly used. After the 1964 seismic regulation demands that all buildings have horizontal reinforced concrete beams and rigid horizontal diaphragms or horizontal reinforced concrete beams, reinforced concrete columns and rigid horizontal diaphragms depending on the seismic zone and number of the floors. For the two-floor buildings in the seismic area up to VIII degree (in Kaštela), RC vertices (columns) were not required. Typical buildings built in 1960s, 1970s and 1980s are shown in Fig. 14.

After 1980, stricter regulations for construction in earthquake areas were applied. Therefore, the use of unbounded masonry was not allowed in area of medium and high seismicity. In Kaštela area it is allowed to build two-storey masonry construction without vertical RC columns and three-storey for structures with vertical RC columns.

Buildings built from 2005 onwards are seismically resistant structures due to the applications of modern design standards based on European regulations (Eurocode 8), firstly implemented through the prestandards (ENV) and finally by introducing the full European standard (Eurocode 8) in 2011 into the Croatian national legislation (HRN EN 1998-1:2011 [7]).

The general problem with assessing earthquake vulnerability of the buildings, regardless of the construction period, is that many private residential buildings were built illegally, so it is not possible to determine whether their construction has been complied with applicable regulations.

Fig. 14. Typical residential buildings outside the historical centre built in period 1960-1980.

4 Data collection and identification of the building characteristics

Presentation of the spatial distribution of the critical zones most prone to seismic risk in HR test site, but also to sea flood and extreme sea waves demands:

- geodetic basis of the area
- identification of the geometrical, structural and material properties of the buildings

4.1 Geodetic basis of the area

The detailed geodetic survey of terrain was performed using geodetic drone recording, and then high-resolution geodetic basis of the HR test site was created in order to define cadastral and building parcels and topographic characteristics of the area. This basis is high quality foundation used to show spatial distribution of the vulnerability of the buildings and to develop seismic vulnerability map. In addition, it will be used to present other risks that will be analysed in the project (sea flood and extreme sea waves), as well as single-hazard and multi-hazard exposure of the area. The geodetic basis of the HR test site and the cadastral and building parcels are shown in Figs. 15 and 16.

Fig. 15. Geodetic basis of the HR test site

Fig. 16. HR test site with marked cadastral and building parcels

4.2 Collection of the geometrical, material and structural data of the buildings

The seismic assessment of buildings in the test area requires the knowledge of their geometrical, material and structural characteristics as well as soil type. The methodology for data collection was organized as follows:

- Investigation of the buildings using historical documentation [40] and archival documentation of the city of Kaštela;
- Detailed survey of geometrical characteristics, architectural measurements, and creation of architectural drawings (floor plans and cross sections);
- Identification of structural systems and materials through visual inspection, using archive documentation, literature, and thermographic imaging in the several specific cases where, due to non-documented reconstructions, it was not possible to recognize the material and structural characteristics of the buildings;
- Characterization of the soil type by means of a geophysical survey.

Additional assistance came from high-resolution geodetic maps of the test site with precise plan dimensions, from Google Maps with Street View options, and also a map of the area made in 1968 that allowed the identification of reconstructions.

Investigations of archival documentation and visual inspection were used to detect the main structural features crucial to the seismic vulnerability assessment, such as the type and configuration of the structural system, the texture and quality of masonry walls based on the distribution of blocks and mortar joints, as well as their thickness, the mortar quality, the type of floors, and the floor-wall connections. Furthermore, other important aspects that were investigated were the resistance along two main horizontal directions based on estimates of the maximum resistant shear of the structure, the position and foundations of the building, its horizontal and vertical configurations, the maximum distance among the walls, the typology and weight of the roof, the presence of non-structural elements, and the state of conservation. The mechanical properties of the materials (stone walls, mortar) were taken from the literature [41, 42]. A valid seismic regulation in the past was also used to identify the material properties in the case of reconstructions.

The HR test site was divided firstly into two parts: historical core and the area outside of the historical core. The buildings in the historical core are mostly stone masonry, built to the end of 19th century and at the beginning of 20th century. Outside of the historical core the buildings are mostly built after the 1948 and can be classified according to the construction period. Furthermore, the part outside the historic centre is divided into northern, eastern and western parts (see Fig. 17).

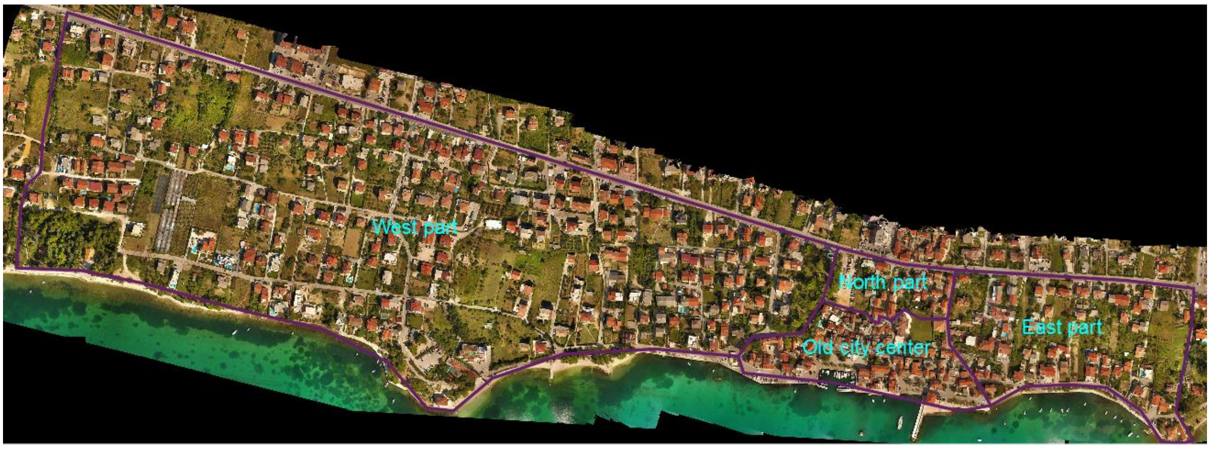


Fig. 17. Test site Kaštel Kambelovac divided into four characteristic parts

The methodology for collecting data on buildings is different for each of these parts.

In the historical centre no technical documentation was found for most of the buildings, an architectural survey, collection of photo documentation and visual inspection of material and construction characteristics were performed by field data collection. Several buildings in the historic core where material and structural features are not evident from visual inspection were examined by thermographic imaging. The results of the field investigation of the building characteristics are presented by drawn floor plans and cross sections of the buildings, photo documentation and description of the structural and material characteristics. These data are used in defining individual building vulnerabilities. As the special emphasis in the project is on researching the seismic vulnerability of old stone buildings, all the buildings in the historic centre were analysed (Fig. 18). Processing on a large sample enables later definition of characteristic typologies with conclusions about their vulnerability indexes and application to other old town cores in the Dalmatian coast.

Fig. 18. Old city centre divided into blocks

Identification of the properties of the buildings outside of the historical core was performed as previously mentioned, using existing documentation from the archives of the Kaštela city, high-resolution geodetic map, Google Map with Street View options, a map of the area from 1968 and visual inspection of the buildings. Detail survey and thermographic imaging of the buildings outside of the historical core was made only for few important buildings (kindergartens, former ballet school).

Geometrical, material and structural characteristics of the buildings were analysed and the parameters, which influence to the seismic vulnerability, were calculated. Finally, seismic vulnerability indexes were calculated using seismic vulnerability index form.

4.3 Thermographic examination of the buildings in the historical core

Few buildings in historical core and outside of the core have external plaster which covers the walls. Some of them were reconstructed and new parts were added. An original wall thickness of 50 to 60 cm suggests that these are stone masonry buildings or buildings made of unreinforced concrete that began to be used in the early 20th century. Furthermore, the material of the subsequently added parts is unknown. The material and structural properties were detected by thermographic examination. Rowing club, former ballet school, kindergartens and few residential buildings in historical core are example of those buildings.

From the thermal images of the facade and details of photos, lot of characteristics and details were detected such us material and way of construction of the walls, existence of horizontal and vertical confinement, material of floor and roof structures, heterogeneities of the material, etc.

For example, thermographic examination of Rowing club building has shown that the ground was made of unreinforced concrete, while the first floor was built of stone and clay blocks. Vertical confining elements of the first floor made of concrete with few steel reinforcements were detected at the corners of the building. Thermographic images also showed the existence of rigid concrete floor instead of flexible wooden floor. This conclusion was based on the different thermal conductivity of concrete and wooden ceilings. Additionally, detection of the cracks in junction zones between partly reinforced confining concrete and stone/brick masonry wall testifies to the degree of damage and preservation of the building. These conclusions significantly contributed to the process of the detection of the material and structural characteristics and calculation of the vulnerability indexes.

Figs. 19-24 show an examples of the examined buildings with their thermographic images.

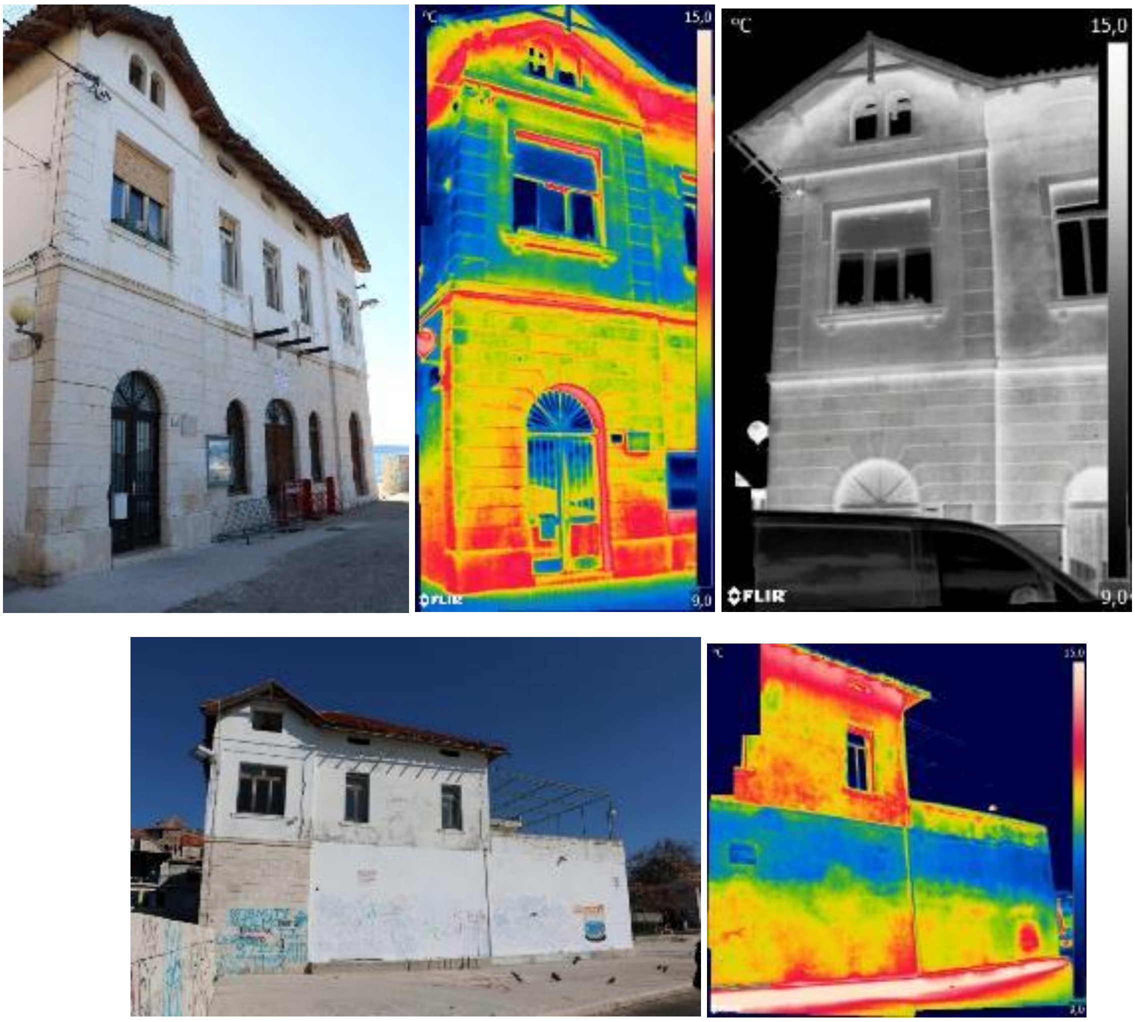


Fig. 19. Rowing club: the ground made of unreinforced concrete, the first floor was built of stone and clay blocks, existence of rigid concrete floors.

Fig. 20. Residential building Cambijev trg 11: Unreinforced stone masonry building, the ground is made in different period from the first and the second floor, significant cracks of gable wall.

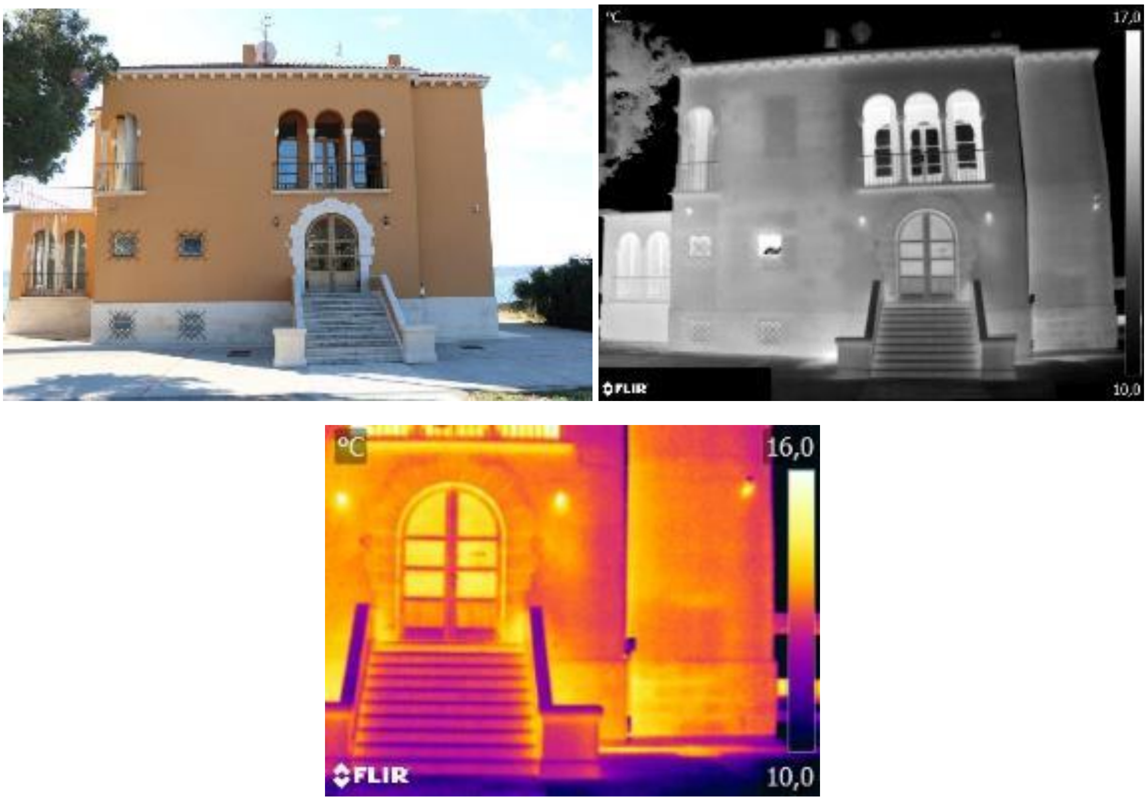


Fig. 21. Former Ballet school: The building was reconstructed several times; horizontal lines are characteristics of the unreinforced concrete poured into wooden framework.

Fig. 22. Residential building Drage Britvića 13: Stone masonry walls with RC floors, balcony and roof slabs is characteristics of reconstructed old stone masonry building.

Fig. 23. Kindergarten Franje Tuđmana bb: Structure is brick masonry wall with vertical corner RC confinements, horizontal confinement and RC roof slab covered by roof tiles.

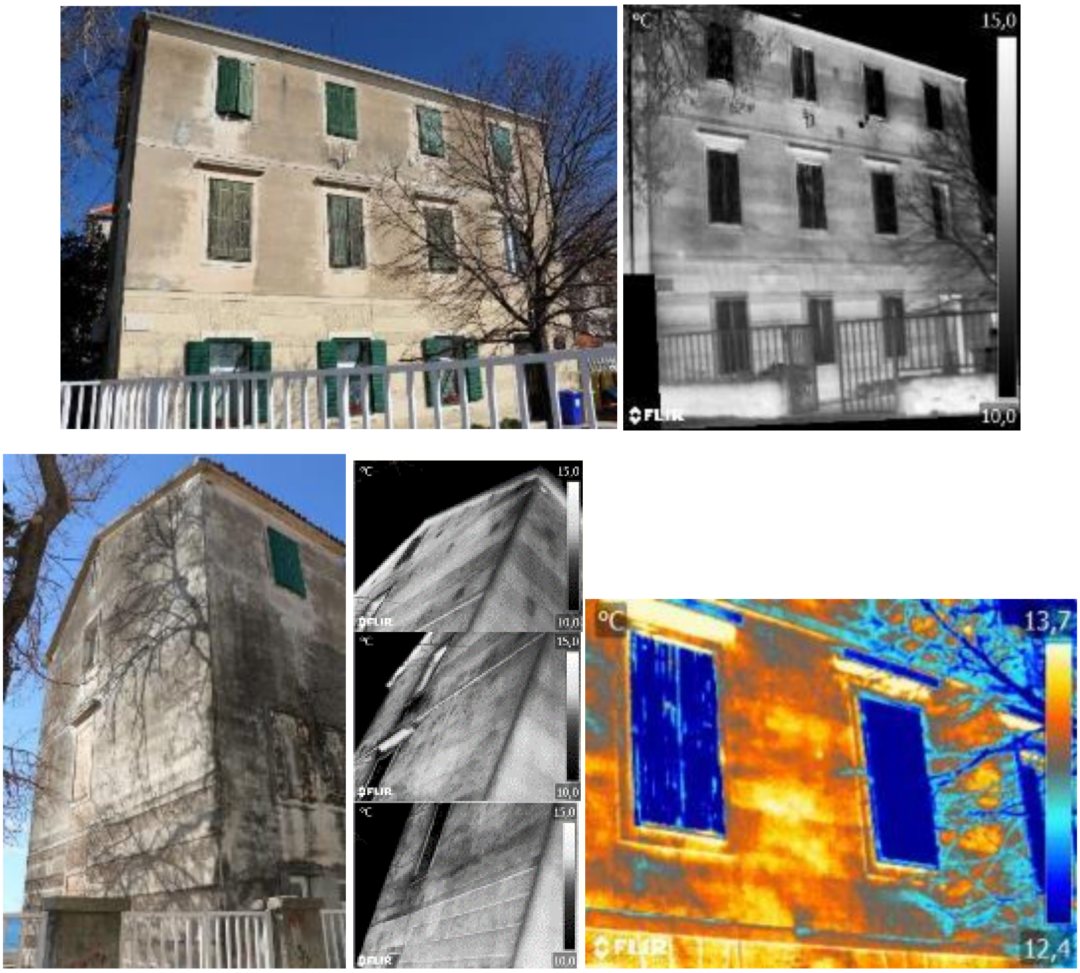


Fig. 24. Kindergarten Don Klementa Tadina 2: Horizontal lines are clearly depicting layer of concrete made in technique of poring 40 to 50 cm of concrete into wooden framework while differences in temperature conduction is caused by segregated concrete with poor mechanical properties.

5 Seismic vulnerability assessment of the buildings in the HR test site

Vulnerability index method is used for seismic vulnerability assessment of the buildings in the HR test site. The vulnerability index in PMO-GATE project is based on original Vulnerability index method for masonry structures developed by the Italian National Research Council and the Italian National Group for the Defense Against Earthquake from 1984 onwards [1, 2]. The method is upgraded with the modifications, proposed by Tuscany Region in 2003 [3], caused by substitution of light wooden floors with heavier ones made of reinforced concrete slabs.

The method consists in filling in a survey form composed of 11 geometrical and structural vulnerability parameters, calculations of those parameters and finally, calculation of the vulnerability index for the building. The main parameters of vulnerability index method applied in PMO-GATE project, consider type and organization of the resistant system, quality of resistant system, conventional resistance in the two main horizontal direction of the building based on estimation of the maximum resistant shear of the structure, position of the building and foundations, typology of floors, planimetric and elevation configuration, maximum distance among the walls, typology and weight of the roof, the presence of non-structural elements and state of conservation. For each parameter, the surveyor has to give a judgment (four possibilities, from "A" - optimal condition to "D" - unfavourable condition). For each judgment, a numerical score value is given by the method. Using the weight coefficients related to each parameter (provided in order to take into account the relative importance of each parameter in the global definition of vulnerability), a Vulnerability Index I_v is calculated and normalized in a 0-100% range. A low index means that the structure is not so vulnerable and has a high capacity under seismic action while the high index shows that structure is vulnerable and has a low seismic capacity.

Also regarding the weights of the structural elements and the imposed loads new reference summary tables have been introduced, with broad typological cases. A substantial change, which however does not enter directly into the procedure for completing the form, was that of the variation of the weights in the classes of parameters 1, 5 and 9, weights that influence the relative partial index, necessary for the calculation of the vulnerability index. In particular:

- in parameter 1 the weight changes from 1 to 1.5;

- in parameter 5 the weight (variable) changes from a range 0.5 - 1 to arrange 0.5 - 1.25;

- in parameter 9 the weight (variable) changes from a range 0.5 - 1 to arrange 0.5 - 1.5;

Finally, in this project the current Croatian standards for the design of masonry structures HRN EN 1996-1 [8] and design of structures for earthquake resistance HRN EN 1998-1 [7], is adopted in definition of the classes for each parameter. The adopted methodology is presented in Deliverable 3.3.1. Guidelines of the assessment procedure for earthquake vulnerability in HR test site [9].

The vulnerability parameters used in this project, their numerical score values and the weight coefficients are shown in Table 1. The maximum value of Vulnerability Index I_v is 438.75.

Table 1 Vulnerability parameters and their weights (PMO-GATE)

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

Based on the presented methodology, Vulnerability index form in Excel was developed and used for calculation of vulnerability indexes for the masonry historical buildings in HR test site.

The methodology demands information about the geometrical, structural and material characteristics of the buildings. Those characteristics were identified from the existing technical documentation, field investigations of the buildings, architectural measures, drawing the floor plans and cross sections of the buildings and photo documentation.

On the basis of analysis of architectural, structural and material features of the buildings, filling of the vulnerability index form and calculation of 11 geometrical and structural vulnerability parameters, were performed. Finally, the seismic vulnerability indexes for buildings in HR test area Kaštel Kambelovac), were calculated.

Vulnerability index form for example of Cambi Tower is shown in Table 2.

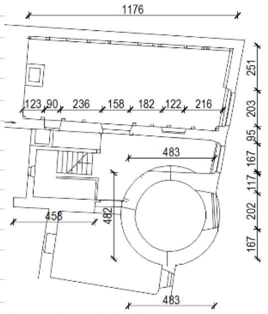
Table 2 Vulnerability index form (PMO-GATE), example of Cambi Tower

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

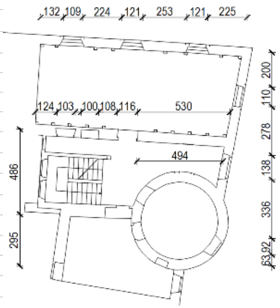
NO.	PARAMETER:			CLASS
1	TYPE AND ORGANISATION OF THE RESISTANT SYSTEM			D
	Built in accordance with HRN EN 1998-1	No		
	Retrofitted in accordance with HRN EN 1998-3	No		
	Confinement hor. tie beams on every floor	No		
	Confinement ver. tie beams on every floor	No		
	Walls connection quality:	Poor		
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. γ = 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	
	α = 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:	$\beta_1 = a/l =$	41.7	%
Planimetric ratio:	$\beta_2 = b/l =$	40.0	%

Weight for parameter 7: W7 = 0.50 for 1st floors with porches; W7 = 1.00 for all other cases

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	

Section cut:

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 $\mathbf{l_a}$:		44.0	m

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

$\alpha_1 =$ 0.00

$\alpha_2 =$ 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

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	

	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

5.1 Vulnerability indexes of the buildings in the historical core

The vulnerability indexes for 75 buildings in the historical core were calculated and all collected data were digitalized. Some of these buildings are located in several cadastral parcels and have different owners, but they were put in the same class because of their structural integrity. To improve the interpretation of the results, these individual vulnerability parameters, as well as other important input information, were integrated into a geographical information system (GIS) tool. The GIS software adopted in this study was the open-source suite ESRI ArcGIS Runtime 100.4 v1, in which geo-referenced graphical data (vectorised information and orthophoto maps) were combined with building parameter information.

Vulnerability indexes with the photos, position of the buildings in the settlement and construction period are shown in Table 1 of APPENDIX1.

The buildings were divided into 4 vulnerability classes: low vulnerability for $I_v < 30$, medium-low vulnerability for $30 < I_v < 45$, medium-high vulnerability for $45 < I_v < 60$, and high vulnerability for $I_v > 60$. Fig. 25 shows the vulnerability map of the area, whereas the distribution of the vulnerability is shown in Fig. 26. Most buildings in the historical core belonged to the high vulnerability class (25%) and to the medium-high vulnerability class (47%). A small number of buildings were classified as medium-low vulnerability (21%). Only a few buildings were of low vulnerability (7%), and these were old stone buildings that were reconstructed. Buildings with vulnerability index of 45 and larger were considered highly vulnerable, as expected, given the age of the town centre. The indexes ranged from 11.1, corresponding to one of the newer houses that was completely renovated at the boundary of the core, to 76.9, the vulnerability index of the Cambi tower. Houses made of poorly connected walls, with flexible floor structures, irregular in layout and height, were revealed to be more endangered. In addition to these basic aspects, the degree of general preservation of the building and the presence of subsequent reconstructions significantly affected the vulnerability indexes.

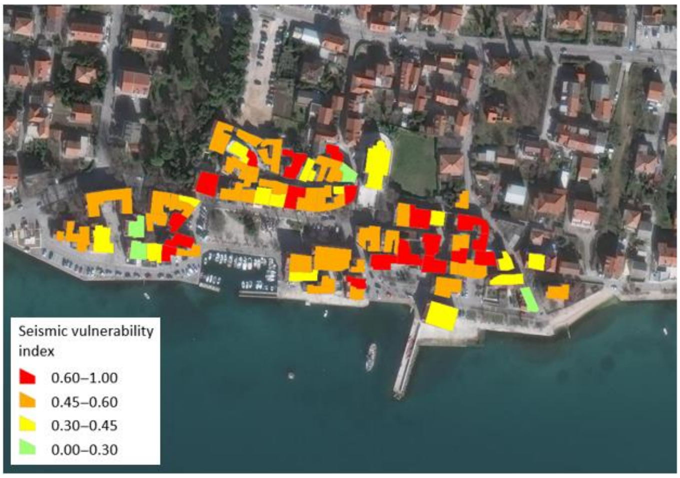


Fig. 25. Vulnerability map of the historical core

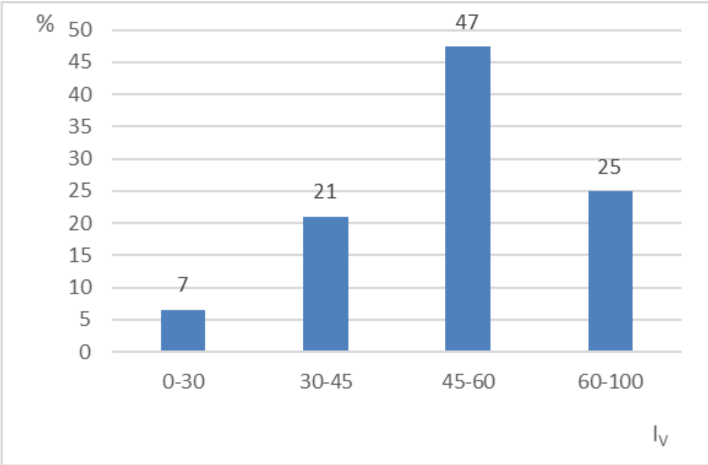


Fig. 26. Vulnerability index distribution in the historical core

The investigation of the parameters necessary for evaluating the seismic vulnerability index allows us to characterize and draw some indicators that can help to better understand the vulnerability results. The spatial distribution of these parameters was analysed and the results are shown in Fig. 27.

We firstly focused on parameter “type and organization of the resistant system”, which measures the presence of connections among perpendicular walls and connections among floors or roofs in masonry buildings, which are necessary to ensure the three-dimensional box behaviour of the building. We detected that about 77% of the buildings were made without any confining elements and with poorly connected stone walls (class D), whereas 21% were also made without confining elements but had strongly connected walls (class C). Only 1% were in class A, having been reconstructed. Most buildings were made without confining elements (classes C and D), this being one of the main aspects that can lead to significant damage and to the separation of the walls.

The “quality of the resistant system” parameter is based on the type of masonry, considering the type of material, the shape of the elements, and the homogeneity of the walls. Fig. 27b shows the distribution of the buildings between vulnerability classes A to D, with 3%, 3%, 57%, and 37% of buildings, respectively. The majority of them belonged to classes C and D, indicating medium to high vulnerability.

The “conventional resistance” parameter estimates the maximum shear strength of the structure, accounting for the resistant area of the walls in the two main horizontal directions. As shown in Fig. 27c, the buildings were distributed between vulnerability classes A to D, with 1%, 16%, 68%, and 15% of the buildings, respectively.

The “position of the building and foundation” parameter considers the influence of the local morphology of the site and the natural slope of the ground. Its distribution in Fig. 27d showed a low level of vulnerability. Specifically, 57% of buildings were in class A and 43% were in class B. The reason for such a distribution is the presence of solid soil of type A and the relatively small slope of the terrain.

The “typology of floors” parameter evaluates the in-plane stiffness of the floor and the presence of efficient floor-to-wall connections. The buildings were distributed between vulnerability classes A to D, with 19%, 0%, 3% and 78% of buildings, respectively (Fig. 27e). The reason for this high percentage of buildings in class D is the presence of wooden floors which were poorly connected to the walls.

Regarding the parameter of “planimetric configuration”, which measures the regularity of the planimetric shape of the building, 8%, 17%, 23%, and 52% of the buildings were distributed in vulnerability classes A to D, respectively (Fig. 27f). The most populated classes were C and D, because a high level of horizontal irregularity was detected.

The “elevation configuration” parameter evaluates vertical regularity through the analysis of the stiffness of different floors, the presence of porticos, lodges, towers, and other structural elements which affect the distribution of the masses at each floor. In terms of this parameter, 19%, 11%, 41%, and 29% of the buildings belonged to vulnerability classes A to D, respectively, as displayed in Fig. 27g, with classes C and D proving to be most relevant.

The “maximum distance among the walls” parameter validates the presence of structural walls orthogonally connected to transversal ones. The buildings in the historical core had a favourable distribution, as most of them belonged to classes A and B, with 69% in class A, 12% in class B, 13% in class C, and 5% in class D, as shown in Fig. 9h.

In terms of the parameter “roof”, which evaluates the roof’s typology and weight, buildings were distributed among vulnerability classes A to D, with 9%, 40%, 15%, and 36% of the buildings, respectively (Fig. 27i).

The presence of “non-structural elements”, which can cause damages due to falling, highlighted an area of high vulnerability because most buildings had weakly connected non-structural elements and belonged to classes C and D. The distribution shown in Fig. 27j was 12%, 0%, 53%, and 35% for classes A to D, respectively.

The “state of conservation” parameter analyses the condition of the building and the presence of cracks in structural walls. The relevant distribution, shown in Fig. 27k, was as follows: 27% were in class A, 0% in B, 45% in C, and 28% were in class D. Most old stone masonry buildings, which have not yet been reconstructed, are in a bad condition and belong to classes C and D, whereas reconstructed buildings are in class A.

(**a**)

(**b**)

(c)

(d)

(e)

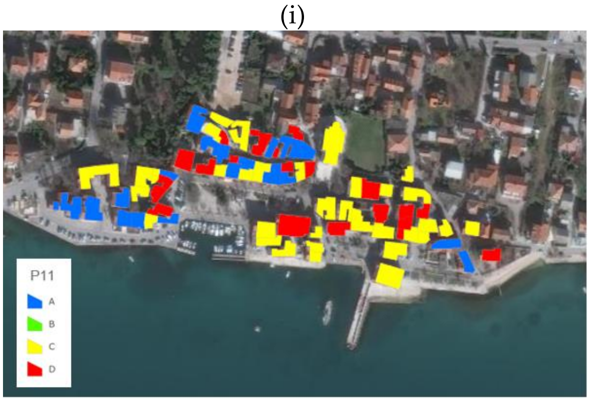
(f)

(g)

(h)

(i)

(j)



(k)

Fig. 27. Spatial distribution of the 11 parameters that comprise the seismic vulnerability index: (a) type and organization of the resistant system; (b) quality of the resistant system; (c) conventional resistance; (d) position of the building and foundation; (e) typology of floors; (f) planimetric configuration; (g) elevation configuration; (h) maximum distance among walls; (i) roof; (j) non-structural elements; (k) state of conservation.

If we consider the ratings in the analysis of the vulnerability of the Cambi tower, it can be determined exactly how much each parameter contributed to the total vulnerability.

Table 3. Contribution of each parameter in the vulnerability index, example of Cambi tower

Parameter	Score (s_{vi})				Weight (w_i)	Product	Normalized	Participation %
	A	B	C	D				
Type and organization of the resistant system (P1)	0	5	20	45	1,50	67,50	15,4	20,0
Quality of the resistant system (P2)	0	5	25	45	0,25	11,25	2,6	3,3
Conventional resistance (P3)	0	5	25	45	1,50	67,50	15,4	20,0
Position of the building and foundation (P4)	0	5	25	45	0,75	0,00	0,0	0,0
Typology of floors (P5)	0	5	15	45	1,00	45,00	10,3	13,3
Planimetric configuration (P6)	0	5	25	45	0,50	22,50	5,1	6,7
Elevation configuration (P7)	0	5	25	45	1,00	45,00	10,3	13,3
Maximum distance among the walls (P8)	0	5	25	45	0,25	0,00	0,0	0,0
Roof (P9)	0	15	25	45	0,50	22,50	5,1	6,7
Non-structural elements (P10)	0	0	25	45	0,25	11,25	2,6	3,3
State of conservation (P11)	0	5	25	45	1,00	45,00	10,3	13,3
					Σ	337,50	76,9	100

It can be seen that with the largest share in the sum, parameters 1 and 3 participate with 20%, followed by 5, 7 and 11 with 13.35% and then 6 and 9 with 6.7%. It is shown graphically in the diagram (Fig. 28).

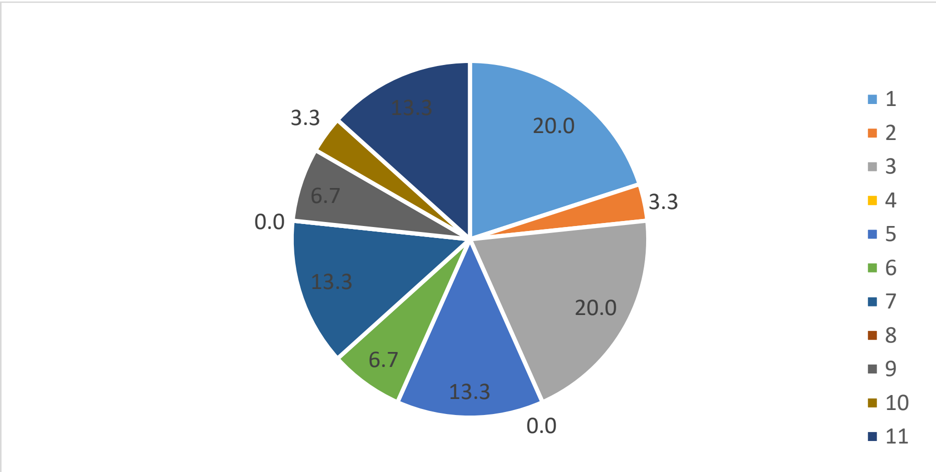


Fig. 28. Influence of each parameters of vulnerability index method in vulnerability index of Cambi tower

5.2 Vulnerability indexes of the buildings outside of historical core

Vulnerability indexes of the buildings outside of the historical core with photos and position of the buildings in the settlement are shown in Table 2 of APPENDIX1 (northern part), Table 3 of APPENDIX1 (eastern part) and Table 4 of APPENDIX1 (western part). For the buildings without technical documentation, vulnerability indexes are estimated based on the buildings with similar geometrical, material and structural characteristics and include directly in seismic vulnerability map.

Fig. 29 shows, in addition to the distribution of building vulnerability indexes in the historic core, and indexes in the northern, eastern and partially in the western parts of the test site.

In the northern part, which does not belong to the protected historic core, there are a number of stone masonry buildings. Therefore, most of the buildings belong to High, Medium-High and Medium-Low vulnerability classes. Only two buildings have Low vulnerability class.

In the eastern and western parts of the test site, the buildings are built with concrete or clay blocks, without confinement, only with confinement horizontal tie beams or with horizontal and vertical confinement. They mostly belong to the low vulnerability class (I_v<30%).

Fig. 29. Vulnerability of the buildings in the whole test site divided into 4 classes

Fig. 30 shows detail classification of the buildings according their vulnerability indexes with division into classes of 10%.

Fig. 30. Vulnerability of the buildings - 10% division intervals

Figs. 29 and 30 clearly show the difference between the vulnerability of the historical core and the area north of the core, where mostly stone masonry buildings were built from the 15th to the beginning of the 20th century, from the rest of the area where buildings are built of concrete and clay blocks, mostly after 1950 until today.

6 Seismic characteristics of the terrains in the HR test site

6.1 Seismic hazard in Croatia

Activity 3.3 of the PMO-GATE project "Assessment of climate-unrelated hazard exposure in urban and coastal area" aims to develop single hazard exposure map for earthquake for the selected HR test site. Namely, seismic risk is a result of an interaction between seismic hazard and vulnerability.

Seismic hazard level for Croatia, have been presented with two maps, expressed in terms of the peak horizontal ground acceleration during an earthquake, which is exceeded on average once in 95 or 475 years. The maps have been accepted as a part of the National Annex in HRN EN 1998-1:2011 [7]. In the map, which is used in designing earthquake resistant buildings, the reference peak ground acceleration on type A for the return period of 475 years with a probability of exceedance of 10% in 50 years is shown (Fig. 31). According to HRN EN 1998-1:2011 [7], soil type A is defined as ground where the velocity of propagation of seismic waves exceeds v > 800 m/s is composed of rock or other rock-like geological formations, including at most 5 meters of weaker material at the surface. This map is used for determination of seismic risk in Croatia. Another map, for the return period of 95 years with a probability of exceedance of 10% in 10 years, shown in Fig. 32 is used in order to satisfy the fundamental requirements in damage limitation states. In Croatia, the type 1 response spectrum for an earthquake magnitude higher than 5.5 was adopted.

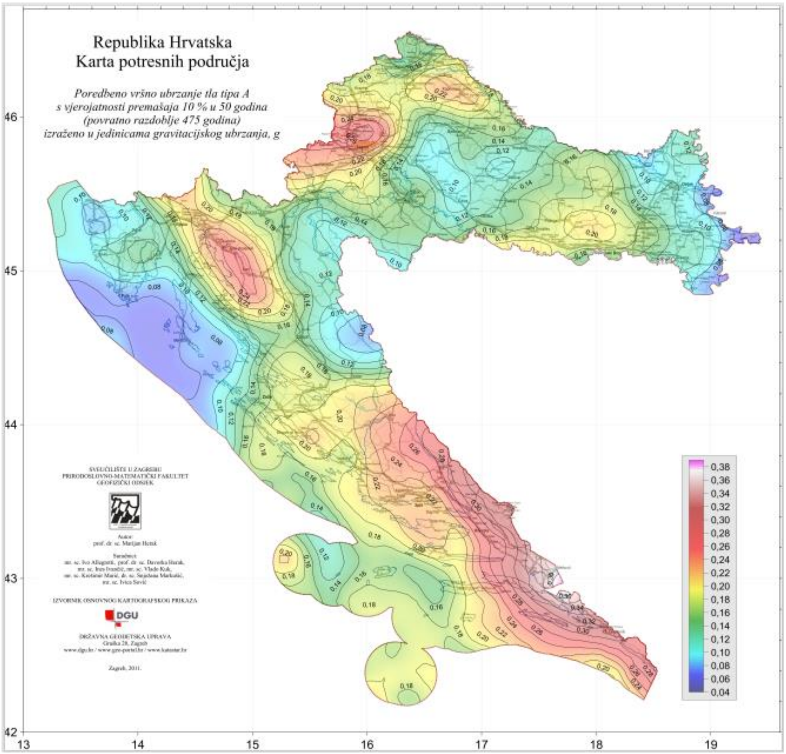


Fig. 31. Seismic hazard maps for Republic of Croatia (PGA) for return period of 475 years [7]

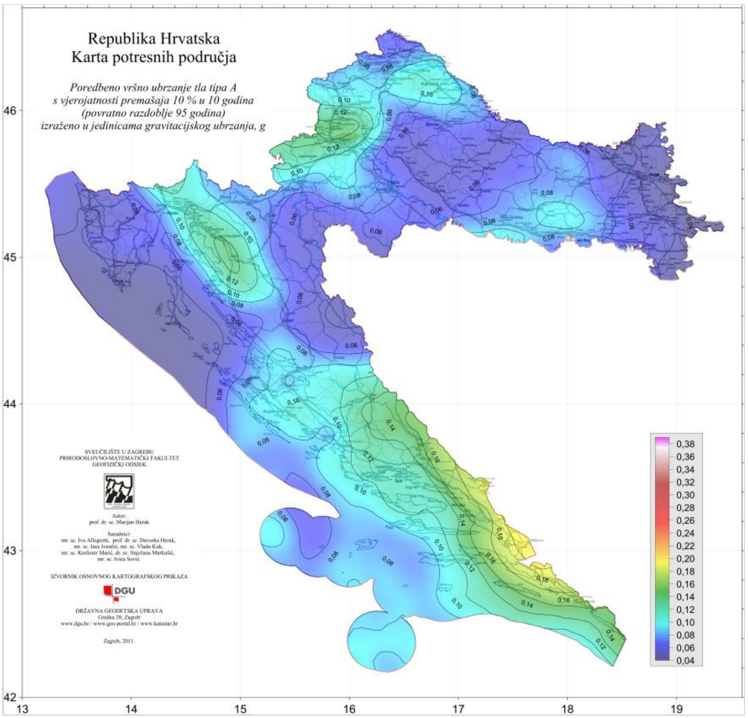


Fig. 32. Seismic hazard maps for Republic of Croatia (PGA) for return period of 95 years [7]

Recently, new seismic hazard map for return period T=225 years has been developed (Fig. 33).

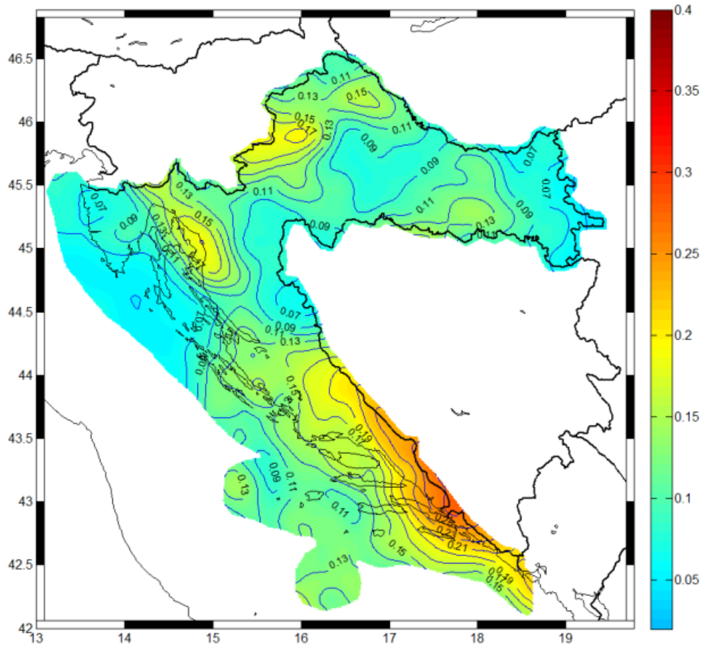


Fig. 33. Seismic hazard maps for Republic of Croatia (PGA) for return period of 225 years

In the Split and Kaštela area, the seismic hazard, measured via the peak ground acceleration for the soil type A, is equal to a_g = 0,22 g and a_g = 0,11 g for the return periods of 475 and 95 years, respectively.

According to EN 1998-1:2011 [10] and HRN EN 1998-1:2011 [7], the soil factor S, for ground types different from A, increases the ordinate of the elastic response spectrum, see Fig. 34 and Table 4. The real hazard for a certain location can be obtained by combining the peak ground acceleration for ground type A with the soil factor S, describing the influence of local ground conditions on the seismic action.

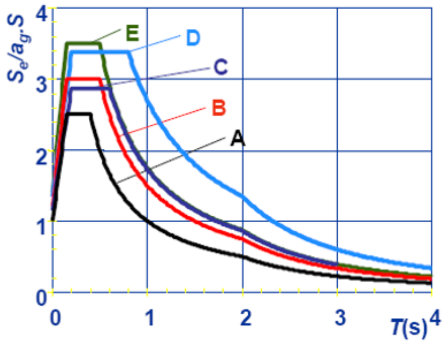


Fig. 34. Shape of the elastic response spectrum Type 1 according to [7]

Table 4. Values of the parameters describing the recommended Type 1 elastic response spectra [7]

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

The ground type can be classified according to the value of the average shear wave velocity $v_{s,30}$ (Table 5) [7].

Table 5. Ground types [7]

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

6.2 Geophysical investigations of soil characteristics at the test site

The test site was investigated to classify soils according to [7] and define local ground conditions and its influence to the local seismic hazard in the HR test site (Fig. 35). Three seismic lines were acquired in May 2019 in the test site [7] (Fig. 36). A velocity analysis based on travel time tomography of P, SV, and SH arrivals, acquired on three seismic lines on the shore of the Kaštela Bay, was performed. The $V_{S,30}$ map along each line was also computed by averaging the vertical V_{SH} tomographic values from the surface to a depth of 30 m. Relatively high obtained values, between 1.2 and 1.7 km/s, indicate the presence of shallow hard rock [13, 14], which can be classified as soil type A according EN 1998-1:2011 [11].

Fig. 35. Geophysical survey operated by OGS in Kaštel Kambelovac, May 14-24th, 2019.

Fig. 36. a) Location of the town of Kaštela on the Dalmatian coast. b) Position of the seismic lines in the historical centre of Kaštela. Image taken from Google Earth on April 3rd 2020 [14]

Considering the results of the velocity analysis based on travel time tomography of P, SV and SH arrivals acquired on three seismic lines on the shore of the Kaštela Bay (Croatia), the size of the test area and knowledge about ground properties from previous geotechnical investigations, the whole area was taken to have type A ground. Thus, the seismic hazard for all buildings in the test area has been assumed to be constant for all buildings in the area.

Spatial distribution of the seismic hazard of the buildings over the test area for return period T=475 years is presented in Fig. 37.

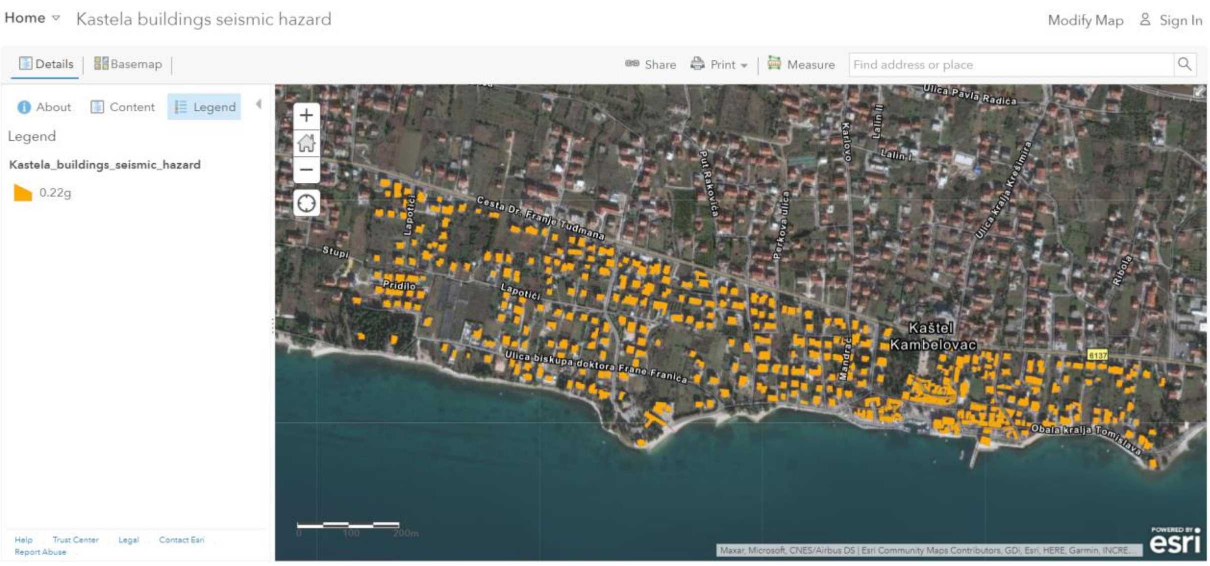


Fig. 37. Spatial distribution of the seismic hazard over the HR test area, T=475 years

7 Static non-linear (pushover) analysis of representative buildings in the HR test site

7.1 Generally about static non-linear analysis

The seismic vulnerability assessment of the buildings in the urban centres by vulnerability index method represents a fast approach for vulnerability evaluations of a large number of buildings. The approach is appropriate in analysis of vulnerability on a territorial scale and may be of great importance in seismic risk prevention and management. The vulnerability index method [9] is used for the first time in analysis of historical masonry buildings characteristic, not only for the selected test area in the PMO-GATE project, but also for the wider Dalmatian area. Therefore, one of the aim of the project, is additional vulnerability assessment by evaluation of the global structural capacity of representative buildings, based on the peak ground acceleration of capacity of the structure. Non-linear method and calculation of damage and indicator of the seismic risk [10] will be used in order to achieve this goal.

Two main methodologies for the evaluation of the global structural safety according to Eurocode 8 [11] (and corresponding Croatian standard HRN EN 1998-1:2011 [7]) can be adopted: static non-linear analysis, carried out under the conditions of constant gravity loads and monotonically increasing lateral horizontal loads, and dynamic non-linear (time history) analysis. In this project, the static non-linear analysis has been chosen to investigate the vulnerability of the structures expressed in terms of the peak ground acceleration. The analysis will result with the pushover curves which represents structural capacity up to the collapse. Therefore, the pushover curve is reliable indicator of the post-elastic behaviour of the structure.

According to European and Croatian seismic regulations [7, 10-11], when pushover analysis used for structural evaluation, the response of the structure should be 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.

The pushover curve represents the total base shear of the structure related to the corresponding top displacement. It is obtained by gradually increasing of the structural lateral load, accompanied by transferring of the structural elements from elastic to plastic range, with the decreasing of the stiffness as consequence. When the maximum base shear decrease greater than 20%, the analysis finishes, according to the HRN EN 1998-3 [12].

The seismic evaluation, which implies checking if the structure satisfies seismic demand, is performed on equivalent single degree of systems (SDOF). Pushover curve obtained for multi degree of freedom system (MDOF) is convert to SDOF by scaling using transformation factor $\Gamma = \frac{\sum m_i \phi_i}{\sum m_i \phi_i^2}$, where m_i is the mass of the node i and ϕ_i is the i-th component of the eigenvector. For MDOF base shear force F_b and associated top displacement d_c, the corresponding SDOF scaled values are $F_b^* = F_b / \Gamma$ and $d_c^* = d_c / \Gamma$. The peak base shear force is given by $F_{bu}^* = F_{bu} / \Gamma$.

The seismic demand is defined by elastic acceleration response spectrum $S_{ae}(T)$, where the spectral acceleration is defined as a function of the natural period T of the structure. Type 1 response spectrum and soil class A are used for HR test site Kaštel Kambelovac. The design ground acceleration defined by seismic hazard map for the return period of 475 years is equal to a_g=0.22g. In PMO-GATE project, the seismic capacity of the buildings will be defined by checking if the seismic demand represents with 475 years is satisfied.

7.2 Application of static non-linear analysis to historical centre of HR test site

TREMURI software [13] developed and distributed by S.D.A.DATA is used to evaluate seismic vulnerability of historical masonry buildings at HR test site Kaštel Kambelovac.

Complete 3D models of masonry structures can be obtained assembling 2-nodes macro-elements, representing the non-linear behaviour of masonry panels and piers. The macro-element considers both the shear-sliding damage failure mode and its evolution, controlling the strength deterioration and the stiffness degradation, and rocking mechanisms, with toe crushing effect, modelled by means of phenomenological non-linear constitutive law with stiffness deterioration in compression. Monotonic pushover analyses provide capacity curves which can be used for seismic evaluation of the buildings.

Analysis of each structure is performed according the following schedule:

- Creation of three-dimensional model in the TREMURI software
- Application of pushover analyses in the two main orthogonal direction of the buildings. Three lateral load distributions (uniform, linear and modal distribution) with the presence of eccentricity in positive and negative direction give in total 36 analyses.
- Each pushover analysis results with the MDOF capacity curve. After the transformation of the MDOF curve in the SDOF one bilinear curve is obtained. Then, capacity of the structure expressed in terms of peak ground acceleration corresponding to the end of bilinear curve PGA_{NC} is calculated. Additionally, peak ground accelerations for yield point (damage limitation) PGA_{DL} nd significant damage PGA_{SD} for structural model are determined.

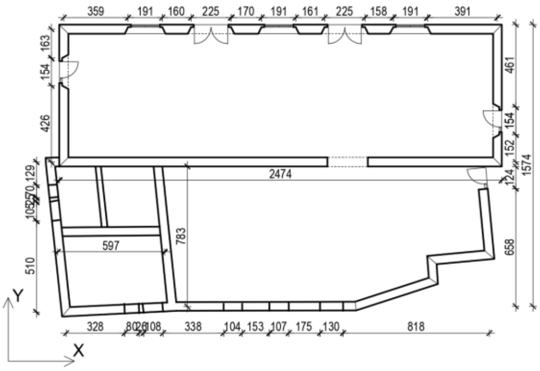
The procedure of the evaluation of the global structural capacities, based on the peak ground accelerations for different limit states are shown below on the examples of Public Library and Cambi tower. The results of critical peak ground accelerations for all analysed buildings are given in Sections 7.2.3 and 7.2.4.

7.2.1 Static non-linear analysis of Public Library

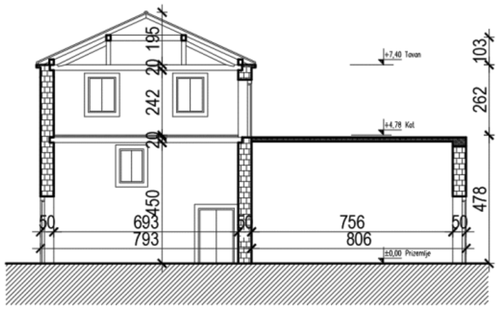
Public Library is a stone masonry structure built in the 19th century (Fig. 38). It has three floors (ground floor, first floor, and attic) and is intended for public usage. It consists of a library and a hall for events on the ground floor and offices on the first floor. The structure is made of stone walls and wooden floors. The walls were built of natural carved stone, 50-cm thick, with medium-quality mortar. The roof above the attic was made as a wooden structure with double-sided water lines and a tile cover. The building is irregular in plan and elevation. The floors are modelled as flexible. Geometrical characteristics and structural model are shown in Fig. 39.

Fig. 38. Public Library: photo and position of the building.

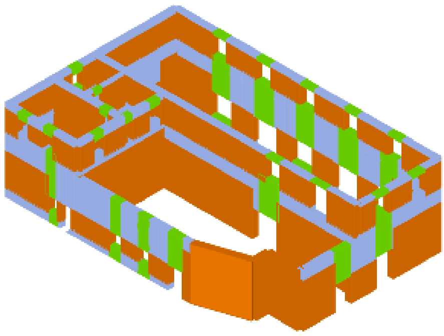
Ground floor plan:



Section cut:



Structural model:



Rendered model:

Fig. 39. Public Library: geometrical characteristics and structural model

The material properties of the walls were estimated according to [15], considering the results of in situ tests of the heritage building walls [16]. The mechanical material properties of the Public Library were assumed as an average of the reference values for building walls made of cut stone masonry with good bonding [15] as follows: compressive strength 2.60 MPa, tensile strength 0.10 MPa, modulus of elasticity 1700 MPa, shear modulus 580 MPa, and specific weights 21 kN/m^3. A limited knowledge level (KL1) of the building and a confidence factor of CF = 1.35 according to HRN EN 1998-3 [11] were assumed.

The seismic demand was defined by the elastic acceleration response spectrum. A type 1 response spectrum [7] and soil class A [7] were used for the test site. The input parameters taken in the seismic analysis were the importance factor γ_I = 1.2, the design ground acceleration ag = 0.22 g, and the soil factor S = 1.0. Fig. 40 shows the results of a total of 36 pushover analyses for uniform, linear and modal distributions of lateral forces obtained by TREMURI software. The behavior of the building was different along the two main directions and depended on the direction of the lateral loads.

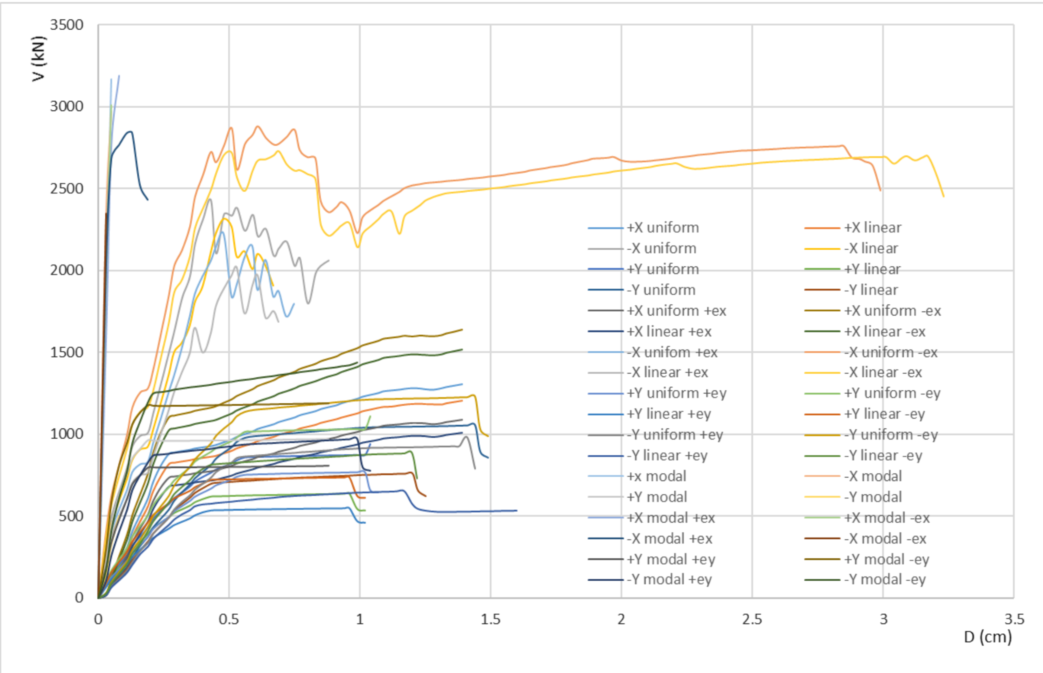


Fig. 40. Pushover curves for the Public Library building

The seismic capacity was evaluated, comparing the displacement capacity and the displacement demand obtained through a pushover analysis for the same control point. This procedure was performed for each of 36 cases and for two orthogonal directions (Table 6).

Each analysis result with MDOF pushover curve which is transformed in SDOF curve. After that, bilinearization of the SDOF curve is performed. Bilinear curve is used to validate structural performance of the building represented with the displacement and spectral acceleration for three significant state. Beginning of yielding is assumed as early damage or damage limitation state and associated with peak ground acceleration equal to $PGA_y = PGA_{DL}$ (see explanation in Deliverable 3.3.1). Significant damage state with acceleration PGA_{SD} corresponds to ¾ of the ultimate displacement capacity. Global structural capacity of the building is taken equal to the ultimate displacement capacity and named as near collapse state with corresponding peak ground acceleration $PGA_C = PGA_{NC}$.

The pushover curves which give the lowest capacity accelerations PGA_C in the x and y directions are shown in Fig. 41, together with their bilinear idealizations, which are essential for capacity identification. The lowest capacity in both directions was obtained for the linear distribution.

Table 6. Results of pushover analyses (PGA_{DL}, PGA_{SD} and PGA_{NC}) for structural model

Direction	Load	Eccentricity	PGA_{DL}/g	PGA_{SD}/g	PGA_{NC}/g
+x	uniform	0	0,063	0,115	0,148
+x	linear	0	0,057	0,112	0,144
+x	modal	0	0,217	0,223	0,264
−x	uniform	0	0,121	0,156	0,188
−x	linear	0	0,121	0,125	0,147
−x	modal	0	0,235	0,245	0,257
+y	uniform	0	0,046	0,074	0,095
+y	linear	0	0,033	0,068	0,088
+y	modal	0	0,051	0,134	0,168
−y	uniform	0	0,054	0,102	0,132
−y	linear	0	0,039	0,081	0,105
−y	modal	0	0,060	0,139	0,173
+x	uniform	+5%	0,052	0,100	0,130
+x	uniform	−5%	0,079	0,136	0,171
+x	linear	+5%	0,048	0,095	0,123
+x	linear	−5%	0,072	0,130	0,166
+x	modal	+5%	0,248	0,251	0,264
+x	modal	−5%	0,231	0,234	0,245
−x	uniform	+5%	0,109	0,126	0,151
−x	uniform	−5%	0,143	0,435	0,553
−x	linear	+5%	0,098	0,109	0,131
−x	linear	−5%	0,136	0,436	0,557
−x	modal	+5%	0,229	0,321	0,413
−x	modal	−5%	0,107	0,156	0,208
+y	uniform	+5%	0,040	0,069	0,089
+y	uniform	−5%	0,055	0,084	0,107
+y	linear	+5%	0,028	0,061	0,079
+y	linear	−5%	0,038	0,078	0,100
+y	modal	+5%	0,042	0,114	0,144
+y	modal	−5%	0,066	0,170	0,212
−y	uniform	+5%	0,048	0,091	0,118
−y	uniform	−5%	0,063	0,114	0,146
−y	linear	+5%	0,031	0,093	0,122
−y	linear	−5%	0,045	0,089	0,115
−y	modal	+5%	0,049	0,123	0,156
−y	modal	−5%	0,071	0,155	0,194

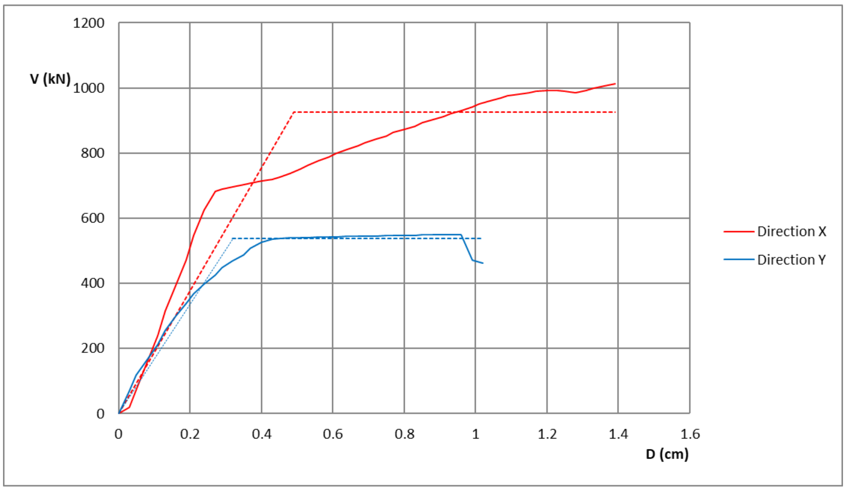


Fig. 41. The worst cases of pushover curves

The peak ground accelerations associated with the collapse limit state were computed according to EC8 [7, 10]. The capacity acceleration in terms of the collapse for the case study is equal to 0.123 g (0.561 a_g) in the x direction and 0.079 g (0.363 a_g) in the y direction, where a_g represents the design ground acceleration, defined by the seismic hazard map for the return period of 475 years, and it is equal to a_g = 0.22 g.

The same procedure was applied to obtain the peak ground accelerations PGA_y, corresponding to the yield point. It is worth mentioning that the lowest values of PGA_y and PGA_C do not necessarily correspond to the same distribution of lateral forces. In this case, the capacity acceleration in terms of the yield is equal to 0.048 g (0.218 a_g) in the x-direction and 0.028 g (0.130 a_g) in the y-direction.

Local mechanism failure was also analyzed to check local mechanisms such as those induced by a lack of connection among perpendicular walls, and poor connec-tions among floors/roofs and walls. In the case of the Public Library building, the analysis showed that the lowest value of the failure acceleration for the local mechanism is 0.130 g. Hence, critical acceleration was obtained through global failure analysis.

Figs. 42 and 43 shows state of the damage for the building for the worst cases of pushover curves in x and y direction.

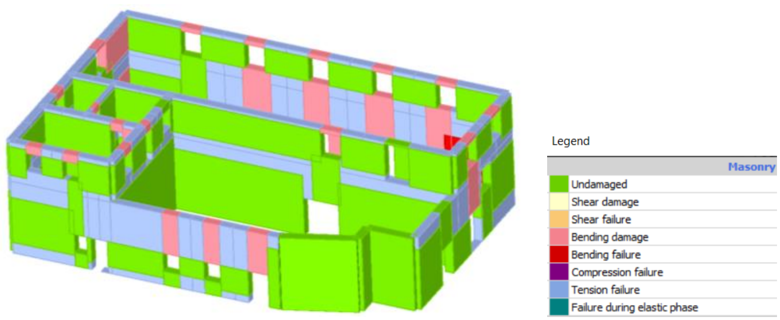


Fig. 42. State of damage for analysis in x direction

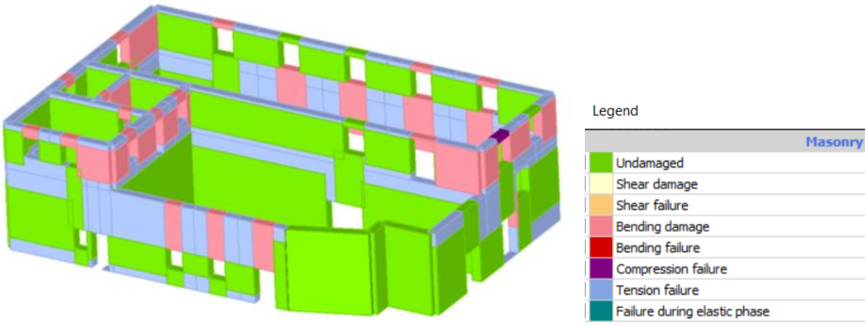


Fig. 43. State of damage for analysis in y direction

7.2.2 Static non-linear analysis of Cambi Tower

Cambi Tower has been built in 15th century as stone masonry structure (Fig. 44). It is composed of 4 parts. Among them the tower stands out. The structure consists of stone walls, flat and circular, wooden floors and wooden roof. It was made of roughly shaped blocks, poorly organized with poor quality of mortar. The building is irregular in plan and elevation. The floors are modelled as flexible. Geometrical characteristics and structural model are shown in Fig. 45.

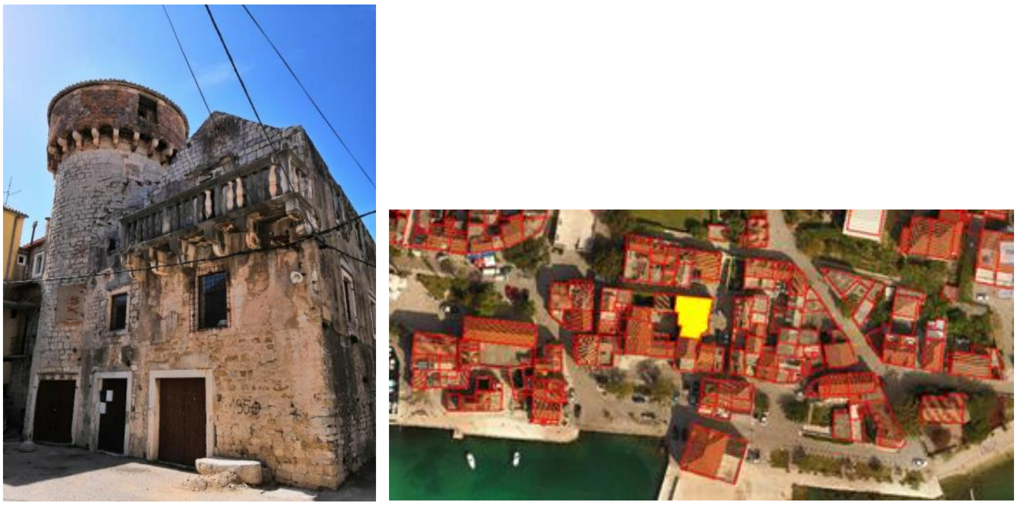
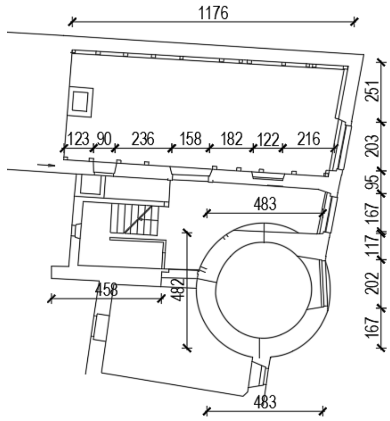


Fig. 44. Cambi Tower: photo and position of the building

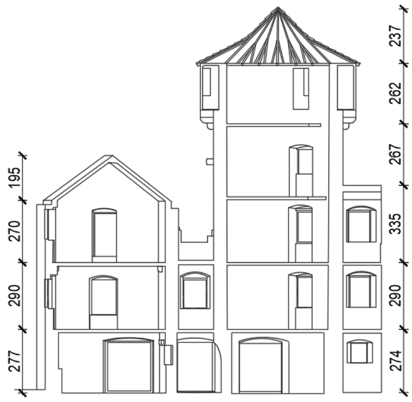
The material properties of the walls were estimated according to [15], considering the results of in situ tests of the heritage building walls [16]. The mechanical material properties of the Public Library were assumed as the reference values for building walls made of irregular stone masonry [15] as follows: compressive strength 1.80 MPa, tensile strength 0.048 MPa, modulus of elasticity 1050 MPa, shear modulus 350 MPa, and specific weights 21 kN/m^3. A limited knowledge level (KL1) of the building and a confidence factor of CF = 1.35 according to HRN EN 1998-3 [11] were assumed.

The seismic demand was defined by the elastic acceleration response spectrum. A type 1 response spectrum [7] and soil class A [7] were used for the test site. The input parameters taken in the seismic analysis were the importance factor γ_I = 1.2, the design ground acceleration a_g = 0.22 g, and the soil factor S = 1.0. Fig. 46 shows the results of a total of 36 pushover analyses for uniform, linear and modal distributions of lateral forces obtained by TREMURI software. The behavior of the building was different along the two main directions and depended on the direction of the lateral loads.

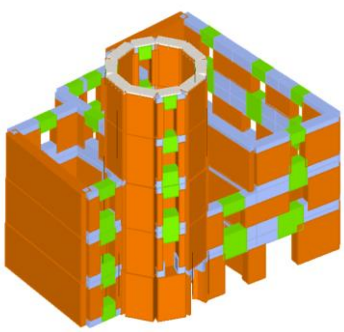
Ground floor plan:



Section cut:



Structural model:



Rendered model:

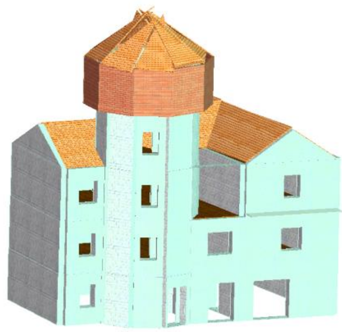


Fig. 45. Geometrical characteristics and structural model

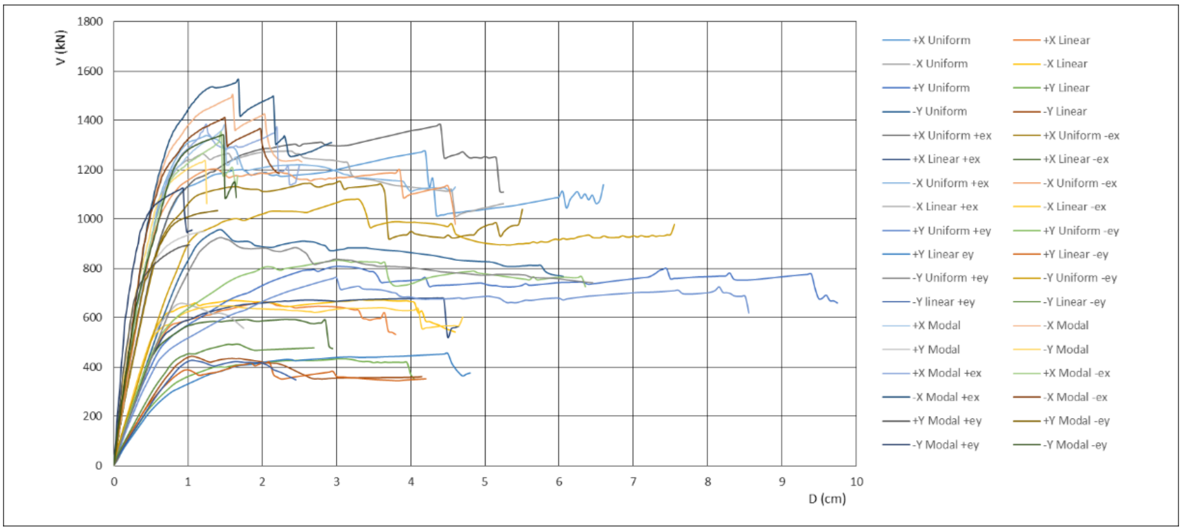


Fig. 46. Capacity curves for uniform, linear and modal distribution of the forces

The seismic capacity was evaluated, comparing the displacement capacity and the displacement demand obtained through a pushover analysis for the same control point. This procedure was performed for each of 36 cases and for two orthogonal directions (Table 7).

Table 7. Results of pushover analyses (PGA_{DL}, PGA_{SD} and PGA_{NC}) for structural model

Direction	**Load**	**Eccentricity**	**PGA_{DL}/g**	**PGA_{SD}/g**	**PGA_{NC}/g**
+x	uniform	0	0,255	0,479	0,619
+x	linear	0	0,133	0,203	0,264
+x	modal	0	3,556	4,824	5,830
−x	uniform	0	0,258	0,435	0,558
−x	linear	0	0,138	0,271	0,353
−x	modal	0	3,775	6,949	8,583
+y	uniform	0	0,130	0,326	0,432
+y	linear	0	0,072	0,113	0,151
+y	modal	0	1,998	3,012	3,659
−y	uniform	0	0,149	0,266	0,348
−y	linear	0	0,072	0,131	0,174
−y	modal	0	2,699	3,416	4,074
+x	uniform	+5%	0,274	0,389	0,498
+x	uniform	−5%	0,238	0,408	0,526
+x	linear	+5%	0,140	0,241	0,314
+x	linear	−5%	0,123	0,163	0,211
+x	modal	+5%	3,563	6,314	7,816
+x	modal	−5%	3,432	4,771	5,780
−x	uniform	+5%	0,257	0,389	0,497
−x	uniform	−5%	0,247	0,384	0,491

−x	linear	+5%	0,093	0,116	0,147
−x	linear	−5%	0,132	0,275	0,359
−x	modal	+5%	3,906	7,778	9,667
−x	modal	−5%	3,634	6,345	7,799
+y	uniform	+5%	0,120	0,258	0,344
+y	uniform	−5%	0,134	0,249	0,327
+y	linear	+5%	0,083	0,121	0,162
+y	linear	−5%	0,064	0,128	0,170
+y	modal	+5%	1,877	2,877	3,474
+y	modal	−5%	2,202	3,182	3,874
−y	uniform	+5%	0,141	0,270	0,354
−y	uniform	−5%	0,167	0,340	0,445
−y	linear	+5%	0,030	0,059	0,078
−y	linear	−5%	0,081	0,091	0,120
−y	modal	+5%	2,371	3,556	4,246
−y	modal	−5%	2,895	3,793	4,586

Each analysis result with MDOF pushover curve which is transformed in SDOF curve. After that, bilinearization of the SDOF curve is performed. Bilinear curve is used to validate structural performance of the building represented with the displacement and spectral acceleration for three significant state. Beginning of yielding is assumed as early damage or damage limitation state and associated with peak ground acceleration equal to $PGA_y = PGA_{DL}$ (see explanation in Deliverable 3.3.1). Significant damage state with acceleration PGA_{SD} corresponds to ¾ of the ultimate displacement capacity. Global structural capacity of the building is taken equal to the ultimate displacement capacity and named as near collapse state with corresponding peak ground acceleration $PGA_C = PGA_{NC}$.

The pushover curves which give the lowest capacity accelerations PGA_C in the x and y directions are shown in Fig. 47, together with their bilinear idealizations, which are essential for capacity identification. The lowest capacity in both directions was obtained for the linear distribution.

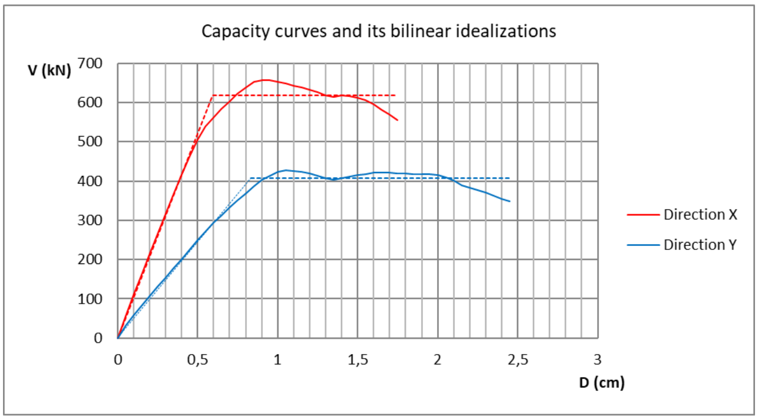


Fig. 47. The worst cases of pushover curves

The capacity acceleration in terms of the collapse for the case study is equal to 0,147 g (0,668 a_g) in the x direction and 0,078 g (0,355 a_g) in the y direction, where a_g represents the design ground acceleration, defined by the seismic hazard map for the return period of 475 years, and it is equal to a_g = 0,22 g. Finally, seismic capacity of the Cambi Tower, shown in terms of collapse acceleration of the building, is equal to 35,5% of demand acceleration PGA_D.

The same procedure was applied to obtain the peak ground accelerations PGA_Y, corresponding to the yield point. The capacity acceleration in terms of the yield is equal to 0.093 g (0.423 a_g) in the x-direction and 0.030 g (0.136 a_g) in the y-direction. Figs. 48 and 49 shows state of the damage for the building for the worst cases of pushover curves in x and y direction.

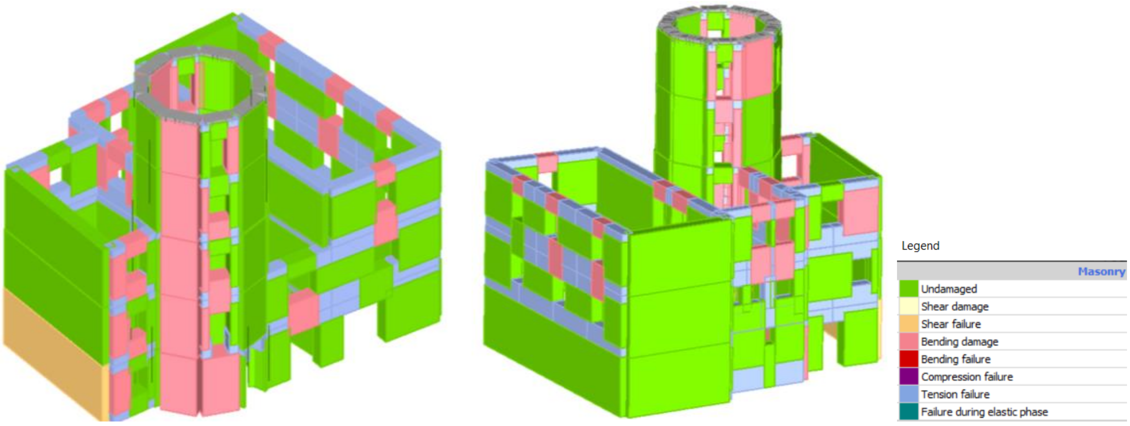


Fig. 48. State of damage for analysis in x direction

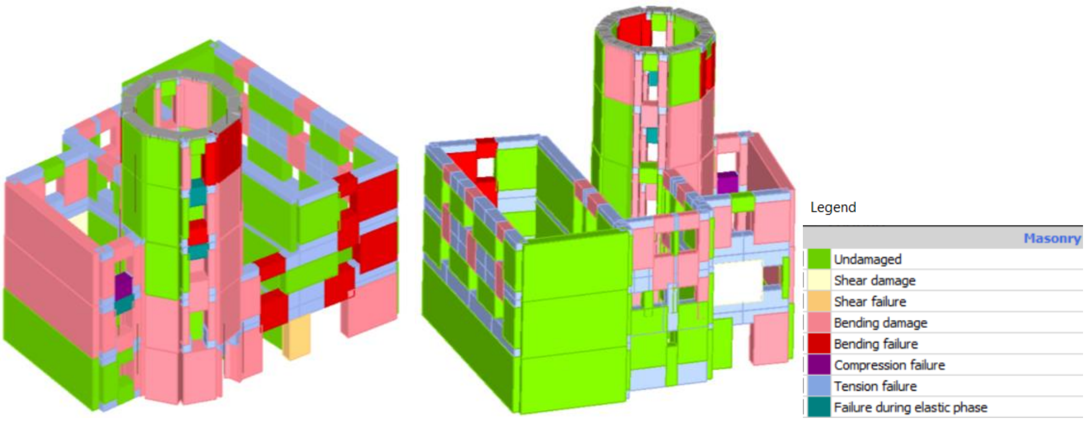


Fig. 49. State of damage for analysis in y direction

7.2.3 Results of static non-linear analysis for the buildings in historical center

Eleven stone masonry buildings (Fig. 50) in historical center have been analysed by static non-linear analysis.

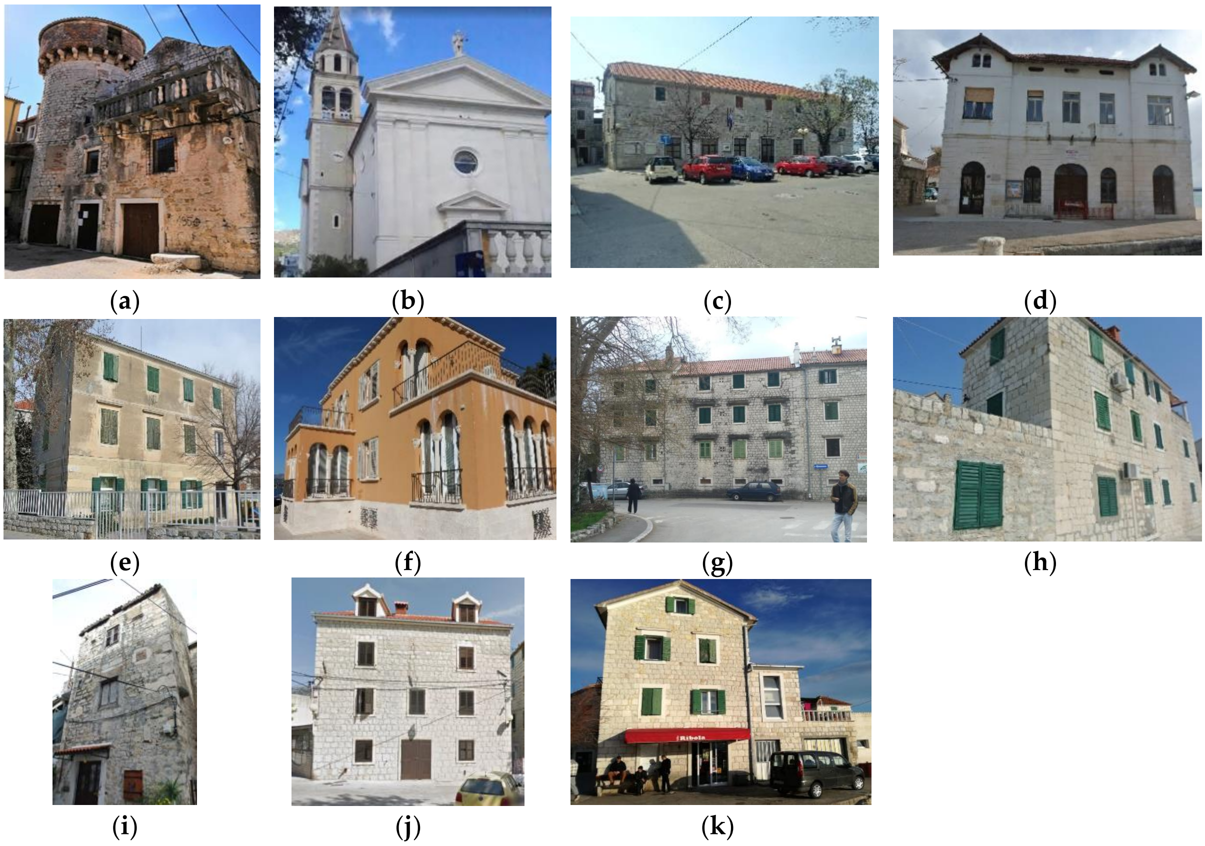


Fig. 50. Analysed buildings: (a) Cambi Tower; (b) St. Mihovil Church; (c) Public Library; (d) Rowing club; (e) Kindergarten; (f) Ballet School; (g) Dudan Palace; (h) Folk Castle; (i) Kumbat Towers; (j) Residential building—Obala kralja Tomislava 1; (k) Perišin house (Ribola).

Seismic capacity of the buildings have been expressed in terms of the collapse acceleration corresponding to the end of the capacity (pushover) curve. The results for those buildings are shown in the Table 8.

Numerical predictions of the maximum value of peak ground acceleration corresponding to the collapse of the structure obtained by non-linear static analyses show that no building meets the seismic requirement equal to a_g=0,22g in either direction. Namely, the peak ground acceleration corresponding to the collapse of the buildings are in the range of 0,078g and 0,183g. The highest resistance of 0,183g is achieved for Ballet School because the building was completely reconstructed with RC floors and RC walls. Several other buildings that have been partially restored and reconstructed (Ribola Building, Residential Building) have achieved collapse acceleration up to 0,152g. The buildings with originally stone masonry walls and flexible wooden floors collapsed for the significant lower accelerations between 0,078g and 0,102g. The failure occurs due to different collapse modes such as shear, bending, tension and compression failures. The analyses show that pushover analysis of stone masonry buildings can provide an insight into both global seismic resistance and the mechanisms that lead to the structural failure.

Table 8. Results of pushover analyses for the chosen stone masonry buildings

No.	Building	PGA_C (m/s^2) x direction	PGA_C (m/s^2) y direction	Collapse acceleration PGA_C / g
1	Cambi Tower	1,559	0,768	0,078
2	St. Mihovil’s Church	1,158	1,001	0,102
3	Public Library	1,126	0,780	0,079
4	Rowing Club	1,896	1,382	0,141
5	Kindergarten	0,912	0,907	0,092
6	Ballet School	2,404	1,796	0,183
7	Dudan’s Palace	0,814	0,888	0,083
8	Folk Castle	0,793	0,788	0,080
9	Kumbat’s Towers	1,857	1,010	0,103
10	Residential building, Obala kralja Tomislava 1	1,619	1,495	0,152
11	Perišin house (Ribola)	1,184	2,281	0,121

The same procedure was applied to obtain the peak ground accelerations PGA_y, which is an indicator for damage limitation state ($PGA_y = PGA_{DL}$), and PGA_{SD} which corresponds to significant damage state.

The critical peak ground acceleration results for three limit states and vulnerability indexes of the buildings are shown in the Table 9.

Table 9. Vulnerability index I_V and critical accelerations PGA_{DL}, PGA_{SD} and PGA_{NC} for three limit states

No.	Building	Vulnerability index I_v [%]	PGA_{DL} [g]	PGA_{SD}[g]	PGA_{NC} [g]
1	Cambi Tower	76,9	0,030	0,059	0,078
2	St. Mihovil Church	40,5	0.057	0,086	0,102
3	Public Library	59,0	0,028	0,061	0,079
4	Rowing Club	40,2	0,064	0,110	0,141
5	Kindergarten	41,0	0,059	0,070	0,092
6	Ballet School	23,9	0,103	0,142	0,183
7	Dudan Palace	50,1	0,051	0,068	0,083
8	Folk Castle	58,7	0,081	0,061	0,080
9	Kumbat’s Towers	65,2	0,057	0,087	0,103
10	Residential building, Obala kralja Tomislava 1	34,8	0,081	0,095	0,152
11	Perišin house (Ribola)	48,7	0,058	0,061	0,121

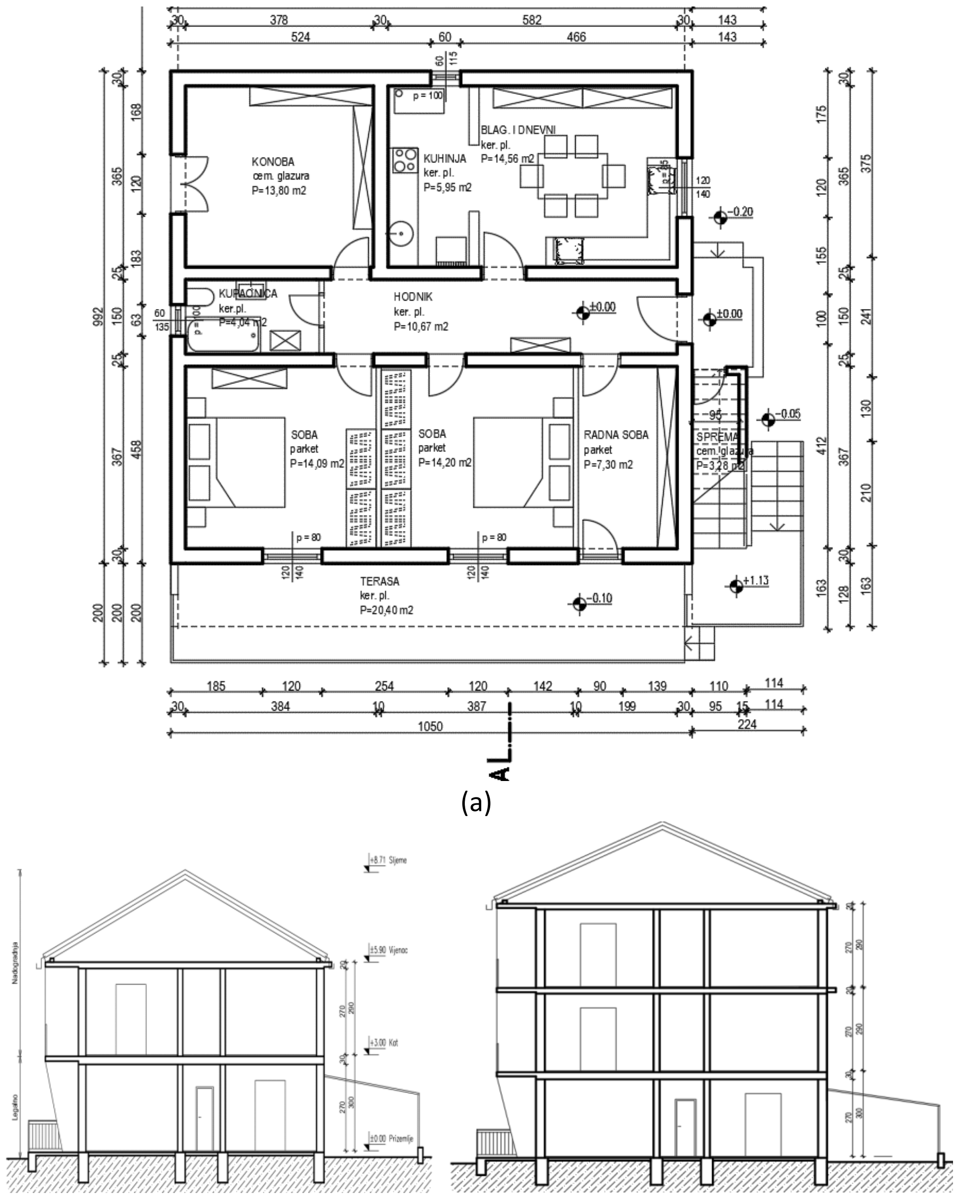
7.2.4 Results of static non-linear analysis for the buildings outside of historical centre

A part of test site outside of the historical centre consists of the buildings dating from the beginning of the 20th century to the present. 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.

In the northern part, which does not belong to the protected historic centre, there are a number of stone masonry buildings constructed before 1948. In the eastern and western parts of the test site, the buildings are built with concrete or clay blocks, without confinement, only with confinement consist of horizontal tie beams or with horizontal and vertical confinement (beams and columns), depending on the construction period.

Eight masonry buildings typical for the constructions outside of historical center have been analysed by static non-linear analysis. The sample includes the buildings built of concrete or brick hollow blocks after the 1948 and can be classified according to the construction period. Between them there are following categories of the buildings: (1) unbounded concrete masonry built before the first seismic regulation in 1964; (2) concrete masonry with horizontal RC beams typical for the period between 1964 and 1980; (3) concrete masonry with horizontal RC beams and RC columns built between 1980 and 2005 and (4) clay masonry with horizontal RC beams and RC columns which are seismically resistant structures due to the applications of modern design standards based on Eurocode 8. The buildings have rigid RC slabs while the roof is mainly wooden with roof tiles. Two different elevation configuration have been analysed: (a) P+1 which consists of ground, one floor and roof; and (b) P+2 which consists of ground, two floors and roof. Typical buildings outside of historical centre are shown in Fig. 51. Plan and section view of the buildings are shown in Fig. 52.

Fig. 51. Typical buildings outside of historical centre.



(a)

(b) (c)

Fig. 52. Typical configuration outside of historical centre: (a) plan; (b) section view P+1; (c) section view P+2

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. The critical PGA results and vulnerability indexes for the analysed buildings are shown in Table 10.

Table 10. Results of pushover analyses for the buildings outside of historical centre

No.	Building	Number of floors	Building period	PGA_{DL} [g]	PGA_{SD}[g]	PGA_{NC} [g]
1	Clay masonry, RC horizontal and vertical confinement, EC8	P+1	After 2005	0,130	0,218	0,270
2	Clay masonry, RC horizontal and vertical confinement, EC8	P+2	After 2005	0,103	0,188	0,243
3	Concrete masonry, RC horizontal and vertical confinement	P+1	1980-2005	0,115	0,187	0,220
4	Concrete masonry, RC horizontal and vertical confinement	P+2	1980-2005	0,065	0,158	0,189
5	Concrete masonry, RC horizontal confinement	P+1	1964-1980	0,075	0,175	0,206
6	Concrete masonry, RC horizontal confinement	P+2	1964-1980	0,098	0,145	0,175
7	Unconfined concrete masonry	P+1	Before 1964	0,083	0,144	0,173
8	Unconfined concrete masonry	P+2	Before 1964	0,061	0,114	0,142

8 Development of vulnerability curves

This chapter aims to express seismic risk of buildings in terms of the damage caused by an earthquake of a certain intensity. The vulnerability indexes for the buildings at the test site were linked with the critical yield and collapse peak ground accelerations obtained by static non-linear analysis and new damage-vulnerability-peak ground acceleration relationships were derived. The damage index is expressed in the [0–1] space via a tri-linear law, 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). Procedure of the development of the new damage-vulnerability-peak ground acceleration relationships, named as vulnerability curves, have been presented in Deliverable 3.3.1.

8.1 Vulnerability curves and damage for the buildings in historical centre

The results for the analysed buildings in the historical centre, represented with vulnerability indexes and critical yield PGA_y and collapse PGA_C accelerations are shown in Table 11.

Table 11. Vulnerability index, yield acceleration, and collapse acceleration of the buildings

No.	Building	Vulnerability index I_v [%]	Yield acceleration PGA_y [g]	Collapse acceleration PGA_C [g]
1	Cambi Tower	76,9	0,030	0,078
2	St. Mihovil Church	40,5	0.057	0,102
3	Public Library	59,0	0,028	0,079
4	Rowing Club	1,896	0,064	0,141
5	Kindergarden	41,0	0,059	0,092
6	Ballet School	23,9	0,103	0,183
7	Dudan Palace	50,1	0,051	0,083
8	Folk Castle	58,7	0,081	0,080
9	Kumbat's Towers	65,2	0,057	0,103
10	Residential building, Obala kralja Tomislava 1	34,8	0,081	0,152
11	Perišin house (Ribola)	48,7	0,058	0,121

Fig. 53 shows the relationship between the vulnerability index and collapse / yield accelerations for 11 buildings in the historical centre. The cloud of points represents the sample of buildings analysed with the pushover analysis. The trend lines I_v–PGA_y and I_v–PGA_c for the yield and collapse states were obtained. The most representative functions were chosen. They can be used to approximately evaluate the yield and collapse peak ground accelerations for the historic centre of Kaštel Kambelovac.

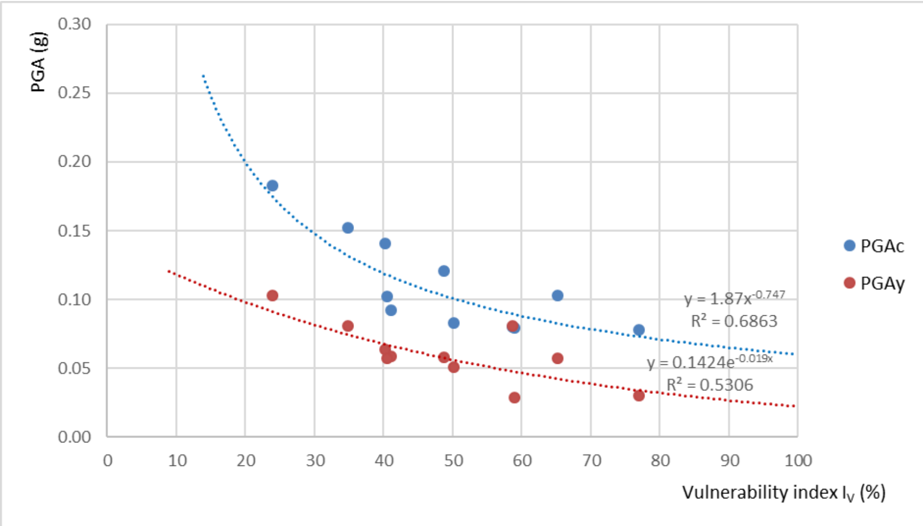


Fig. 53. Trend lines I_v–PGA_y and I_v–PGA_c

The vulnerability indexes for the buildings at the test site and the trend lines shown in Fig. 53 were used to obtain the peak ground accelerations for early damage and collapse state. Fig. 54 shows the collapse accelerations for the buildings in the historical center of the test site.

Fig. 54. Map of collapse accelerations

Definition of the vulnerability curves for the historical center is next step to define seismic risk of the buildings. Tri-linear vulnerability curves have been determined using yield and collapse peak ground accelerations, PGA_y and PGA_c, obtained according the procedure developed and shown in [15] and using the relations shown in Fig. 46. 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. These vulnerability curves are shown in Fig. 48.

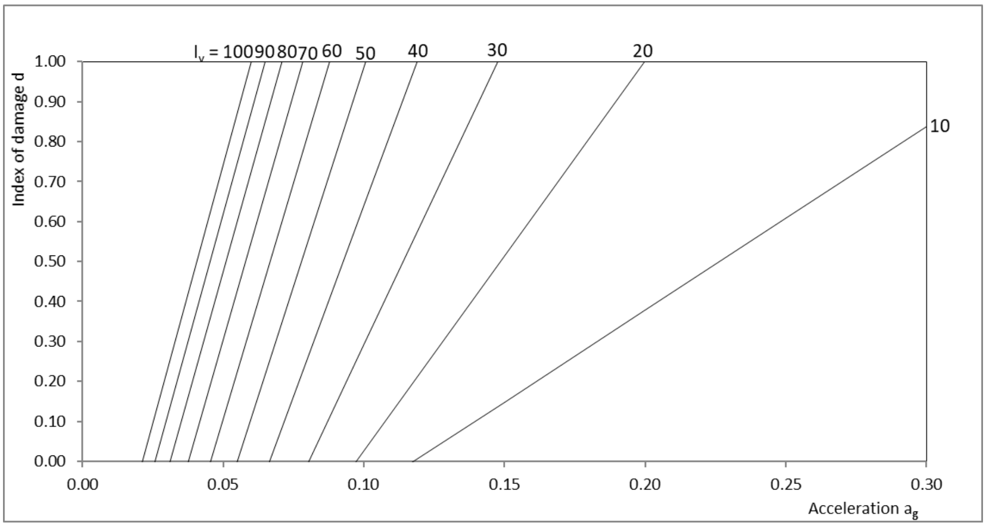


Fig. 48. Vulnerability curves for the historic center of Kaštel Kambelovac

The obtained vulnerability curves were exploited to define the damage index based on the vulnerability indexes. In the Kaštela area, the seismic hazard measured via the peak ground acceleration for the return periods of 475 and 95 years is equal to a_g = 0.22 g and a_g = 0.11 g for soil type A, respectively.

The damage indexes of the buildings were determined for the corresponding peak horizontal ground accelerations of 0.22 g and 0.11 g, and are presented in the maps shown in Figs. 49 and 50.

Fig. 49. Map of the damage index for PGA = 0.11 g and a return period T = 95 years

Fig. 50. Map of the damage index for PGA = 0.22 g and a return period T = 475 years

8.2 Vulnerability curves and damage for the buildings at the whole test site

The results for 19 analysed buildings (11 in the historical centre and 8 outside of the centre) represented in tables 8 and 9, are used to establish relationship vulnerability index – peak ground acceleration for the DL, SD and NC limit states at the whole test site.

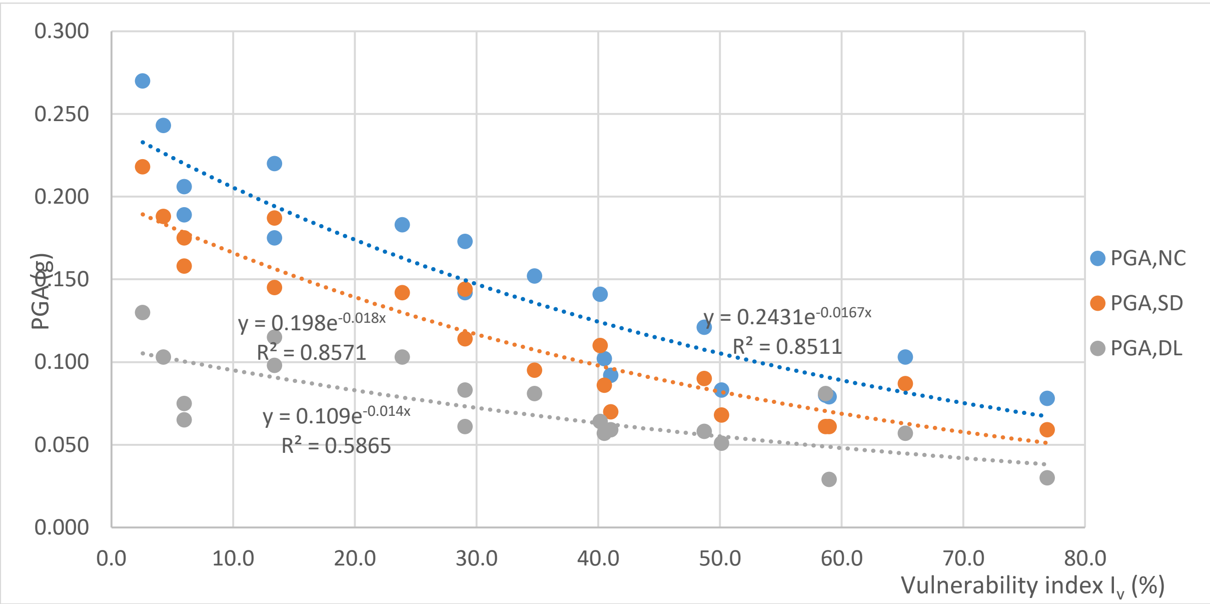


Fig. 51. Trend lines I_v–PGA_{NC}, I_v–PGA_{SD} and I_v–PGA_{SD}

Fig. 51 shows cloud of points representing the relationship between the vulnerability index calculated on the basis of 11 parameters I_v and critical peak ground accelerations associated with the DL, SD and NC limit. The trend lines Iv–PGA_{DL}, Iv–PGA_{SD} and Iv–PGA_{NC} for the damage limitation, significant damage and near collapse states were obtained and shown in Fig. 51. The exponential functions were chosen as the most representative. They are used to approximately evaluate the yield, significant damage and collapse peak ground accelerations for the whole test site. The values of yield and collapse accelerations are basis for deriving of vulnerability curves.

Vulnerability curves are slightly changed in comparison to those shown in Fig. 53 due to the data for the buildings outside of the historical centre with the vulnerability indexes less than 30%. New vulnerability curves which are used for estimation of damage index of the buildings at the whole test site are shown in Fig. 52.

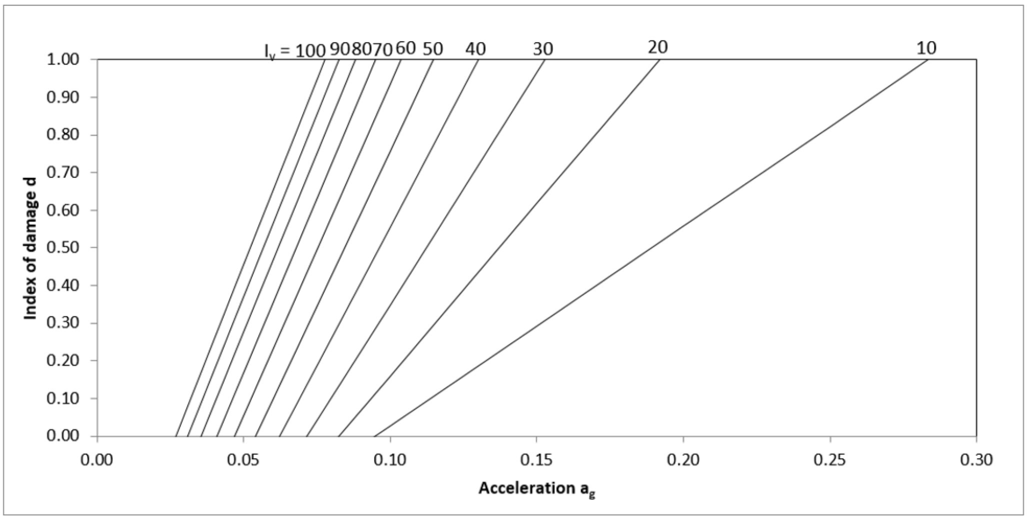
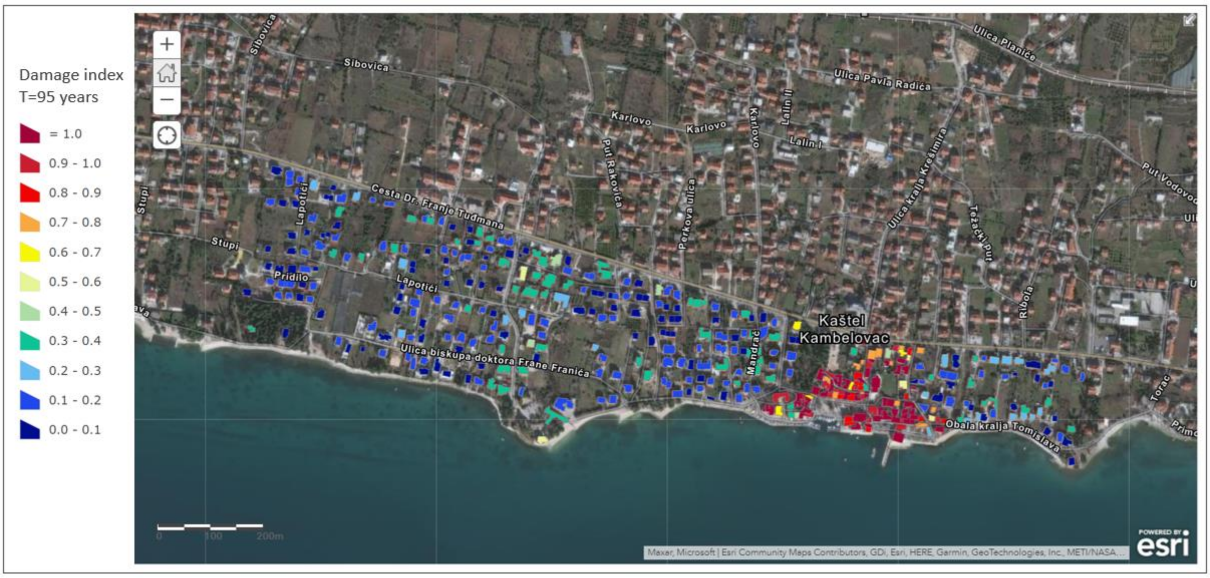
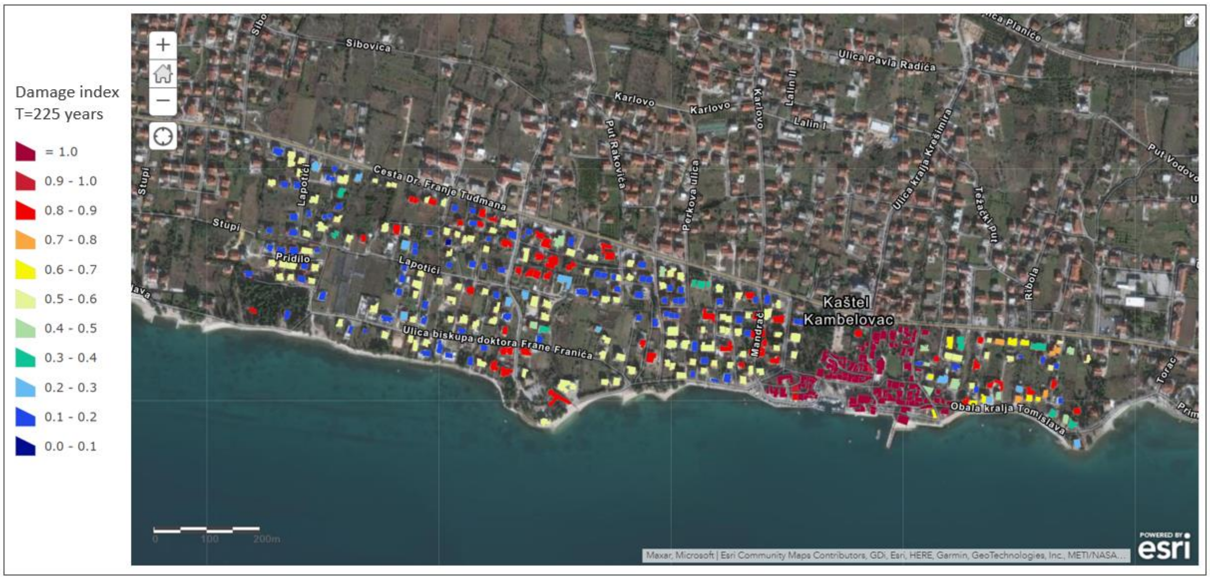


Fig. 52. Vulnerability curves for the test site

Spatial distribution of the damage is represented by the damage index maps of the investigated area for given intensity of the earthquake. Three seismic scenarios corresponding to return periods 95, 225 and 475 years and demand peak ground accelerations of 0.11g, 0.17g and 0.22g, respectively, have been chosen. The damage of the buildings for different scenarios are shown in Fig. 53.


Damage index
T=95 years
= 1.0
0.9 - 1.0
0.8 - 0.9
0.7 - 0.8
0.6 - 0.7
0.5 - 0.6
0.4 - 0.5
0.3 - 0.4
0.2 - 0.3
0.1 - 0.2
0.0 - 0.1
Kaštel Kambelovac
Cesta Dr. Franje Tuđmana
Ulica biskupa doktora Frane Franića
Obala kralja Tomislava
Ulica Pavla Radića
Ulica kralja Krešimira
Sibovica
Karlovo
Lalin I
Lapotići
Pridilo
Stupi
Mandrač
Ulica Planiče
esri

(a)


Damage index
T=225 years
= 1.0
0.9 - 1.0
0.8 - 0.9
0.7 - 0.8
0.6 - 0.7
0.5 - 0.6
0.4 - 0.5
0.3 - 0.4
0.2 - 0.3
0.1 - 0.2
0.0 - 0.1
Kaštel Kambelovac
Cesta Dr. Franje Tuđmana
Ulica biskupa doktora Frane Franića
Obala kralja Tomislava
Ulica Pavla Radića
Ulica kralja Krešimira
Sibovica
Karlovo
Lalin I
Lapotići
Pridilo
Stupi
Mandrač
esri

(b)

(c)

Fig. 53. Damage index distribution at the test site: (a) T=95 years; (b) T=225 years; (a) T=475 years.

9 Index of seismic risk of the buildings at the test site

In this chapter methodology for assessment of seismic risk at the test site, proposed in Deliverable 3.3.1 have been applied to evaluate seismic risk in terms of index of seismic risk for the buildings at the test site. The methodology 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 vulnerability index – peak ground acceleration relations for early damage, significant damage and near collapse states, presented in Fig. 51 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.

Index of seismic risk is defined as a ratio between the peak ground acceleration corresponding to the capacity of the structure PGA_C and the demand ground acceleration. It is expressed in a form:

$$\alpha_{PGA,C} = \frac{PGA_C}{PGA_D} \tag{1}$$

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

The values $\alpha_{PGA}>1$ are related to safe structures, while the values $\alpha_{PGA}<1$ are related to non-safe structures.

In this project the index of seismic risk for the collapse of the structure which corresponds to NC limit state is estimated for three return periods. The results are presented in Fig. 54.

(a)

(b)

(c)

Fig. 54. Risk maps in terms of index of seismic risk: (a) T=95 years; (b) T=225 years; (a) T=475 years.

10 Conclusion

This Deliverable presents results of seismic vulnerability assessment of the HR test site Kaštel Kambelovac. 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 static (pushover analysis). A new vulnerability-peak ground acceleration relations for different limit states corresponding to the yield, significant damage and collapse, as well as damage-vulnerability-peak ground acceleration curves has been developed using the vulnerability indexes and the critical accelerations of buildings obtained through non-linear static analysis.

The evaluation procedure consisted of the following stages:

- Identification of architectural, structural, and material characteristics of the buildings through the investigation of historical and archival documentation, literature, visual inspection, and thermographic imaging;
- Characterization of the soil type through a geophysical survey;
- Calculation of seismic vulnerability indexes for the buildings in the area;
- Calculation of the peak ground accelerations for early damage, significant damage and collapse states of the buildings through non-linear static (pushover) analysis of representative buildings;
- Development of a new damage–vulnerability–peak ground acceleration curves, which estimates the damage of the buildings under specific seismic action;
- Risk analysis in terms of seismic damage;
- Risk analysis in terms of index of seismic risk.

Results of seismic risk evaluation of the test site are represented with the seismic vulnerability indexes, damage indexes, critical accelerations for three limit states (early damage, significant damage, near collapse) and indexes of seismic risk for three scenarios represented by the return periods (95, 225 and 475 years).

Developed vulnerability, damage and risk maps are basis for seismic risk management actions. Obtained results provide a better insight into individual seismic vulnerability, the capacity and expected damage of the buildings subjected to certain seismic actions, than the insight that can be reached by simply associating buildings to a class and determining vulnerability and damage for the whole class. The results have important operational outcomes in terms of the planning and management activities for the investigated site for the purpose of reduction of seismic risk.

11 References

[1] Benedetti D, Petrini V. Sulla vulnerabilita di edifici in muratura: posta di un metodo di valutazione. L'industria delle Construzioni, Vol. 149, No. 1, pp. 66-74, 1984.

[2] Rilevamento della vulnerabilita sismica degli edifici, Istruzioni per la compilazione della scheda di 2° livello (Muratura). Roma: GNDT - Gruppo Nazionale per la Difesa dai terremoti, 1993.

[3] Ferrini M, Melozzi A, Pagliazzi A, Scarparolo S. Rilevamento della vulnerabilita sismica degli edifici in muratura, Manuale per la compilazione della Scheda GNDT/CNR di II livello, Versione modificata dalla Regione Toscana. S.l.: Regione Toscana, Direzione Generale delle Politiche Territoriale e Ambientali, Settore - Servizio Sismico Regionale, 2003.

[4] Development Strategy of Kaštela City 2016. – 2020. Kastela City, 2016. (in Croatian)

[5] Marasović K. Kaštel Kambelovac. Kaštela Journal, Vol. 7, pp. 35-61, 2003. (in Croatian)

[6] Deliverable 3.3.1. Guidelines of the assessment procedure for earthquake vulnerability in HR test site, PMO-GATE project, 2022.

[7] HRN EN 1998-1:2011. Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. Croatian Standards Institute, 2011.

[8] HRN EN 1996-1:2012. Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures. Croatian Standards Institute, 2012.

[9] Dolce M, Moroni C. Inventario e vulnerabilita degli edifici pubblici e strategici dell'Italia centro-meridionale, Vol. IV – La valutazione della vulnerabilita e del rischio sismico degli edifici publici mediante le procedure VC (vulnerabilita c.a.) e VM (vulnarability muratura). INGV/GNDT – L'Aquila, 2005.

[10] EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. European Committee for standardization CEN, 2004.

[11] HRN EN 1998-3 Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofiting of buildings. Croatian Standards Institute, 2011.

[12] TREMURI software, S.T.A.DATA, Professional version, Torino, 2019.

[13] Bohm G, Da Col F, Accaino F, Meneghini F, Schleifer A, Nikolić, Ž. Characterization of shallow sediments in an urban area (Kaštela, Croatia) by analysis of P, SV and Sh seismic velocities using a tomographic approach, Near Surface Geoscience Conference & Exhibition 2020, 30 August - 3 September 2020, Belgrade, Serbia.

[14] Da Col F, Accaino F, Bohm G, Meneghini F. Characterization of shallow sediments by processing of P, SH and SV wavefields in Kaštela (HR). Engineering Geology 2021, 293, 106336.

[15] Uranjek M, Žarnić R, Bokan-Bosiljkov V, Bosiljkov V. Seismic resistance of stone masonry building and effect of grouting. Građevinar 2014, 66, 715–726.

[16] Bosiljkov V, Kržan M. Results of laboratory and in-situ tests on masonry properties and tables with mechanical parameters to be adopted in numerical modelling, PERPETUATE (ECFP7 project), Deliverable D15 (www.perpetuate.eu), 2012, 225.

[17] Nikolić Ž, Runjić L, Ostojić Škomrlj N, Benvenuti E. Seismic Vulnerability Assessment of Historical Masonry Buildings in Croatian Coastal Area. Applied Sciences, 2021, 11(13), 5997.