SEISMIC BEHAVIOUR AND VULNERABILITY OF
EXISTING BUILDINGS IN PRISHTINA – KOSOVA

Doctoral dissertation

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Florim Grajçevci
Abstract

Although natural phenomena can not be prevented, their effects can significantly be reduced if improved construction standards, more sophisticated land use policy and better vulnerability source identification of the main elements at risk or mitigation of consequences and their reconstruction.

The importance of assessing the seismic resistance of existing masonry structures has drawn strong and growing interest in the recent years.

The territory of Kosova is actually included in one of the most seismic-prone regions in Europe. Therefore, the earthquakes are not so rare in Kosova; and, when they occurred, the consequences have been rather destructive.

Engineering included those of Vulnerability, Loss of Buildings and Risk assessment, are also of a particular interest. This is due to the fact that this rapidly developing field is related to great impact of earthquakes on the socio-economic life in seismic-prone areas, as Kosova and Prishtina are, too. Such studies for Prishtina city may serve as a real basis for possible interventions in existing buildings, in order to adequately strengthen and/or repair them, by reducing the seismic risk within acceptable limits.

In order to have a clear picture of the integral work computed and presented in this PhD thesis, the complete study is divided and presented in two parts. I Part, presented is theoretical background of the implemented method and brief review of existing concepts for structural vulnerability analysis. In separate part presented is integral and consistent methodology implemented for seismic vulnerability analysis of selected representative masonry buildings. Part - I consists of four chapters. In Chapter 1, “state of art” presented are the existing vulnerability assessment methods loss estimation procedures in order to provide the reader with an overview of the state of the art. This overview cannot be fully complete listing. However, it is tried to present some of commonly used different methodological approaches.

General concept implemented for seismic risk assessment based on the developed theoretical building vulnerability functions, is given in Chapter 2. This chapter shows evaluation procedures and calculations of expected Vulnerability and seismic risk.
Considering the specific topic of present PhD studies, focused to Vulnerability of masonry buildings under earthquake impacts, the adopted methodology for vulnerability and loss calculations for this specific type of structures is given in Chapter 3. Analysis of building vulnerability is performed with application of sophisticated INERA method (Inelastic Earthquake Response Analysis). In this chapter presented is theoretical part used for Nonlinear Dynamic Response Analysis of building structures, under specified earthquake ground motion. This analysis provide important data to define Damage level of Structural and Nonstructural Elements Based on previously defined Load Bearing and Deformability Capacity curves, For masonry buildings, damage propagation is evaluated in five different categories (DG-1 to DG-5), in order to be able to implemented and define the vulnerability and loss functions. Computation of non-linear seismic response analysis for all selected buildings was done using special purpose computer NORA 2000 and also developed computer program ANOLOS 2000 providing computation of representative seismic vulnerability function based on computed results, computing program NORA..

Chapter 4 of Part-I, presented is theoretical background for analysis of non-linear seismic response of buildings with non-linear structural and non-structural elements.

In order to solve computed dynamic non-linear mathematical problem such is dynamic non-linear response analysis of masonry structure, used are effective step-by-step integration methods, based on Wilson – (Θ) Theta method and Newmark – (β)Betha method. For each formulated non-linear dynamic model first are calculated mode shapes and frequencies, based on specific software module named EIGEN problem solution.

In the final part of this chapter are presented also Analysis Option and Flow-Chart of the Developed Computer Program NORA 2000 for Nonlinear Earthquake Response Analysis of Specific Structures Based on Proposed micro (Stress-Strain), macro and global Modeling.

In the Second part of the presented dissertation (Part-II) presented are the obtained original results of the conducted comparatively vulnerability study of the selected representative masonry buildings in the city of Prishtina and is divided in five separate chapters. Within chosen 55 masonry buildings, we have identified 15 buildings to be comparatively analyzed. In Chapter 1 of Part-II given is brief description of a set of 55 selected masonry buildings, criteria for selection of 15 most typical buildings for further analysis. Basic criteria for
selection of 15 buildings which are further analyzed include consideration number of stories, building base shape, usability of the buildings, shape of the structure, and are described in detail in this chapter.

Site inspections, measurements and data collection were required for all buildings selected for analysis, in order to define all geometric characteristics, type of quality of used materials in structures (material of load bearing walls, types of mezzanine structures, roof structure), structural system and all other characteristics for each building, in particular data on overbuilding, presented wall openings etc.

Having in mind that large part of the present study include realization of extensive non-linear dynamic (seismic) analysis of buildings, we need to define the representative seismic ground motions to be used, in Chapter 2 of Part-II given is description of specific seismic ground parameters for analyzed building locations, including Kosovo seismic maps derived based on historical seismic information, as well as seismic izoline map of Kosovo. For the purpose of providing better results in seismic vulnerability of selected buildings, as seismic input motion selected are three typical earthquake record: (1), Ulcinj-Albatros, (2), El-Centro and (3), Pristina Synthetic (artificial) earthquake record. In order to implement dynamic analysis for increasing earthquake intensity adopted are 11 different Peak Ground Acceleration levels (PGA) from 0.025g to 0.50g.

The computed very large number of 990 non-linear seismic response analysis results for all 15 analyzed buildings are systematically evaluated and presented in Chapter 3 of Part-II. Description of basic characteristics of structural system is given for each of 15 analyzed buildings. This type of data gathered from the field inspection, is presented through photos and originally for this study created basic plans.

For each building, all needed non-linear seismic response analysis where performed separately and consistently for longitudinal direction x and transversal direction y. Also, under the impact of each earthquake type along direction x we have performed 11 non-linear analysis. For each building, under the impact of three earthquakes and along both orthogonal directions, we have performed 66 non-linear analysis. In total, for 15 analyzed buildings, we have performed 990 non-linear seismic response analysis. From the calculated results for each building and along each orthogonal direction x and y, presented are only selected results in tabular and graphical form.
Firstly presented are the formulated Non-Linear Mathematical Models of the Building, for both orthogonal directions x and y. The formulated non-linear mathematical model for each building is defined as “shear type”, formulated based on systematic implementation of “multi componential” concept. Next, for each analyzed building are presented the first two mode shape and corresponding frequencies (periods) for each direction of the building.

For each building presented are in graphic form envelope curves showing building bearing capacity for each storey and for both orthogonal directions x and y. Based on the calculated results conducting maximal response forces and maximal displacements, developed are specific displacement spectral diagrams that show maximum response of the buildings under different intensity level earthquake impact. Basic original relations established between the increasing input earthquake intensity parameter (PGA) and the resulting inter-story drifts (ISD), based on calculated data for all stories and all three earthquake motion types are presented in separate tables.

For each building derived is the analytical vulnerability function of the integral Building in x and y direction, expressing the total losses in percent of the total building cost for increasing the PGA level. The final results from this analysis are obtained throughout completion of several subsequent systematic steps. Through the adapted ratio, defined are loss functions for structural and non-structural elements.

In order to have a clear evidence on the damage propagation, each building, presented are in specific original tabular form damage propagation pattern using appropriate color code. Each color present different level of damage for both SE and NE. Level DG1 – Non-Damaged Elements, are presented as pattern, DG2 – Cracked Elements, are presented with yellow color, DG3 – Light Damaged Elements, are presented with green color, DG4 – Heavy Damaged Elements, are presented with blue color and DG5 – Collapsed Elements, are presented with red color.

Considering the implemented selection criteria, the analyzed buildings, are classified in categories and calculated results were analyzed and compared within each category. Chapter 4 shows analysis of obtained final results for defined four damage propagation levels in buildings. Seismic Vulnerability of analyzed Masonry Buildings is analyzed and described for derived different building classes according Number of Stories, building Usability, Quality of Construction and, building classes according to General Floor shape in base.
This final results of obtained effect of earthquake impact different earthquake intensity (acceleration values) show variations for different classes and this evidence is very important for different representative conclusions and recommendations regarding construction and reconstruction of existing masonry buildings. Therefore this analysis is shown in Chapter 5 of Part-II for the following seismic input acceleration values: PGA = 0,025g, 0,10g, 0,15g and 0,25g.

In the final chapter 6, Part-II, presented are the derived conclusions and recommendations important for mitigation of seismic risk and vulnerability of existing and new buildings constructed as masonry structures with masonry load bearing and non-structural walls.
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PART-1

Chapter 1

THEORETICAL BACKGROUND OF THE APPLIED AND BRIEF REVIEW OF EXISTING CONCEPTS FOR STRUCTURAL VULNERABILITY STUDY AND CONSISTENT METHODOLOGY FOR SEISMIC VULNERABILITY ANALYSIS OF REPRESENTATIVE MASONRY BUILDINGS IN THE CITY OF PRISTINA.

1. STATE OF THE ART

1.1. Introduction

A big research effort has been made on the prediction of earthquakes in the last decades, and in fact the exploration of the new techniques aiming to foresee the occurrence of seismic events is in a continuous progress. However, the possibility of knowing in advance the occurrence of major earthquake is still far from being a reality, and the preparedness to face an eventual emergency has to be made from the point of view of the prevention rather than the prediction.

In our minds, there are strong memories of the stirring pictures of the latest earthquakes that have occurred around the world. The most recent event was the earthquake in Turkey with over 18 000 victims, many more injured and thousands of heavily damaged and collapsed buildings. Similar devastating effects have been also observed in the latest earthquakes that struck many other countries like Greece, Taiwan, Kobe in Japan, China, United States, Russia, Italy, Montenegro, Algeria, Mexico, Peru, etc.

The losses caused by the earthquake that took place in the city of Kobe – Japan in 1995 is estimated at more than 100 milliard dollars and all this happened within only 20 – 30 seconds. More than 5500 people lost their lives and even more were injured.

Several analytical tools have been developed around the world in order to estimate, with different degrees of accuracy, the vulnerability of buildings and the probable loss of lives and economic resources, due to the occurrence of an earthquake [Br. 94]. Those tools are intended to be used by government agencies, and even by insurance companies, as a mean for planning
of emergency preparedness procedures and response strategies, and also for the reconstruction phases. Nonetheless, most of the current available tools require a large amount of resources, in terms of money, time and computational effort, in order to be properly implemented and effectively used.

Additionally, large portions of buildings in non-developed countries, especially old constructions, are built with unreinforced masonry, under several forms. In most modern design codes the use of unreinforced masonry has been banned for the construction of new buildings in moderate to high seismic regions [Ca. 00].

From the arguments exposed in the previous paragraphs, it is possible to say that the development of a simplified methodology for the seismic vulnerability, or better seismic risk assessment of unreinforced masonry buildings [Ka.No. 08], would have a considerable relevance, especially taking into account the social benefits that some new developments in this field will bring to poor communities in developing countries, providing useful tools for governments and other agencies.

The goals of this study are to improve the assessment of seismic hazard, to investigate the vulnerability of the built environment and, finally, to combine the results to elaborate risk scenarios as the first fundamental step in the mitigation process:

\[ \text{Risk} = \text{Hazard} \times \text{Vulnerability} \times \text{Exposure} \]  

(1.1)

The simplified methodology presented in this dissertation includes, apart from the considerations for the in-plane failure mechanism, a formulation for the out-of-plane behavior [Go 03]. Out-of-plane collapse mechanism has received less research compared to the in-plane behavior, and the aspects related to the dynamic behavior and vulnerability assessment still a lot of space for new developments [Gi. 02].

1.2. Importance of Earthquake Loss Estimation

The single most effective tool in reducing earthquake risk is a sound seismic design code, rigorously and effectively enforced at both the design and construction stages. The provisions of the code, in terms of specified design levels of earthquake shaking and performance criteria
(expressed as stresses and displacements) for building to meet under the expected ground motions, can ensure that the majority of structures built after the publication of the code will not collapse during future earthquakes. The code may thus prevent loss and also limit disruption, injury, homelessness and the economic impact of the next earthquake.

Although the application of a good seismic design code may effectively increase the earthquake resistance of a new building conforming to the code requirements, the impact of a new seismic code on the risk in an urban area in a seismically active region may initially be very low [Ke 02].

Since seismic design are generally not applicable retrospectively (i.e. their provisions only apply to new construction), it is clear that several decades may need to pass before a new seismic code makes a very significant impact on the level of risk in a major town or city. Paradoxically, the process can be accelerated by a strong earthquake removing a large part of the most vulnerable building stock, which could then be replaced by new structures confirming to a new or improved seismic design code.

1.3. Earthquake Loss Estimation Methodologies

The features of an earthquake loss estimation model depend on its purpose: for emergency planning a single scenario, possibly the repetition of an historical earthquake, is usually used, whereas for the calculation of annual losses, it is necessary to consider all feasible earthquake scenarios and to rank the resultant losses according to their probability of occurrence [McGuire, 2001]. A complete earthquake loss model must include all of all of the elements of hazard including tsunami, fault rupture, liquefaction and landslides, but the predominant cause of damage worldwide is strong ground shaking, and it is only this that will be the focus of the current work [Lu 03].

Traditionally, earthquake loss estimation studies have employed macroseismic intensity scales, such as the Modified Mercalli (MM) or European Macroseismic (EMS) scales, to represent the ground shaking. Intensity is an obvious choice because of its direct relation with damage in different classes of building [Musson, 2000] but its utility is diminished because prediction of intensity values for future earthquake, especially when taking account of soil
amplification effects requires the treatment of these discrete index values as continuous variables [Mi. 93].

These limitations are overcome by the use of instrumental parameters of the ground motion, such as peak ground acceleration, on PGA, which has been widely used as the basis for loss estimation studies [e.g. King et al., 1997]. Nonetheless, it is widely recognized that PGA has a very poor correlation with structural damage during earthquakes. Peak ground velocity, PGV, which is related to the energy in the ground motion, provides a better indicator of the damage potential than PGA and has been used as the basis for some earthquake loss functions [e.g. Miyakoshi et al., Yamazaki & Murao, 2000], [Ma 06].

However, single parameters such as PGA and PGV do not reflect the frequency content of the ground motion and hence their use does not take account of the influence of the natural and effective period of vibration of buildings in determining the level of loading that they experience during earthquake [B.C.S.V 06]. This can only be represented by more complete descriptions of the ground motion such as response spectra. Some loss estimation methodologies have made use of acceleration response spectra [e.g. Scawthorn et al., 1981; Shinozuka et al., 1997] but generally, in common with the use trends in seismic design mentioned next item, the state-of-the-art is now to in some way use the displacement response spectrum to represent the destructive capacity of the ground motion [e.g. Calvi, 1999; Faccioli et al., 1999: HAZUS, 1999].

Table 1.1: Methods for the assessment of the vulnerability of buildings

<table>
<thead>
<tr>
<th>expenditure</th>
<th>increasing computation effort</th>
</tr>
</thead>
<tbody>
<tr>
<td>application</td>
<td>building stock</td>
</tr>
<tr>
<td>methods</td>
<td>observed vulnerability</td>
</tr>
</tbody>
</table>

1.4. Observed Vulnerability

Observed vulnerability refers to assessments based on statistics of past earthquake damage. It is especially suitable for non-engineered structures made of low-strength materials such as timber and unreinforced masonry whose earthquake resistance is rather difficult to calculate.
Several methods for vulnerability assessment have been developed in recent years, considering different approaches for in the input data and for the output [Ba 95]. Corsanego and Petrini [1990] proposed the following classification for the methods, according to the type of building [Mi.Tr. 03]:

- **Direct**: these methods are subdivided in two groups. The typological group based on data gathered after the occurrence of real earthquakes that are statistically manipulated to obtain damage probability matrices for a limited number of classes of buildings. The mechanical group in based on numerical models and the output is usually the level of damage for a given building of a given class.

- **Indirect**: these techniques use both the data obtained after the occurrence of real earthquakes and the data obtained through numerical models.

- **Conventional**: these correspond to methodologies based on judgments given by experts. The output is a vulnerability index for a given class of building but this vulnerability is not directly correlated with any specific level of damage.

A different classification was proposed by Dolce [1994], based on the input, the methodology and the output, considering the options shown in Table 1.2.

<table>
<thead>
<tr>
<th>Type of Input</th>
<th>Method Type</th>
<th>Type of Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Damage data</td>
<td>1. Statistical methods</td>
<td>1. Absolute vulnerability</td>
</tr>
<tr>
<td>2. Geometric and qualitative features</td>
<td>2. Mechanical methods</td>
<td>2. Relative vulnerability</td>
</tr>
<tr>
<td>4. Seismic demand features</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Geological and geotechnical data</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The selection of the adequate methodology must be based on the following criteria.

- Purpose and regional scale of the damage scenario.
- Accuracy of the earthquake scenario used for the damage scenario.
- Available resources in terms of time and funds to prepare the building catalogue.
- Quantity and accuracy required in the information according to the different types of feasible methodologies.
- Existing information, either from government bodies, databases, libraries, etc.

From the available literature and considering the vast collection of methodologies, the following six procedures are explained in more detail, taking into that they haven been used
or proposed to be used for the assessment of unreinforced masonry buildings [No.Ri. 91] [P.S.N.R 2.97].

1.5. Damage Probability Matrix

The damage probability matrix method or $DPM$ is based on the idea that a set of buildings having a common structural typology would have the same behavior under the action of an earthquake, and as a consequence, the level of damage would be the same for the set of building. The damage is characterized by some level of uncertainty described by a damage probability matrix [Whitman, 1973; Di Pasquale et al., 2001]. Each element of the matrix is expressed according to Eq.(2.1):

$$DPM(DV,I,T) = P(DV/I,T)$$  \hspace{1cm} (1.2)

Where $DV$ corresponds to a given level of damage, $T$ is an specific structural typology and $I$ is the earthquake intensity, normally described by some macroseismic scale, for example the European Macroseismic Scale $EMS$, given in Table 1.3.

<table>
<thead>
<tr>
<th>Scale</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Not felt</td>
</tr>
<tr>
<td>II</td>
<td>Scarcely felt</td>
</tr>
<tr>
<td>III</td>
<td>Weak</td>
</tr>
<tr>
<td>IV</td>
<td>Largely observed</td>
</tr>
<tr>
<td>V</td>
<td>Strong</td>
</tr>
<tr>
<td>VI</td>
<td>Slightly damaging</td>
</tr>
<tr>
<td>VII</td>
<td>Damaging</td>
</tr>
<tr>
<td>VIII</td>
<td>Heavily damaging</td>
</tr>
<tr>
<td>IX</td>
<td>Destructive</td>
</tr>
<tr>
<td>X</td>
<td>Very destructive</td>
</tr>
<tr>
<td>XI</td>
<td>Devastating</td>
</tr>
<tr>
<td>XII</td>
<td>Completely devastating</td>
</tr>
</tbody>
</table>

This methodology is considered as a direct method because it allows estimating the vulnerability in one single step, considering the building as a member within a specific class. The level or class of vulnerability according to the structural typology of masonry buildings is shown in Figure 1.1, according to the proposal given in the $EMS$-98, being A the highest vulnerability and F the lowest.
<table>
<thead>
<tr>
<th>Type of Structure</th>
<th>Vulnerability Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rubble stone, fieldstone</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Adobe (earth brick)</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Simple stone</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Massive stone</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Unreinforced, with manufactured stone units</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Unreinforced, with RC floors</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
<tr>
<td>Reinforced or confined</td>
<td>◌ ◌ ◌ ◌ ◌ ◌ ◌</td>
</tr>
</tbody>
</table>

![Figure 1.1. Differentiation of structures (buildings) into vulnerability classes [Gruntal, 1998]](image)

- ◌ Most likely vulnerability class;
- --- Probable range;
- ----- Rang of less probable, exceptional cases

According to Eq.(1.2), the DPM would be a matrix having the probability of reaching some level of damage for a given earthquake intensity.

One of the main disadvantages of the DPM method is the use of a discrete measure of the earthquake intensity through the use of a macroseismic intensity scale, rather than using a different definition of intensity, for example a continuous parameter like acceleration or displacement, with a better correlation with the level of damage. A key issue here is that the input is described by the effects of the ground motion on structures, rather than other parameters measuring directly the input ground motion, which is somehow like using the responses to compute the response. Another disadvantage is that the DPM does not consider the uncertainty on the demand, which are the components of uncertainty that are required to be included in a complete vulnerability assessment, as discussed in next session. Finally, the DPM method does not allow the estimation of the vulnerability for a single building, but just the evaluation as a part of a class of buildings; thus, it is not possible to individuate all the features of each specific building.
1.6. Vulnerability functions based on expert opinions

One of the first systematic attempts to codify the seismic vulnerability of buildings came from the Applied Technology Council (a non-profit corporation established in 1971 for the assistance of the practicing structural engineer to keep abreast of technological developments) summarized in a report [ATC 13] which was funded by the Federal Emergency Management Agency (FEMA). ATC-13 essentially derived damage probability matrices for 78 different earthquake engineering facility classes, 40 of which refer to buildings, by asking 58 experts (noted structural engineers, builders, etc.) to estimate the expected percentage of damage that would result to a specific structural type subjected to a given intensity [Po.So 98].

A second major attempt to develop a methodology for vulnerability assessment was undertaken by the National Institute of Building Science (NIBS), funded again by FEMA. HAZUS is an acronym for “HAZard in the U.S.” and corresponds to a methodology developed by the Federal Emergency Management Agency [Whitman et al., 1997; FEMA, 1999]. The methodology is based on three fundamental concepts: capacity curve, design point and fragility curve.

1.6.1. HAZUS

The result was an interactive software for risk assessment, HAZUS®, released for the first time in 1997 and updated in 1999 [HAZUS 99][KNKH 97]. In HAZUS® intensities were replaced by spectral displacements and spectral accelerations as a measure of the seismic input. However, the HAZUS® study continues to rely on expert opinion to estimate the state of damage that would result from a given spectral displacement and acceleration.

Capacity curve is the relationship between the lateral load resistance of given structure and its characteristic lateral displacement, and is typically obtained by means of a static pushover analysis. The capacity curve is then converted to spectral acceleration and roof displacement in order to be compared with the demand spectrum. Figure 1.2. has shown an example of the capacity curve and the demand spectrum for a given building. It can be seen that the capacity curve is controlled by the yield capacity, or restoring force, and the ultimate capacity, being possible to represent with this single curve the corresponding strength at a certain
displacement limit state, which is turns can be directly correlated with some damage limit state. The damage limit states considered by *HAZUS* are: slight, moderate, extensive and complete; their description can be found in Kircher *et. Al.* [1997].

For a given buildings class, defined by the structural system, the building type and occupancy class, a specific capacity curve is defined, for which the design point \( (S_a, S_d) \) is obtained. The design point corresponds to the intersection of capacity and demand curves.

The fragility curve concept represents the function \( CDF \) for the probability of reaching or exceeding a specific damage limit state for a given peak response to a given ground motion demand. In *HAZUS* the fragility curves are assumed to be represented by lognormal functions, hence they can be fully described by the median and the standard deviation, according to Eq.(1.3):

\[
P[ds / S_d] = \Phi \left[ \frac{1}{\beta_{ds}} \ln \left( \frac{S_d}{\bar{S}_{d,ds}} \right) \right]
\]

Where:
- \( S_d \) is the spectral displacement (seismic hazard parameter),
- \( \bar{S}_{d,ds} \) is the median spectral displacement for which the building reaches the damage limit state \( ds \),
- \( \beta_{ds} \) is the standard deviation of the natural logarithm of spectral displacement for the damage limit state, \( ds \); and,
\( \Phi \) is the standard normal cumulative density function.

The original FEMA/NIBS approach proposes that the median values of structural fragility are based on building drift ratios that describe the threshold of damage states. Damage-state drift ratios are converted to spectral displacement by using the following equation:

\[
\overline{S}_{d,ds} = \delta_{R,Sds} \alpha_2 h
\]  

(1.3a)

where:
\( \delta_{R,Sds} \) is the drift ratio at the threshold of structural damage state, \( ds \)
\( \alpha_2 \) is the fraction of the building (roof) height at the location of pushover model displacement
\( h \) is the typical height of the model building type of interest.

The variability of a given damage state \( \beta_{ds} \) is obtained with Eq.(1.4), where \( \beta_D \) is the variability of the ground – motion demand, \( \beta_C \) is the variability of the capacity response and \( \beta_{T,ds} \) is the variability of the damage state threshold.

\[
\beta_{ds} = \sqrt{(CONV[\beta_C,\beta_D])^2 + (\beta_{T,ds})^2}
\]  

(1.4)

In Eq.(1.4) \( CONV \) represent the convolution process applied to the variability of the ground – motion demand and the capacity response that it necessary to carry out, taken into account that demand spectrum is dependent on building capacity. The convolution process is explained in Kircher et al. [1997]. Nonetheless, when the variability of the ground – motion demand has already been incorporated into the seismic hazard assessment, which is the normal practice, the variability of a given damage state must be computed with Eq.(1.5) in order to avoid a double counting of \( \beta_D \).

\[
\beta_{ds} = \sqrt{(\beta_C)^2 + (\beta_{T,ds})^2}
\]  

(1.5)

In HAZUS, typical median values \( \overline{S}_{d,ds} \) of spectral displacement and standard deviation values \( \beta_{ds} \) of the natural logarithm of spectral displacement are given for 36 different classes
of buildings, based on experimental tests, experience and judgment. Those parameters are strictly valid for buildings within the United States. Figure 1.3 shows an example of fragility curves for slight, moderate, extensive and complete damage states obtained as explained above.

Once the design point and the fragility curve for a specific class of building have been computed, the probability of reaching or exceeding a specific damage limit state is obtained. The process of building damage estimation is schematically shown in Figure 1.3. It is worth noting that in HAZUS fragility curves for non-structural drift sensitive components and non-structural sensitive components are also provided.

Figure 1.3. Example of fragility curves for slight, moderate, extensive and complete damage states [FEMA, 1999], for a specific class of buildings
An important advantage of HAZUS is its ability to estimate the damage under a given earthquake scenario, considering damages not just in buildings but also in lifelines, transportation systems, utility system, essential and high potential loss facilities. In fact, with HAZUS is possible to compute the damage due to earthquake hazard, inundation, fire and hazardous materials.

The main disadvantage stems from its complexity. HAZUS is to complete and powerful that is takes a lot of resources to be implemented in a real application for a small to medium community. In HAZUS’s users manual [FEMA, 1999] it is stated that the methodology allows for different levels of funding which means different levels of inventory collection. However, when low levels of funding are available, the information that is missing has to be assumed as
similar to the one provided in the HAZUS database, which has been validated to be used in the United States.

1.6.2. Score assignment

Score assignment procedures aim to identify seismically hazardous buildings by exposing structural deficiencies. They often form the first phase of a multi-phase procedure for identifying hazardous buildings which then must be analyzed in more detail in order to decide on upgrading strategies. Potential structural deficiencies are identified from observed correlations between damage and structural characteristics. The scores for different deficiencies are usually calibrated by experts [Ba.Da.Bu].

Again, it was the Applied Technology Council that developed a first comprehensive methodology for the evaluation of existing buildings in order to identify those buildings which present a risk to human lives [ATC 14]. The life-safety hazard in a building consists of the failure of any structural element of the building. The methodology therefore aims at identifying flaws and weaknesses which could cause structural failure.

A method for vulnerability assessment and damage estimation for earthquake scenarios based on score assignments was also developed and applied successfully in Italy (so called GNDT method). Based on visual observations to identify the primary structural system of the buildings and significant seismic related deficiencies collected through field surveys, a vulnerability index is assigned to each building.

1.6.3. GNDT and II Level Approaches

The Gruppo Nazionale per la Difesa dai Terremoti – BNDT is the Italian government research body in charge of the seismic risk evaluation and definition of the required measures to reduce it. Two, somehow complementary, approaches have been published by this group, with the aim of being applied in the assessment of the seismic risk in the Italian territory.

The GNDT level approach is nothing more than a DPM method, having three classes of vulnerability, from A to C, each of three having a DPM. For this approach the demand is considered through the use of the EMS-98 intensity scale and the damage is described by means
of a qualitative description, according to the level of damaged reached by the building. Further description of this methodology can be found elsewhere [e.g. GNDT, 1993].

The GNDT II level approach is based on a survey from designed to gather information regarding the typology and constructive features for each single building, that are combined afterwards to get a vulnerability index $I_V$. Eleven parameters are combined with different scores and relative weights, depending on four classes of vulnerability, as shown in Table 2.3. $I_V$ is an absolute measure from 0 to 382.5 but eventually can be normalized from 0 to 100, being 0 the best vulnerability condition and 100 the worst.

Table 1.4. Scores and relative weights to compute $I_V$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Class</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Type and layout of resistant system</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Quality of resistant system</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Conventional resistant</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Location of building and foundation</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Horizontal elements (floor, diaphragm)</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Configuration in plan</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Configuration in height</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Max. distance between walls</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Roof</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>Non-structural elements</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>State of conservation</td>
<td>0</td>
<td>5</td>
</tr>
</tbody>
</table>

For each vulnerability index there is a corresponding curve correlating the damage ratio and the demand represented by the PGA, by means of a tri-linear curve resembling somehow the so called “fragility curves”, which are better explained before in Section 1.6. The damage is computed in terms of economical loss, correlated as a function of the peak ground acceleration PGA [Giovinazzi and Lagomarisiino, 2001]. Figure 1.5 shows the acceleration versus damage ratio tri-linear curves for masonry buildings proposed in the GNDT II level approach.
Some important drawbacks of the I and II level approaches in that their intervals of confidence are very large, both for the arbitrary way used to defined the points and weight for the vulnerability index, and lack of correlation between PGA and the level of damage of a given building [Giovinazzi and Lagomarsion, 2001].

1.6.4 VULNUS

The VULNUS procedure was developed in the second half of the ‘80s at the University of Padova, with the purpose of evaluating the seismic vulnerability of a single building or group of buildings [Bernardini, Gori and Modena, 1990]. The methodology is based on the evaluation of the geometrical and mechanical characteristics of each building, which is combined with the evaluation of some other important factors controlling the response of the structure, that are handled through qualitative judgments. The whole procedure is developed under the fuzzy set theory that is used for the definition of the safety criterion. This method could be considered inside the mechanical group because it makes use the so called collapse multipliers.

The geometrical and mechanical characteristics are described with two indicates or multipliers, according to Eqs (1.6) and (1.7)
\[ I_1 = \min \left( \frac{V_x, V_y}{W} \right) \]
\[ I_2 = \min \left( \frac{I'_2 + I''_2}{I_2} \right) \]

\[ I_1 \] is the collapse multiplier for in-plane behavior considering shear failure at ground floor, being \( W \) the total weight of the building and \( V_x \) and \( V_y \) the strength at mid-storey height of the ground floor, according to Eq. (1.8):
\[ \{V_x, V_y\} = \{F_x, F_y\} \cdot \frac{f_t}{1.5\omega \left[ 1 + \frac{W}{f'(F_x + F_y)} \right]^{1/2}} \]

Where \( \{F_x, F_y\} \) are the total areas of the walls in the \( x \) and \( y \) direction respectively, \( f_t \) is the tensile strength of masonry [Ministerio dei Lavori Pubblici, 1981] and \( \omega \) is a factor to include the effects of plan regularity. In this expression it has been assumed that the walls are rigidly jointed to the slabs and are subjected to uniform vertical compression.

\( I_2 \) is the collapse multiplier for the out-of-plane behavior, considering each wall and several failure modes, namely: overturning, flexural tension, arch crushing, shoulders overturning and tension. A detailed description of the computation procedure for this collapse multiplier can be found in Bernardini, Gori and Modena [1990].

Once the in-plane \( I_1 \) and out-of-plane \( I_2 \) indices have been computed, a safety criterion is chosen to estimate the vulnerability of a given building. The definition of the safety criterion is shown in Figure 1.6, where \( c_1 = 0.5, c_2 = 1.0, c_3 = 1.0 \) are the values typically assumed for buildings in Italy. The parameter \( u \) is defined with Eq. (1.9), being \( A \) the maximum base shear divided by the total weight of the building \( W \).

\[ u = \frac{c_3 + c_1 - c_2 + \left[ \left( \frac{I_1}{A} - c_1 \right) \left( \frac{I_2}{A} - c_2 \right) \right]^{1/2}}{2c_3 + ac_4} \]

The parameter \( a \) depends on the qualitative judgments that are performed taken into account large databases gathered in Italy by means of several surveys performed in the past. An important feature of this parameter is that it can be updated after the continuous improving of the database [Bernardini, 2000]. The qualitative judgments are expressed as a combination of
seven vulnerability factors $S_i$ and their corresponding weights $W_i$, and is evaluated with Eq.(1.10)

\[ I_3 = \sum_i \frac{W_i S_i}{45x3.15} \]

Tables 1.5 and 1.6 shows the proposed values for the size and the weight of each vulnerability factor, respectively.

**Table 1.5. Classification and corresponding values of the vulnerability factors [Bernardini, Gori and Modena, 1990]**

<table>
<thead>
<tr>
<th>Class</th>
<th>Size $S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Good or corresponding to code</td>
<td>0</td>
</tr>
<tr>
<td>2 Almost good</td>
<td>15</td>
</tr>
<tr>
<td>3 Almost poor</td>
<td>30</td>
</tr>
<tr>
<td>4 Poor or unsafe</td>
<td>45</td>
</tr>
</tbody>
</table>

**Table 1.6. Vulnerability factors related to qualitative judgment and their Corresponding weights [Bernardini, Gori and Modena, 1990]**

<table>
<thead>
<tr>
<th>Vulnerability factors</th>
<th>Weight $W$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Walls system quality</td>
<td>0.15</td>
</tr>
<tr>
<td>2 Soil and foundations interaction</td>
<td>0.75</td>
</tr>
<tr>
<td>3 Floors interaction</td>
<td>0.50</td>
</tr>
<tr>
<td>4 Elevations regularity</td>
<td>0.50</td>
</tr>
<tr>
<td>5 Roof interaction</td>
<td>0.50</td>
</tr>
<tr>
<td>6 Interaction of non-structural elements</td>
<td>0.25</td>
</tr>
<tr>
<td>7 General maintenance conditions</td>
<td>0.50</td>
</tr>
<tr>
<td>Total</td>
<td><strong>3.15</strong></td>
</tr>
</tbody>
</table>
Once $I_3$ has been obtained, a linguistic relationship between $a$ and $I_3$ is established, according to Table 1.7.

*Table 1.7. Linguistic relationship between $a$ and $I_3$ [Bernardini, Gori and Modena, 1990]*

<table>
<thead>
<tr>
<th>$J$</th>
<th>Condition</th>
<th>Linguistic Relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>If $I_3$ is very large</td>
<td>$a$ is very large</td>
</tr>
<tr>
<td>2</td>
<td>If $I_3$ is large</td>
<td>$a$ is very large</td>
</tr>
<tr>
<td>3</td>
<td>If $I_3$ is medium</td>
<td>$a$ is very medium</td>
</tr>
<tr>
<td>4</td>
<td>If $I_3$ is small</td>
<td>$a$ is small</td>
</tr>
<tr>
<td>5</td>
<td>If $I_3$ is very small</td>
<td>$a$ is very small</td>
</tr>
</tbody>
</table>

After the three parameters $I_1$, $I_2$, and $I_3$ have been computed and the linguistic relationship has been established, fuzzy set theory is applied in order to compute the vulnerability value. Details of the corresponding fuzzy set theory are not reported here but can be found in Bernardini, Gori and Modena [1990].

The main advantages of this method are that it allows to compute an absolute value for the vulnerability with respect to the intensity of the given ground motion, and allows to classify the surveyed buildings in an orderly way with respect to the vulnerability measure.

### 1.7. Detailed analysis procedures

Already the methods for the assessment of the vulnerability of buildings based on score assignments are rather detailed and therefore time-consuming. More sophisticated methods, implying a more detailed analysis and more refined models, take even more time and serve therefore for the evaluation of individual buildings only, possibly as a further step after the rapid screening of potential hazardous buildings in a multi-phase procedure [ATC 96]. They are not suitable for earthquake scenario projects where a large number of buildings have to be evaluated. Nevertheless, the concepts behind those methods can be valuable for the development of new simple methods and hence, the main analysis procedures shall be briefly outlined [Ab 97].

The analysis procedures can be divided into linear procedures (linear static and linear dynamic) and nonlinear procedures (nonlinear static and nonlinear dynamic).
1)  **Linear static procedures**

In a linear static procedure the building is modeled as an equivalent single-degree of-freedom (SDOF) system with a linear elastic stiffness and an equivalent viscous damping. The seismic input is modeled by an equivalent lateral force with the objective to produce the same stresses and strains as the earthquake it represents. Based on an estimation of the first fundamental frequency of the building using empirical relationships or Rayleigh’s method, the spectral acceleration is determined from the appropriate response spectrum which, multiplied by the mass of the building, results in the equivalent lateral force:

$$V = S_a \cdot m \cdot \sum_i C_i$$  \hspace{1cm} (1.11)

The coefficients take into account issues like second order effects, stiffness degradation, but also force reduction due to anticipated inelastic behavior. The lateral force is then distributed over the height of the building and the corresponding internal forces and displacements are determined using linear elastic analysis.

These linear static procedures are used primarily for design purposes and are incorporated in most codes. Their expenditure is rather small. However, their applicability is restricted to regular buildings for which the first mode of vibration is predominant.

2)  **Linear dynamic procedures**

In a linear dynamic procedure the building is modeled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modeled using either modal spectral analysis or timehistory analysis. Modal spectral analysis assumes that the dynamic response of a building can be found by considering the independent response of each natural mode of vibration using linear elastic response spectra. Only the modes contributing considerably to the response need to be considered. The modal responses are combined using schemes such as the square-root-sum-of-squares. Time-history analysis involves a time-step-by-time-step evaluation of building response, using recorded or synthetic earthquake records as base motion input. In both cases the corresponding internal forces and displacements are determined using again linear elastic analysis.
The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered which makes them suitable for irregular buildings. However, again they are based on linear elastic response and hence their applicability decreases with increasing nonlinear behavior which is approximated by global force reduction factors.

3) **Nonlinear static procedures**

In a nonlinear static procedure the building model incorporates directly the nonlinear force-deformation characteristics of individual components and elements due to inelastic material response. Several methods exist (e.g. [ATC 40] [FEMA 273]). They all have in common that the nonlinear force-deformation characteristic of the building is represented by a pushover curve, i.e. a curve of base shear vs. top displacement, obtained by subjecting the building model to monotonically increasing lateral forces or increasing displacements, distributed over the height of the building in correspondence to the first mode of vibration, until the building collapses (cf. Section 3.4). The maximum displacements likely to be experienced during a given earthquake are determined using either highly damped or inelastic response spectra. Clearly, the advantage of these procedures with respect to the linear procedures is that they take into account directly the effects of nonlinear material response and hence, the calculated internal forces and deformations will be more reasonable approximations of those expected during an earthquake. However, only the first mode of vibration is considered and hence these methods are not suitable for irregular buildings for which higher modes become important.

4) **Nonlinear dynamic procedures**

In a nonlinear dynamic procedure the building model is similar to the one used in nonlinear static procedures incorporating directly the inelastic material response using in general finite elements. The main difference is that the seismic input is modeled using a time-history analysis which involves time-step-by-time-step evaluation of the building response. This is the most sophisticated analysis procedure for predicting forces and displacements under seismic input. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; therefore several time-history analyses are required using different ground motion records. The main value of nonlinear dynamic procedures is as a research tool with the objective to simulate the behavior
of a building structure in detail, i.e. to describe the exact displacement profiles, the propagation of cracks, the distribution of vertical and shear stresses, the shape of the hysteretic curves, etc.

1.8. General Remarks

The purpose of the revision report in this chapter was to identify possible features that are be considered in the new procedure, and problems that be either solved or avoided in order to have a sounder methodology [Be.Le 07].

The features that have been identified to be ideally included in the new procedure are:

- The use of displacement or drift as an indicator of the demand level, which has a better correlation with the level of damage.
- To consider uncertainly on the demand, recognized to be one of the components of uncertainty that are required to be included in a complete seismic risk assessment.
- The use of a mechanics – based approach for the definition of the structure response. Or in the conversely, the problems identified to be overcome are:
- The use of a discrete measure of the earthquake intensity through the use of a macroseismic intensity scale.
- The uncertainties in the methodology originated in the arbitrary definition of factors and weights.
- Complexity in the application of the procedure in real cases.

1.9. A method to evaluate the Vulnerability of Existing Buildings

For the purpose of seismic risk assessment, an evaluation proposed method for the city of Prishtina, Kosova, to determine the seismic vulnerability of existing buildings [Be.Be 98]. In this chapter, the principle of the evaluation method is introduced in a general way, valid for masonry buildings [Si 02].

1.9.1. Positioning of the method

In This Chapter 1 currently available methods for the evaluation of existing buildings were introduced aging from very simplified and rather global loss estimation methods based on
observations and expert opinions, via simple analytical models and score assignments, to rather detailed analysis procedures describe in previous [Ra.Go.Je 04].

Global loss estimation methods based on observations and expert opinions have been used successfully in earthquake prone areas where they have a lot of experience with earthquakes and a statistical evaluation of observations is possible; however, the validity for cities in Switzerland and their building techniques is questionable. Score assignments are already rather time consuming and also require some experience from earthquakes in order to rate the structural deficiencies [Ta. 07]. The linear analysis procedures, although rather simple, are not considered suitable acknowledging the importance of the nonlinear displacement capacity for the seismic behavior of a building. The nonlinear dynamic analysis procedures, however, imply very high computational effort with a rather limited validity (a unique building subjected to a specific earthquake) and are therefore not very practical for earthquake scenarios where a large number of buildings have to be evaluated. Also, the link from the results of a nonlinear dynamic analysis to some statement on the loss is usually not made.

For the earthquake scenario project for the aim study, city of Prishtina, Kosova, it was therefore to use an analytical approach with simple models of the buildings based on the nonlinear static procedures. The method, which is presented in the following, is simple enough to allow the evaluation of a large number of buildings; still, the use of engineering models of the structure allow an understanding of the important parameters.

### 1.9.2. Difference between design and evaluation

The essential difference between the design of new buildings and the evaluation of existing buildings is the point of view. In design the objective is to create a new building which can resist the expected forces (horizontal and vertical) with an appropriate safety margin. Starting from a structural model of the building and the expected applied forces the required sections of the structural elements have to be determined for a chosen material. It is common practice to choose a slightly conservative model, i.e. to neglect the positive influence of some elements, firstly to simplify the model and secondly to be on the safe side. Also, the material strength is usually multiplied by a certain strength reduction factor, whereas the expected applied forces are enhanced to take into account uncertainties.
The choice of the strength reduction factors and the design forces are governed by the aim for economic optimization, however they are usually chosen to keep the risk of damage extremely low in building design this compares with an accepted annual probability for achieving the ultimate capacity of about 0.01%[PP 92]. In earthquake engineering a rational design becomes more important accepting a higher risk of damage. Here the annual probability for achieving the ultimate capacity can be as high as 1 to 3%. In evaluation the objective is to determine how an existing building will respond to given forces. This corresponds to an analysis of a building structure where the structural elements, the materials and the dead loads are given. It is not desired to calculate a worst case scenario by choosing a conservative model and making conservative assumptions on the material properties but to assess the most probable behavior of the building subjected to the applied action. Thus, the real material properties and the real loading have to be taken without any safety factors as these would falsify the results. Also the model should be as close as possible to reality taking into account all structural elements that help to support the applied forces.

It follows that the use of codes of practice for the evaluation of existing buildings is not always appropriate as these are usually too stringent in order to assure a safe design of a new building. This is especially true for unreinforced masonry buildings for which the code procedures tend to be very conservative (usually based on elastic mechanics of materials) due to a lack of understanding. In fact, non-compliance to most codes for unreinforced masonry buildings does not necessarily imply an inadequate seismic behavior; some unreinforced masonry buildings have performed excellently during major earthquakes [Br 94a].

The evaluation of existing buildings plays an important role in earthquake scenario projects where the risk of damage in a certain area is estimated in order to decide on appropriate risk reduction strategies

1.9.3. Definition of a vulnerability function

In general, a vulnerability function is a relationship which defines the expected damage for a building or a class of buildings as a function of the ground motion (Figure 1.7a, b, c). The two key elements of a vulnerability analysis are the capacity of the building and the seismic demand. In order to estimate the damage D, the ability of the building to resist constraints
(capacity of the building) must be compared with the constraints on the structure due to the earthquake ground motion (seismic demand).

**Principle of a vulnerability function**

**Figure 1.7.a. Capacity of the building.**  
**Figure 1.7.b. Seismic demand**

**Figure 1.7.c. Vulnerability function of the building**

In earthquake engineering the capacity of a building to resist seismic action is presented by a capacity curve which is defined as the base shear $V_b$ acting on the building as a function of the horizontal displacement at the top of the building $\Delta$, also often referred to as a pushover curve. The shear capacity of the building refers to the maximum base shear the building can sustain $V_{bm}$ and the displacement capacity refers to the ultimate displacement at the top of the building $\Delta_{bu}$. 
In a more general way, it is possible to express the capacity of any structure (building) or structural element (wall, wall element) to resist seismic action by the shear force acting on it as a function of the horizontal displacement at the top (capacity curve). Likewise, the shear capacity of any structure or structural element refers to the maximum shear force it can sustain, and the displacement capacity refers to its ultimate horizontal displacement.

To express the seismic demand, until very recently, the “intensity” was used nearly exclusively. This is a descriptive parameter of an earthquake based on observations of the effects of the earthquake on the environment. It has the advantage that historical data on earthquakes are available. However, information on the real ground movement is lost and empirical relationships between intensity and peak ground acceleration vary a lot (cf. Section 7.3). Some methods use the peak ground acceleration as the parameter defining the earthquake. However, in that case, not only the information on the duration of the earthquake is lost, but also the information on the frequency content. Thus, a better parameter is the spectral acceleration $S_a$, or, as we will see, the spectral displacement $S_d$.

The ground movement due to an earthquake does not happen in a fixed direction, on the contrary, in a horizontal plane the direction of the ground movement varies, including all angles from 0 to 360°. However, the biggest amplitudes of the ground movement usually occur in one direction, the amplitudes in the other directions, especially orthogonal to the direction of the biggest amplitudes, are much smaller [Mo 93]. Thus the constraints on the building are predominant in the direction of the biggest amplitudes which is referred to in the following as the ‘direction’ of the earthquake.

For regular buildings, it is common practice in earthquake engineering to consider the earthquake action (i.e. the direction of the biggest amplitudes of the ground movement) separately in two orthogonal directions, usually corresponding to the principal axes of the building, using plane analysis. Thus for one building two vulnerability functions are calculated. For earthquake scenarios, the direction of an earthquake is usually not taken into account and, based on the two vulnerability functions in the two principal directions, a single representative vulnerability function of the building has to be calculated [Thom. 96]. This representative vulnerability function should describe the overall behavior of the building and hence should be some sort of ‘mean’ of the two vulnerability functions in the two principal directions. Choosing the more unfavorable vulnerability function of the two would lead to a “worst case scenario” which is not desired in the case of earthquake scenarios, as it can be
assumed that on average the building behaves better. For very irregular buildings the two vulnerability functions in the two principal directions might be very different and thus the direction of the earthquake action plays an important role. Since this is not taken into account, the inaccuracy resulting from the introduction of a single representative function increases. This has to be kept in mind when considering the evaluation method proposed here.

1.9.4. Capacity curve of a building

A building capacity curve, termed also as ‘pushover’ curve is a function (plot) of a buildings’ lateral load resistance (base shear, $V$) versus its characteristic lateral displacement (peak building roof displacement, $\Delta R$). Building capacity model is an idealized building capacity curve defined by two characteristic control points: 1) Yield capacity, and 2) Ultimate capacity, i.e.:

![Building Capacity Model](image)

**Figure. 1.8. Building Capacity Model**

*Yield capacity* (YC, Fig. 1.8) is the lateral load resistance strength of the building before structural system has developed nonlinear response. When defining factors like redundancies in design, conservatism in code requirements and true (rather than nominal as defined by standards for code designed and constructed buildings) strength of materials have to be considered.
**Ultimate capacity** (UC, Fig. 1.8.) is the maximum strength of the building when the global structural system has reached a fully plastic state. Beyond the ultimate point buildings are assumed capable of deforming without loss of stability, but their structural system provides no additional resistance to lateral earthquake force.

Both, YC and UC control points are defined as:

\[
YC (V_y, \Delta_y): \quad V_y = \gamma C_s \quad \Delta_y = \frac{V_y}{4\pi^2} T^2 \quad (1.12a)
\]

\[
UC (V_u, \Delta_u): \quad V_u = \lambda V_y = \lambda \gamma C_s \quad \Delta_u = \lambda \mu \Delta_y = \lambda \mu \gamma C_s \frac{T^2}{4\pi^2} \quad (1.12b)
\]

Where:
- \(C_s\) design strength coefficient (factor of building’s weight),
- \(T\) true “elastic” fundamental-mode period of building (in seconds),
- \(\gamma\) “overstrength” factor relating design strength to “true” yield strength,
- \(\lambda\) “overstrength” factor relating ultimate strength to yield strength, and
- \(\mu\) “ductility” factor relating ultimate (\(\Delta_u\)) displacement to \(\lambda\) times the yield (\(\Delta_y\)) displacement (i.e., assumed point of significant yielding of the structure).

Up to the yield point, the building capacity is assumed to be linear with stiffness based on an estimate of the true period of the building. From the yield point to the ultimate point, the capacity curve transitions in slope from an essentially elastic state to a fully plastic state. Beyond the ultimate point the capacity curve is assumed to remain plastic.

Building capacity curves could be developed either analytically, based on proper formulation and true nonlinear (Response History Analysis, RHA) or nonlinear static (NSP) analyses of formulated analytical prototypes of model buildings, or on the basis of the best expert’s estimates on parameters controlling the building performance. The latter method, based on parameter estimates prescribed by seismic design codes and construction material standards, in the following is referred as the Code Based Approach (CBA).
1.9.5. Capacity spectrum

For assuring direct comparison of building capacity and the demand spectrum as well as to facilitate the determination of performance point [ES 97], base shear (V) is converted to spectral acceleration (Sa) and the roof displacement (ΔR) into spectral displacement (Sd). The capacity model of a model structure presented in AD format (Fig. 1.9) is termed Capacity Spectrum (Freeman, 1975, 1998). To enable estimation of appropriate reduction of spectral demand, bilinear form of the capacity spectrum is usually used for its either graphical (Fig. 1.9) or numerical [(Ay, Dy) and (Au, Du), Eqs 1.13.] representation.

Conversion of capacity model (V, ΔR) to capacity spectrum shall be accomplished by knowing the dynamic characteristics of the structure in terms of its period (T), mode shape (φi) and lumped floor mass (mi). For this, a single degree of freedom system (SDOF) is used to represent a translational vibration mode of the structure.

Two typical control points, i.e., yield capacity and ultimate capacity, define the Capacity spectrum (Fig. 1.9.):

\[ YC (A_y, D_y): \quad A_y = \frac{C_y \gamma}{\alpha_i} \quad D_y = \frac{A_y}{4\pi^2 T^2} \]  

(Figure 1.9. Building capacity spectrum)
\[ \sum_{i=1}^{n} m_i \phi_i = \sum_{i=1}^{n} m_i \phi_i \]

\[ \alpha_1 = \left( \frac{\sum_{i=1}^{n} m_i \phi_i}{\sum_{i=1}^{n} m_i} \right)^2 \]  

(1.14)

Where:

- \( m_i \) is the \( i \)-th story mass,
- \( \phi_i \) is the \( i \)-th story modal shape coefficient.

Based on first mode vibration properties of a vast majority of structures, literature suggests even more simplified approaches. Each mode of an MDOF system can be represented by an equivalent SDOF system with effective mass (\( M_{\text{eff}} \)) equaling to

\[ M_{\text{eff}} = \alpha_1 M \]  

(1.15)

Where \( M \) is the total mass of the structure. When the equivalent mass of SDOF moves for distance \( S_d \), the roof of the multi-storey building will move for distance \( \Delta \). Considering that the first mode dominantly controls the response of the multi-storey buildings, the ratio of \( \Delta r/Sd = PF_{R1} \) is, by definition the modal participation for the fundamental (first) mode at a roof level of MDOF system:

\[ PF_{R1} = \frac{\sum m_1 \phi_{R1}}{\sum m \phi_{R1}^2} \]  

(1.16)

Where \( \phi_{R1} \) is the first mode shape at the roof of MDOF system.
1.9.6. **Identification of structural and non-structural elements**

In every building it must be distinguished between structural and non-structural elements. Structural elements are those elements of the building that help to support the horizontal and vertical forces acting on a building. The sum of all structural elements constitutes the structural system. The most common structural systems found in buildings are:

- **Structural frame systems**: The structural elements are beams and columns, either made of steel or reinforced concrete, meeting at nodes.
- **Structural wall systems**: The structural elements are (structural) walls, either made of reinforced concrete or masonry.
- **Dual systems**: In these, reinforced concrete frames are combined with reinforced concrete or masonry walls to carry the vertical and horizontal forces.

Non-structural elements are those elements of the building that are connected to the structural system, but without a force bearing function. Examples of non-structural elements are:

- **Non-structural walls (partitions)**, used for separation purposes, however they do not carry any vertical or horizontal forces.
- **Gable walls**
- **Façade elements**, including windows and balustrades
- **Staircases**
- **Ceilings**
- **Installations** (mains, air-conditioning).

In contrast to the design of a building, where the structural system is chosen and therefore known, the evaluation of the building requires first the identification of the structural system with all its structural elements, since only these contributes to the capacity of the building. The non-structural elements add to the weight only. In the case of masonry buildings, this is usually less obvious since all the walls (façade walls and inner walls) consist of masonry and often no clear distinction exists. However, it is common practice to consider all walls with a thickness \( t \geq 12 \text{ cm} \) to be structural walls, i.e. acting to support the vertical and horizontal forces.
Considering the plan of the building in Figure 1.10, the walls shaded in black are considered as structural walls, having a thickness \( t \geq 20 \text{ cm} \) whereas the walls shaded in grey with a thickness \( t < 20 \text{ cm} \) are considered as non-structural walls.

\[ \text{Figure 1.10. Identification of structural and non-structural walls} \]

1.9.7. Terminology and structural models

Considering the building in Figure 1.11, the following terminology used in the context of the construction of the capacity curve of a building, irrespective of the material (masonry or reinforced concrete), is introduced:

- A wall is defined as a structural element of the building of length \( l_w \) and a height equal to the total height of the building, \( H_{\text{tot}} \) (indicated by the hatched area in Figure 1.11).
- A wall element can be any part of a wall of length \( l_w \) and any height \( h \) (not shown in Figure 1.11).
- A pier is a wall element of length \( l_w \) and of a height \( h_p \) equal to the height of the adjacent opening, which can be a window or a door (indicated by the lightly shaded areas).
- The spandrels are those parts of the building which lie between two openings in the vertical direction, thus joining the walls in one plane (indicated by the darkly shaded areas).
• All the walls in one plane joined by floors and spandrels constitute a wall plane. Thus a façade of a building constitutes a wall plane but likewise all the walls in one plane in the interior of the building.

• A wall panel is defined as part of a wall plane of any length and a height equal to the storey height $h_{st}$.

![Figure 1.11 Terminology](image)

Also shown in Figure 1.11 are the applied forces:

• The equivalent horizontal earthquake forces are assumed to be induced at the floor levels where the mass is the highest.

• The vertical loads include the self weight of the structure as a volume force, and the surcharges (non structural elements) and the live loads applied at the floor levels.
Due to the fact that the walls are joined by floors and spandrels, a coupling effect is produced. Depending on the extent of the spandrels, this coupling effect will be bigger or smaller. In the absence of spandrels where the walls are joined only by the floors (usually the case for reinforced concrete buildings) the coupling effect is negligible and the walls can be regarded as interacting cantilever walls. For deep spandrels (often found in masonry buildings) the coupling effect is considerable and has to be taken into account. A system of coupled walls can be analysed using a frame model.

In a general way, every wall plane can be regarded as a system of coupled walls, the case of interacting cantilever walls being a “limit case” where the stiffness of the spandrels becomes negligible with respect to the stiffness of the walls and hence the coupling effect reduces to zero.

*Figure 1.12. Bending moment distribution for the three cases of coupled walls a) negligible coupling effect (interacting cantilever walls), b) intermediate coupling effect and c) strong coupling effect due to horizontally acting earthquake forces and corresponding reactions*
Figure 2.6 shows the bending moment distribution for three cases of coupled walls submitted to horizontal forces. Figure 1.12 a) shows the case where the walls are only joined by the floors and hence the coupling effect is negligible, the whole system can be regarded as interacting cantilever walls. Figure 1.12 c) shows the case of very deep spandrels producing a considerable coupling effect and Figure 1.12 b) shows an intermediate case, with some coupling effect.

In the case of interacting cantilever walls (Figure 1.12 a), the total overturning moment due to the applied horizontal forces is carried by the walls alone, proportional to their stiffness, resulting in very high bending moments at the base of the walls. In the case of strongly coupled walls (Figure 1.12 c), the total overturning moment due to the applied horizontal forces is mainly carried by high normal forces in the outer walls resulting from the vertical shear forces transmitted by the spandrels. The bending moments at the base of the walls are therefore rather small compared to those of a cantilever wall. In the intermediate case (Figure 1.12 b) the frame action is less and hence that part of the total overturning moment carried by the walls is increased whereas the normal forces are reduced.

For regular frames the extent of the coupling effect can be expressed by a single parameter, the height of zero moment $h_0$ (Figure 1.12). The smaller the value of $h_0$, the bigger the coupling effect. For infinitely stiff spandrels the limit value of $h_0 = 0.5 \ h_{st}$. As the coupling effect reduces, the height of the zero moment $h_0$ increases, eventually becoming greater than $h_{st}$.

Note that for $h_0 > h_{st}$, $h_0$, does not indicate the height of a true point of zero moment but corresponds to the height of the extrapolated zero moment of the pier.

The transfer of the horizontal inertia forces of the floors onto the walls has to be provided by the floor-wall connection. In the case of concrete floors, the connection between floors and walls is usually good, and thus the transfer of the horizontal forces onto the walls can be guaranteed. In the case of timber floors, the connection between floors and walls can be very poor if not improved by special means such as steel bar anchorages, and the transfer of forces onto the walls may not be guaranteed leading to an uneven distribution of the forces, overstressing some walls, while others remain almost unstressed.
1.9.8. Construction of the capacity curve

It is assumed that a wall only carries shear forces about its strong axes; the shear carrying capacity about the weaker axes is neglected. Assuming the floors to be totally rigid in their plane, thus assuring equal displacements of the walls at the floor levels, the capacity curve of the building in one direction can be obtained by superimposing the capacity curves of all the walls acting in this direction:

\[ V_h(\Delta) = \sum_j V_j(\Delta) \]  

(1.15)

\( j \) is the wall index, \( j = 1 \ldots n \), \( n \) being the total number of walls acting in one direction. This is allowed as long as the geometry of the building is relatively regular and torsional effects can be neglected.

Figure 1.13 shows plan and three elevations of a fictitious example building. Considering the x-direction, four walls acting in this direction can be identified, denoted by wall 1, wall 2, wall 3 and wall 4. The contribution of the two walls in y-direction is neglected. Wall 3 and wall 2 lie in one plane constituting one wall plane of the building (a façade wall plane). Wall 1 constitutes the second wall plane of the building (also a façade wall plane). Wall 4 constitutes a third wall plane in the interior of the building. Also given are three elevations of the buildings along the axes A-A, B-B and C-C. In the two outer wall planes which constitute the two façades (A-A and C-C) the spandrels are rather deep, producing a considerable coupling effect, whereas in the inner wall plane (B-B) the wall is only ‘linked’ by the floors leading to a very reduced coupling effect.
The corresponding capacity curve as shown in Figure 1.14 is given by:

\[ V_b(\Delta) = V_1(\Delta) + V_2(\Delta) + V_3(\Delta) + V_4(\Delta) \]  

(1.16)

Using a bilinear approximation of the capacity curve of the fictitious example building, the stiffness of the linear elastic part \( k \) corresponds to the sum of the effective stiffnesses of the walls:

\[ k = \frac{V_{bm}}{\Delta_{by}} = \sum_j k_{effj} \]  

(1.17)
Where:

$V_{bn}$ is the shear capacity and

$\Delta_{by}$ the nominal top yield displacement of the building (Figure 1.14.).

In the case shown in Figure 1.14. this leads to a stiffness of the building $k$:

$$k = k_{eff\,1} + k_{eff\,2} + k_{eff\,3} + k_{eff\,4}$$  \hspace{1cm} (1.18)

### 1.9.9. Demand spectrum

The level and frequency content of seismic excitation controls the peak building response. The elastic response spectrum ($S_{ae}$) is an extremely useful tool characterizing ground motions demand. It also provides convenient means to summarize the peak responses of all possible linear SDOF systems to a particular component of ground motion. It is usually computed for 5 percent damping being representative for a waist majority of structures [Tr.Mi 2.05].

### 1.9.10. Seismic demand

The seismic demand is determined using a response spectrum. A response spectrum presents the maximum response of single-degree-of-freedom systems (SDOF) as a function of their frequencies. Traditionally in earthquake engineering an acceleration response spectrum is used with regard to force based design and assessment procedures. Recently, design and assessment procedures focus more on displacements and deformations which are considered to be the more relevant parameters [Tr.Mi 3.05]. The use of a displacement response spectrum seems therefore more appropriate. However, except for very small frequencies ($f < 0.2\,\text{Hz}$) the following simple relationship holds:

$$S_a \approx \omega^2 \cdot S_d$$ \hspace{1cm} (1.11)

$S_a$ and $S_d$ are the spectral acceleration and the spectral displacement respectively, and $\omega$ is the corresponding circular frequency, $\omega = 2\pi f$ ($f$ is the frequency in Hz).
The use of a response spectrum assumes that the building, which can be seen as a multidegree-of-freedom system (MDOF) where the masses are concentrated at the floor levels and the mass of the walls is divided between the two levels above and below (Figure 1.15.), can be described by an equivalent SDOF system characterized with an equivalent mass \( m_E \) and an equivalent stiffness \( k_E \), having the same fundamental frequency as the MDOF system:

\[
f_1 = \frac{1}{2\pi} \sqrt{\frac{k_E}{m_E}}
\]

(1.12)

If the stiffness \( k \) of the real structure obtained from the bilinear approximation of the capacity curve of the building (cf. Figure 1.14 and Equation (1.6)) is used as the equivalent stiffness \( k_E \) of the SDOF system:

\[
k_E = k = \frac{V_{hm}}{\Delta_{by}}
\]

(1.13)

The equivalent mass is given as:

\[
m_E = \sum m_i \phi_i
\]

(1.14)

In which \( m_i \) is the concentrated mass and \( \phi_i \) is the first mode displacement at the i-th floor level normalized such that the first mode displacement at the top storey \( \phi_n = 1 \). The equivalent height is:

\[
h_E = \frac{\sum h_i m_i \phi_i}{\sum m_i \phi_i}
\]

(1.15)
In which \( h_i \) is the height of the \( i \)-th floor level.

Each quantity of the MDOF system can be transformed into the equivalent SDOF system using the following equation:

\[
Q = \Gamma \cdot Q_E
\]  

(1.16)

\( Q \) represents the quantities in the MDOF system (base shear \( V_b \), top displacement \( \Delta \)) and \( Q_E \) represents the quantities in the equivalent SDOF system (force \( F_E \), displacement \( D \), with the maximum displacement denoted as \( S_d \)). \( \Gamma \) is the modal participation factor defined as

\[
\Gamma = \frac{\sum m_i \phi_{i,1}}{\sum m_i \phi_i^2}
\]  

(1.17)

Two different approaches exist to obtain the displacement demand \( \Delta_D \) at the top of the building taking into account the nonlinear behavior of the building. One is the use of inelastic demand spectra, the other is the use of highly damped elastic spectra. Using inelastic demand spectra, the displacement demand \( \Delta_D \) at the top of the building (= \( n \)-th storey) is related to the equivalent elastic displacement \( \Delta_{be} \):

\[
\Delta_D = c_n \cdot \Delta_{be}
\]  

(1.18)

This is illustrated in Figure 3.10 showing the base shear - top displacement relationship for a linear elastic behavior and a nonlinear behavior.

The constant \( c_n \) can be determined as a function of the strength reduction factor \( R \) and the ductility demand \( \mu_D \):

\[
c_n = \frac{\mu_D}{R}
\]  

(1.19)

Where the \( \mu_D \) is defined as:

\[
\mu_D = \frac{\Delta_D}{\Delta_{by}}
\]  

(1.20)
And the strength reduction factor \( R \) is defined as:

\[
R = \frac{V_{be}}{V_{bm}}
\]  

(1.21)

With

\[
V_{be} = k \cdot \Delta_{be} = k \cdot \Gamma \cdot S_d \left( f_1 \right)
\]  

(1.22)

Figure 1.16. Base shear – top displacement relationship for a linear elastic behavior and a nonlinear behavior
1.9.11. Vulnerability function

Varying the “intensity” of the seismic demand by increasing the spectral displacement $S_d(f_1)$ continually from zero onwards, the displacement demand of a building $\Delta_D$ increases continually following Equation (2.22) and a $S_d(f_1) - \Delta$ curve is obtained (Figure 1.17) [Du.St. 94]. However, this is not yet a vulnerability function. Only when the damage is taken into account, the vulnerability function is obtained (Figure 2.1). The top displacement $\Delta$ must therefore be associated with a measure of damage.

$$\Delta_D = c_n \cdot \Gamma \cdot \phi_n \cdot S_d(f_1)$$  \hspace{1cm} (1.23)

![Figure 1.17. Comparison of the different approaches to take into account the effects of non-linearity](image)

The main parameter used as indicator is structural damage, looking at individual walls as well as the whole building [Tr.Mi 1.05].

The vulnerability function is linear for $\Delta < \Delta_{by}$ since the capacity curve of the building for $\Delta < \Delta_{by}$ is in the linear elastic region (Figure 1.18) and hence in Equation (1.23). For $\Delta > \Delta_{by}$ the
capacity curve of the building is in the plastic region and hence \( c_a = \frac{\mu}{R} \) in Equation (1.23).

For buildings with \( f_1 \geq f_{c_1} \) the vulnerability function is therefore nonlinear for \( \Delta > \Delta_{by} \).

\[ V_b \]

\[ V_{bm} \]

\[ V_{m1} \]

\[ V_{m4} \]

\[ V_{m3} \]

\[ V_{m4} \]

\[ \Delta \]

\[ \Delta_{y1} \Delta_{y2} \Delta_{y3} \Delta_{y4} \]

\[ \Delta_{by} \]

**Figure 1.18.** Capacity curve of the fictitious example building of Fig 1.13. Including the damage grades

**Figure 1.19.** Vulnerability function of the fictitious example building of Fig. 1.13.
Chapter 2

GENERAL CONCEPT FOR SEISMIC RISK ASSESSMENT BASED ON DEVELOPED BUILDING VULNERABILITY FUNCTIONS

In many seismically active areas of the world this type of structure only constitutes a small part of the building stock whereas a major part of the buildings are older structures made of unreinforced masonry representing a significant risk during an earthquake. Thus the demand for upgrading strategies of these buildings has become increasingly stronger in the last few years, implying the assessment of existing unreinforced masonry buildings [Ri.Pe.No 90].

2.1. Global Strategy for Seismic Risk Mitigation

Management and mitigation of the expected seismic risk is one of the most important engineering objectives in seismically active regions [Ma. 04]. Engineering activities related to seismic risk mitigation should be focused to buildings in large urban and rural regions, specific metropolitan areas, cities of various sizes, distributed villages, and different types of single structures of the highest importance category [Ri.Zi.Hr 96].

However, to develop and propose the most appropriate strategy for seismic risk mitigation, taking into account the relevant specifics of the area under consideration, seismic risk (or seismic vulnerability) assessment appears as one of the basic and/or essential steps.

Regarding the complexity of the problem of seismic risk assessment, various methods and procedures have been applied in the past for both theoretical and practical purposes. However, most of the proposed procedures appear often quite impractical since various simple scoring or other simplified evaluation systems may not necessarily be relevant for different structures, different areas and area specific conditions.

In the present study suggested is for application a very practical, reliable and uniform procedure for building seismic risk assessment based on application of previously developed so-called empirical or theoretical building vulnerability functions.

The basic concept and subsequent steps of this specific procedure for seismic risk assessment in urban and rural areas through implementation of the developed building vulnerability functions is briefly discussed in the following text of this chapter.
2.2. General Concept for Seismic Risk Assessment Based On Developed Building Vulnerability Functions

The integral procedure presently suggested for assessing of the expected vulnerability and seismic risk of the considered region, sub region, city, etc. should involve the following basic steps:

1. identification of the present elements at risk and their distribution;
2. evaluation of the seismic hazard and its distribution;
3. derivation of the appropriate vulnerability functions applicable to the existing elements at risk (classes of buildings), describing the interrelation between the specific loss and seismic hazard intensity;
4. evaluation of the specific seismic risk per element at risk and the factor of participation in the existing volume of properties; and
5. evaluation of the total and/or cumulative seismic risk for the region under consideration.

Regarding the three factors determining the level of seismic risk, such as the value (cost) of the existing elements at risk, their vulnerability or specific loss potential and the seismic hazard, only the first two factors are under possible control and they can be therefore controlled by the pre-disaster management, risk mitigation programs or pre-disaster prevention programs. Although it is possible to control efficiently the specific loss of elements at risk, for example through their relocating to the regions of lower seismicity, it is still necessary to provide economically justified practical measures for protection of the rest of the elements at risk, which due to favorable natural and other conditions have to be located in the regions with higher seismicity [Hess 08]. For such earthquake-exposed elements at risk (or presently buildings), the level of acceptable risk should be defined and satisfied through the effective control of building vulnerability level. However, the basic criteria or practical level of economically acceptable seismic risk strongly depend on the level of the economic development of the considered region or the entire country.

Considering an earthquake as a natural phenomenon occurring beyond possible human control and bringing economic loss to the stricken area, it is necessary to qualify the exposed vulnerable elements at risk through defined vulnerability and/or loss prediction model of the integral considered region [Mi. 95].
A direct loss prediction model usually refers to physical damage expressed in terms of human fatalities and injuries, damage to regional and local infrastructure (road and railway systems, water and gas supply, sewerage, etc.), residential and other types of building structures or any other property or material goods lost or damaged during or immediately after an earthquake event. Besides the physical damage and functional interruption caused by the earthquake, there are also categories of indirect effects, which can be generally classified into economic and social damage [Dum.De 98]. Stagnation of industrial activities, decreasing of industrial production, regional post-earthquake reconstruction and extra expenditures for immediate rehabilitation of the affected area, are classes of typical indirect economic losses. Interruption of transportation, water and electric power supply system, decrease in civil and information services as well as unfavorable reputation of damaged areas can be considered as typical social damage.

In this study, particular attention is given to evaluation of seismic risk level through the developed building vulnerability functions, relevant for estimation of expected direct physical loss and damage to material properties as the most influential group of elements at risk to the total economic losses due to seismic activity [P.N.R.V 91].

The present procedure for evaluation of expected cumulative seismic risk basically includes the following subsequent steps [P.S.N.R 1.97], :

1. **Completion of building inventory**: This step includes identification of actual distribution of the existing and planned elements at risk (building classes). The existing volume of buildings has to be first classified by structural systems in representative categories for the entire considered region. Alternatives and possible scenarios of the future development could be also considered, and analyzed [Pe.Ri. 2.94].

2. **Determination of seismic hazard**: The seismic hazard of the considered region should be determined for different return periods and frequency of occurrence in relation to the economical lifetime of the considered elements at risk. Particular attention should be given to vibration effects and geological hazards in the densely populated areas by performance of seismic micro zoning studies and elaboration of micro zoning maps [Pe.Ri. 92].
3. **Derivation of empirical or analytical vulnerability functions of representative building classes:** For each element at risk (building type), vulnerability functions should be determined expressed as specific loss for a considered range of seismic hazard defined for the purpose of reconstruction, rehabilitation, pre-disaster measures and planning of new development. Verification of the derived empirical or theoretical vulnerability functions is needed to be performed using sufficiently large and reliable statistical or analysis samples from the region under consideration.

4. **Estimation of the specific seismic risk:** For the identified elements at risk, based on developed vulnerability functions, as well as evaluated levels and distribution of seismic hazard, specific seismic risk distribution could be evaluated for the considered region. It is, however, essential that the total volume of existing buildings classified in the corresponding categories of non-aseismic and aseismic structures, as well as the planned new developments should be included and considered with their space and density distribution within the entire region for the defined several seismic hazard levels expected during the life-time of the buildings.

5. **Estimation of the expected cumulative seismic risk:** Considering the defined elements at risk (their space and density distribution), and using the estimated specific seismic risk for the specified earthquake return period, expected and related seismic risk could be estimated [Ri. 94]. Considering the existing volume of each element at risk (building class) and evaluated corresponding specific seismic risk, estimation of the expected cumulative seismic risk could be derived for the related earthquake hazard levels. Seismic risk can be expressed as portion of the volume of the material used or as value representing evaluated direct economic loss.

The described basic concept and procedure for estimation of cumulative seismic risk level using the derived building vulnerability functions can be implemented consistently for estimation of cumulative seismic risk of a given region, sub region, cities of different sizes etc., but all required and relevant data have to be provided in a convenient form and used for the integral analysis purposes [Ri. 1.99].

One of the most important advantages of this concept is its generality of application, as well as possibility of derivation of comprehensive, systematic and reliable results related to seismic risk prediction.
Chapter 3

ADVANCED INERA-METHOD FOR DEVELOPMENT OF BUILDING VULNERABILITY FUNCTIONS BASED ON INELASTIC EARTHQUAKE RESPONSE ANALYSIS (Software: NORA2000-ANALOS2000)

Analytical prediction of the induced "damage level" of particular building structure under specified strong earthquake ground excitation is highly difficult engineering task because of the existence of simultaneous influences of various complex physical phenomena characterizing earthquake ground motions and the building response. Particular difficulty arises due to nonstationarity of the building stiffness and deformability characteristics and dynamic properties in general. Such effects are present due to strong nonlinear behavior of building structural and nonstructural components under induced seismic forces.

Consequently, analytical assessment of buildings vulnerability or development of so-called building vulnerability functions that will successfully relate variation of building damage level in accordance with selected earthquake hazard intensity parameter appears as an even more complex problem, both theoretically and computationally.

Regarding the objective of the present study, as well as the existing modeling complexity of the entire building including damage propagation process under expected earthquake ground motions with increasing intensity, for development of analytical vulnerability functions of selected representative buildings, an integral applicable procedure is adopted, based on parametric inelastic earthquake response analysis of selected individual building, considering basically restoring force characteristics of both structural and nonstructural elements.

The adopted procedure for analytical development of building vulnerability functions integrates several specific consequent steps established with consideration of some additional engineering and practical assumptions, presented and discussed further in the following text.

3.1. Building Model Formulation and Basic Elements at Risk

Observed severe damages, partial or total collapse of building structures of various structural types like steel, reinforced concrete, mixed, masonry and other buildings, during past earthquakes were mainly caused by severe damage concentration in the most critical structural bearing components, as a result of their inadequate design for this type of loading
and insufficient understanding of their inelastic behavior. Even in the case where under earthquake ground shaking total collapse of the structure is not produced, high vulnerability level or economical losses may be present due to extensive damages induced in the building nonstructural elements.

To achieve optimum conditions for realistic prediction of building dynamic response under given earthquake ground motion it is essential to formulate appropriate nonlinear building model considering the nonlinear effects of both, building structural and nonstructural elements. This is particularly important because the initial effective stiffness of the integral building may be largely increased by the effective stiffness of the present nonstructural elements such are various types of masonry infill, other specific types of nonstructural partition walls, etc.

In order to represent realistically nonlinear behavior characteristics of building constituent structural and nonstructural elements under repeated cyclic loads, appropriate nonlinear analytical models have to be selected and incorporated in the integral formulated nonlinear building model. Another difficulty in the phase of nonlinear building model formulation may appear due to insufficient understanding of nonlinear properties of building construction materials, construction joints, other specific construction details, etc.

It is of particular significance to point out the importance and necessity of using and performing appropriate experimental tests of structural materials and structural components in order to establish corresponding experimental data base necessary to formulate reliable nonlinear model of the integral building structure. With the existence of sufficient experimental data, analytical building model can be formulated to reflect realistically the most important structural physical properties resulting from actual material and construction quality, construction detailing and other construction characteristics specific for the country or region.

On the other hand, formulated discrete structural model has to be also sufficiently simple (considering optimal number of structural degrees of freedom) in order to make it practically applicable for parametric analysis of structural inelastic earthquake response under selected different earthquake ground motions.
Regarding the above stated, in the presently adopted procedure for development of analytical vulnerability functions, building modeling and earthquake response analysis is based on implementation of an equivalent two-dimensional model with reduced number of considered degrees of freedom but capable to realistically simulate the most important stiffness and deformability characteristic of the integral building structure through considered effects of the existing structural and nonstructural elements.

Taking into consideration the fact that the total building collapse may result from the produced failure of structural elements only, as well as considering significant difference in inelastic properties and damage propagation in building structural and nonstructural elements, these two building components are assumed to represent the two basic elements controlling damageability of the building.

With formulation of inelastic building model based on incorporated effects of structural and nonstructural elements as the two principal elements, as well as considering realistically their distribution through the building stories, analysis of building cumulative damage and/or vulnerability has been made possible through computed element extreme deformation excursions under specific input earthquake ground motion.

3.2. Representation of Earthquake Ground Motion

Implementing formulated realistic building model, structural dynamic response under given earthquake ground motion may be quite satisfactorily predicted. Consequently, it means that using computed building response for selected frequency content and intensity of input ground motion, building damage propagation, as well as final damage pattern and or cumulative vulnerability level can also be predicted implementing previously established applicable procedure and developed computer program for such specific analysis purposes.

However, it is apparent that the intensity of building dynamic response strongly depends on the actual frequency content of considered earthquake ground motions. So, for the case of development of predictive building vulnerability functions one of the main problems is selection of the representative input earthquake ground motions.

In order to represent dominant frequency range of expected earthquake ground motions, or simpler to select representative earthquake records to be used for development of analytical vulnerability functions of buildings, it is necessary to have appropriate understanding of local
soil characteristics, since specific dynamic behavior properties of the local soil may highly contribute in modification of actual earthquake induced bedrock vibrations. To solve problems related to quantification of the effects of local soil media, additional experimental and analytical studies have to be previously carried out, or alternatively to select representative existing earthquake records obtained for similar tectonic environment and local soil conditions.

In the case of presently adopted procedure for development of analytical vulnerability functions of the selected buildings, for representation of dominant frequency range of expected earthquake ground motions at foundation level, proposed is application of a representative set of several selected earthquake records. In such a case, building vulnerability is firstly analyzed separately for each selected earthquake record, while in derivation of building average and representative vulnerability functions, complete set of selected input earthquake motions is considered through regression line fitting procedure.

3.3. Analysis of Building Inelastic Earthquake Response

One of principal steps of the procedure for development of vulnerability functions of selected representative building structures includes extensive analysis of its inelastic earthquake response for seismic performances and seismic stability evaluation, considering the defined representative set of earthquake records as input seismic ground excitation. In general studies, earthquake building response is analyzed implementing the formulated inelastic dynamic model separately for longitudinal and transverse direction of the building. Story restoring force properties in analytical model have to be represented by appropriate hysteretic relations. However, realistic values of element model parameters are of essential importance and should be determined based on available experimental data and detailed capacity analysis of the respective structural and nonstructural components.

To analyze various aspects of building dynamic behavior under earthquake excitation, in the range of the first yielding up to the total failure, intensity of input earthquake ground excitation have to be varied in a wide range, starting form very low peak ground accelerations (i.e., PGA = 0.05 g) and subsequently enlarging it in the subsequent analysis cases up to defined maximum expected level. Since earthquake ground excitation is represented by the selected set of several representative ground acceleration time histories, and because their
intensity levels have to be varied, complete studies for each separate building require completion of a large number of nonlinear response analyses.

From the computed results of the building inelastic responses, considering as separate input each component of the previously defined set of earthquake acceleration histories, a significant difference in the building dynamic behavior can generally be observed. Dispersion of building response characteristics under different input earthquake ground excitations will basically express the effects of the existing frequency content of the considered earthquake ground motion.

Adopting practically applicable and simplified building dynamic model, proposed are as essential the following two response parameters directly applicable for controlling the intensity of the structural inelastic behavior: (1) the maximum or demanded inter-story drift (ISD) and (2) response ductility factor (Df) for structural and nonstructural elements of the respective stories. Those response parameters can be usefully applied to related progressive building damage or more specifically to evaluate damage propagation in the structural and nonstructural constituent elements of the integral building under earthquake ground motion.

To successfully evaluate structural vulnerability under increased intensity of earthquake ground excitation it is essential to relate earthquake ground motion parameters with the selected structural response parameters. To derive a practical procedure for vulnerability evaluation of the integral structure, it has been presently considered reasonable to assume peak ground acceleration (PGA) as convenient earthquake intensity parameter, because the effects of frequency content variation have been incorporated through consideration of the representative set of several earthquake records.

Considering the available statistical data from the conducted numerous parametric nonlinear earthquake response analyses, which basically relate the respective earthquake input intensity parameters (PGA) and computed structural response parameters (response inter-story drifts ISD), it is possible to derive corresponding relations representing structural dynamic response in respect to increasing intensity of earthquake ground excitation in the statistical sense. Some details related to derivation of earthquake ground motion - structural response relations are briefly presented in the following section.
3.4. Derivation of Earthquake Ground Motion - Structural Response Relations

For derivation of earthquake ground motion - structural response relations, which are further implemented as basic information representing structural response characteristics, the following assumptions have been initially made:

1. Due to existing differences in building dynamic response in both principal directions and because earthquake ground motion may be expressed dominantly in either longitudinal or transverse direction, nonlinear structural response and resulting vulnerability is considered to be separately analyzed and presented for both principal directions;

2. Parametric analysis results obtained for each implemented earthquake record as representative input ground motion, or more specifically, the computed structural response parameters (for all assumed different PGA levels) are treated as basic available statistical samples;

3. Damage propagation in building structural (bearing) elements (SE) is primarily controlled by their ductility capacity ($D_c$) and response "peak" interstory drifts. In the case of very low inter-story drift demand, zero damage of structural components has been assumed since in this range, those components behave dominantly linear.

4. Similarly, damage propagation in building nonstructural elements (NE) is controlled by the ratio of interstory drift demand and effective inter-story drift capacity ($\text{ISD}_c$). Since initial cracks are possible even for considerably low earthquake ground intensities, zero ISD was assumed as starting point.

5. The computed earthquake response parameters or response "peak" inter-story drifts under considered earthquake ground motions are treated separately for each story of the particular building, which actually enables more realistic location of accumulated damages in both structural (SE) and nonstructural (NE) elements.

Finally, following the assumptions listed above, set of representative earthquake ground motion - building response relations can be obtained for each building story and plotted for earthquake action in both principal directions, based on available statistical data from the computed (sufficient number of) nonlinear structural responses. In the following procedure,
these relations practically are considered as the basic information for damage and building vulnerability prediction through implementation of previously established appropriate damage criteria for existing structural and nonstructural elements, considering their specific individual load bearing and deformability capacity.

3.5. Damage Criteria of Structural and Nonstructural Elements Based on Load Bearing and Deformability Capacity

The previously defined sets of curves plotted for each building story relating earthquake inter-story drift demand (ISD) with the input earthquake intensity parameter (PGA) have been conveniently adopted to assess the induced damage level or representative so-called "induced damage degree" for the structural and nonstructural elements of each building story and different earthquake records [Al.Lu.Gu 05]. To prescribe the corresponding damage degree for existing structural (bearing) elements (SE) and nonstructural elements (NE) from the established relations ISD-PGA representative damage degree criteria have been introduced based on load bearing and deformability capacity characteristics of each story and building constituent elements considered in the formulated analytical model.

Introducing in the present concept the specified important step, which includes evaluation of the induced damage degree to all constituent building structural and nonstructural elements, provided are conditions for implementation of a practical engineering procedure for specific loss prediction at first of each individual element, and then of all building stories and integral building, respectively based on previously adopted element loss functions, in accordance with the available data for the cost of repair and strengthening of earthquake damaged buildings.

To establish practically applicable element damage criteria which will appropriately reflect the most important element damage characteristics, the following phenomenological failure properties, characterizing its hysteretic behavior up to total element collapse have been evaluated and considered:

1. Different building structural or load-bearing elements, such are reinforced concrete columns, steel columns, composite (SRC) columns, reinforced concrete shear walls, braced steel frame bays, mixed construction elements, large prefabricated RC bearing panels, etc., are generally characterized by specific nonlinear or
hysteretic behavior properties under increasing earthquake-like repeated loads up to failure. Most of these specific characteristics as well as corresponding failure modes have been to some extent understood from conducted various laboratory tests by many researchers in the past considering different test specimens and loading conditions. Based on the overall results form presently available experimental evidence, for this study purposes is adopted an uniform damage criterion providing conditions for engineering grading of earthquake induced element damage degree informative enough to get a certain engineering insight.

2. Analogously, in building construction, a large number of different nonstructural elements is optionally applied such are: different infill types (solid brick masonry, infill, hollow brick or hollow block masonry, gypsum, panels, etc.) various nonstructural local frames and partition walls, decorations, facade glasses, finalizations, installations, etc. For this type of building elements similar damage criteria have been adopted to express the particular damage degree induced in building nonstructural elements.

3. In both cases adopted is the same number of different damage degrees, ranging from the lowest level DD = 1 (element without damage) and coming to the highest damage level or degree DD = 5 (element total collapse) [Pe.Ri 1.94].

4. In both cases, adopted is a common interpretation of available results from conducted nonlinear laboratory tests of individual structural and nonstructural elements up to failure which include implementation of a practical criterion to distinguish specific hysteretic behavior characteristics for selected four basic deformation ranges characterizing the element load-bearing and deformability capacity. As it is well known, this four basic element deformation ranges are defined through consideration of three characteristic points (C, Y, U) realistically representing the element specific load-deformation envelope curve, where: (1) point-C represents cracking point, (2) point-Y represents yielding point and (3) point-U represents ultimate point. Consequently, to represent element load-bearing and deformability capacity through polygonal envelope curve, the following six parameters have to be defined:

   a. FC, DC = cracking point force and deformation
   b. FY, DY = yielding point force and deformation
   c. FU, DU = ultimate point force and deformation
5. Further, in both cases adopted are appropriate damage criteria through direct consideration of the above noted commonly applicable basic parameters representing specific nonlinear behavior characteristics of each individual structural or nonstructural element and the hereby introduced two additional points "P" and "L" in order to express more realistically the damage degree differences for the considered five deformation ranges [Pe.Ri 89]. The first or P-point is located between points Y and U and presently its corresponding deformation is defined assuming its middle location, or $DP = DY + 0.5 \times (DU - DY)$. The second - L or limit point is introduced to correct theoretical or instant element failure (defined as point-U) to a more realistic simulation of "practical failure stage" typical for the final physical phenomenon represented by rapid damage increase for small displacement increments. Presently, the displacement of limit point-L is defined as point representing an increase of element theoretical maximum ductility capacity for 10%, or $DL = DU + 0.1 \times (DU - DY)$. With consideration of the above described representative point on defined respective element envelope force - deformation curve for each "damage degree" associated is corresponding "damage degree range" defined by left and right boundary. In that respect, for presently considered five damage degree categories (DD = 1, 2, 3, 4 and 5) [Du.De. 95], the following damage degree ranges or damage degree criteria have been adopted, Fig. 3.1:

a. Range - 1 (DD = 1): $DO \leq d \leq DC$

b. Range - 2 (DD = 2): $DC < d \leq DY$

c. Range - 3 (DD = 3): $DY < d \leq DP$

d. Range - 4 (DD = 4): $DP < d \leq DL$

e. Range - 5 (DD = 5): $DL \leq d$

Where, d represents the peak or maximum relative displacement response of respective element during inelastic response of the integral building under specified earthquake ground motion.

To obtain some engineering sense of computed peak or maximum inter-story (or relative) displacements $d$, its value is presently converted into peak inter-story drift $ISD = d/HS$, where HS represents respective story height. To provide further implementation of ISD, similar conversion is made for displacements related to all specified characteristic points (C, Y, P, U, L) which results in corresponding inter-
story drifts: ISD of C, ISD of Y, ISD of P, ISD of U and ISD of L, defining analogously the left and right boundaries of the defined damage degree ranges.

6. Finally, based on defined damage degrees and respective damage degree ranges, provided are convenient conditions for detailed grading and description of the associated damage level and damage characteristics for each particular range taking into consideration different damaging characteristics of different elements as a result of specific material and construction quality characteristics, geometrical properties, connection and other relevant parameters. Such phenomenological description of the induced element damage degree can only be performed based on detailed analysis of available experimental results obtained from the performed corresponding nonlinear tests up to failure in laboratory conditions. Regarding this, detailed laboratory tests of different structural and nonstructural elements up to total failure appear as a highly important step, especially in the cases where specific nonlinear behavior characteristics of some elements are not enough or not at all investigated before.

![Figure 3.1. Element Typical Force – Displacement Envelope Curve with Five Specified Ranges](image-url)
3.6. Specific Loss Functions of Structural and Nonstructural Elements

Following the established uniform damage criteria of individual structural and nonstructural elements, based on their specific load bearing and deformability capacity [Cr.Pi.Bo 04], the present procedure includes implementation of the so called specific loss functions necessary to calculate the resulting effective (or specific) economical loss at the element level, Fig. 3.2.

![Fig. 3.2. Specific loss functions in structural and non-structural elements](image)

Principal objective and characteristics of presently introduced specific loss functions may be summarized in the following:

1. Element specific loss function basically expresses the required economical investment to repair the damaged element to any defined damage level or different damage degree [S.P.K.P. 90].

2. To satisfy different practical loss evaluation needs, in this procedure suggested is implementation of two different specific loss functions as follows: (specific loss function which expresses element repair cost to achieve the original or initially existing load-bearing and deformability capacity of the considered
elements, and (2) specific loss function which expresses element repair cost to achieve certain level of element upgrading in respect to its original strength and deformability characteristics.

3. Specific loss functions are strictly related to many specific conditions such are element type, material used, construction quality, repair method, etc., and should be established in advance based on detailed repair cost calculations for each damage degree [Po.Ri 93].

4. Presently, representative element specific loss functions are defined as polygonal lines through given ordinates at all five introduced characteristic point (Fig. 3.2).

5. Specific loss D(%) at the element level represents the required economical investment for element repair expressed in percentage of the total element cost (TEC). The considered assumption for element repair cost representation as percentage of the element cost, practically means that total cost of any individual structural (TCSE) or nonstructural element (TCNE) has to be defined in advance in order to enable superposition of computed individual element losses for loss presentation at different levels [Po.Ri 94]. At story level, the presentation commonly includes the computed: (1) partial story loss resulting from story constituent structural elements, (2) partial story loss resulting from story constituent nonstructural elements and (3) total story loss resulting from story constituent both, structural and nonstructural elements. Similarly, at building level the loss presentation commonly includes: (1) building loss resulting from building constituent nonstructural elements, (2) building loss resulting from building constituent nonstructural elements and (3) total building loss resulting from building constituent both, structural and nonstructural elements.

3.7. Seismic Vulnerability Functions of Integral Building

The previously derived damage criteria and corresponding specific loss functions of the constituent structural and nonstructural elements of the integral building, regarding, separately, dominant earthquake motion in building both principal directions, have been further implemented as basic elements for vulnerability evaluation of the integral building.
Before presenting the applied procedure for vulnerability evaluation of the integral building, it is essential to explain the presently adopted basic definition of the term. Vulnerability of any structural part under given earthquake types with earthquake intensity specified by the prescribed PGA value represents effective loss corresponding to needed economical investment to repair it and provide upgraded same pre-earthquake conditions. For the convenience, the effective loss (D) is presently expressed in percent (%) of the total building cost (TBC).

However, normalizing TBC = 1 per unit area and estimating for each constituent element only cost participation in respect to TBC, difficulties related to specification of any actual cost values such are, for example, the total building cost (TBC), the total cost of building structural elements (TCSE), the total cost of building nonstructural elements (TCNE), the story total cost of the structural elements (STCSE) and the story total cost of the nonstructural elements (STCNE), are avoided [R.P.H.Z.N. 94].

Additionally, due to the lack of cost specific data, for the present analysis purposes, a uniform distribution of the cost of structural and nonstructural components through the entire building height (for each story) is assumed.

Based on the derived basic damage criteria and corresponding specific loss functions for constituent structural and nonstructural elements of all building stories and introduced assumption related to building cost distribution, vulnerability evaluation of the integral building has been made possible [Si.No.Ri]. Regarding the presented computational background, the presently adopted procedure for building vulnerability evaluation, considering the dominant earthquake effect in one of the two principal directions actually consists of the following steps:

1. Tabular presentation of specific loss and damage degree distribution of structural and nonstructural elements for all building stories for selected different earthquake motion types and different earthquake intensity or PGA values. The presented specific loss D is expressed in percentage of the total building cost, since its cost participation has initially been specified in the same manner.
2. Using the computed specific loss for structural and nonstructural elements of all stories, evaluation of the cumulative specific losses $D$ for the integral building, and separately for building structural and nonstructural elements, corresponding to the same specified discrete PGA levels, and selected different earthquake types is consequently performed and commonly presented in a tabular and graphic form [Ri. 96].

3. Final evaluation of the integral building average vulnerability functions, or estimated average total loss $D$, percentage of the total building cost (TBC) [Ri]. The defined corresponding discrete points for all considered earthquake motion types are used to interpolate vulnerability functions in the analyzed vulnerability or earthquake intensity range. Since participation factors for structural and nonstructural elements in respect to total building cost are not explicitly available, the average vulnerability functions of the integral building are obtained considering engineering experience of the building construction cost, and analogy with similar structural types [Ri. 2.99].

Based on the described procedure in this chapter, as well as performed nonlinear analyses, using formulated corresponding non-linear multi-component building model (Fig. 3.3) specific loss and vulnerability functions can be developed for each specific building structure [Ri. 92].

This advanced, so-called INERA – Method for development of building vulnerability functions is based on INELASTIC EARTHQUAKE RESPONSE ANALYSIS of a given building.

Because of this, the method itself possess power of evident generality and can be very successfully applied in real engineering practice for evaluation of seismic vulnerability of buildings of different structural systems.
Chapter 4

FORMULATION AND VERIFICATION OF THE NONLINEAR ANALYTICAL MODELING FOR DYNAMIC RESPONSE AND FRAGILITY ON THE BUILDING WITH SIMULATION OF NONLINEARITY STRUCTURAL AND NONSTRUCTURAL ELEMENTS

Often creation of the analytical model for design of buildings resistant to seismic impacts of earthquakes considers only stiffness and deformation characteristics of structural elements of the building. This adaptation of the analytical model in many cases does not correspond to reality, where participation of non-structural elements in the overall stiffness and response can be considerable. Bead on this, both structural elements and non-structural elements should be considered in the evaluation of building loss [Mi.Ri 94].

4.1. Theoretical Concept for Nonlinear Analyses Dynamic Response

Non-linear dynamic response for the building as a whole presents one of the major phases in the definition of analytical fragility models that in our case were realized with the help of the software package NORA. Algorithm based on which the software is created is the basis of the theoretical concept, that is shortly described in this chapter.

Analytical concept of dynamic response for masonry load bearing walls in a plane is based on the results of the non-linear mathematic model. Non-linear mathematic model is a cantilever with concentrated masses in the mezzanines [Po. 03].

4.1.1. Formulation of dynamic nonlinear structural analysis

a) Dynamic analysis

All real physical structures, when subjected to loads or displacements, behave dynamically. The additional inertia forces, from Newton’s second law, are equal to the mass times the acceleration. If the loads or displacements are applied very slowly then the inertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is a simple extension of static analysis [Ch. 95].

All real structures potentially have an infinite number of displacements [Ch. 1.01]. Therefore, the most critical phase of a structural analysis is to create a computer model or program, with
a finite number of massless members and a finite number of node (joint) displacements, that will simulate the behavior of the real structure. The mass of a structural system, which can be accurately estimated, is lumped at the nodes. This is always true for the cases of seismic input or wind loads [Po.Ba 05].

b) Dynamic Equilibrium

The force equilibrium of a multi-degree-of-freedom lumped mass system as a function of time can be expressed by the following relationship:

\[
\{F\}^t_I + \{F\}^t_D + \{F\}^t_S = \{R\}^t
\]  

(4.01)

Where,

\[
\{F\}^t_I = [M] \cdot (\ddot{U})
\]  

is a vector of the nodal inertial forces;

\[
\{F\}^t_D = [C] \cdot (\dot{U})
\]  

is a vector of the nodal damping forces;

\[
\{F\}^t_S = [K] \cdot (U)
\]  

is a vector of the nodal restoring forces;

\[
\{R\}^t
\]  

is a vector of the nodal external forces;

From there, the equation of dynamic equilibrium can be written as:

\[
[M] \{\ddot{U}\} + [C] \{\dot{U}\} + [K] \{U\} = - \{M\} \{\dddot{U}_{s}\}
\]  

(4.02)

Equation (4.01) is based on physical laws and is valid for both linear and nonlinear systems if equilibrium is formulated with respect to the deformed geometry of the structure.

Considering the small increment of time, \(\Delta t\), equilibrium at time \(t + \Delta t\) can be written as:

\[
\left(\{F\}^t_I + \{\Delta F\}^t_I\right) + \left(\{F\}^t_D + \{\Delta F\}^t_D\right) + \left(\{F\}^t_S + \{\Delta F\}^t_S\right) = \{R\}^{t+\Delta t}
\]  

(4.03)

Where \(\{\Delta F\}^t_I\), \(\{\Delta F\}^t_D\) and \(\{\Delta F\}^t_S\) represent the changes of the nodal inertial forces, nodal damping forces and nodal restoring forces for the time increment \(\Delta t\), respectively. The total force vector for the time \((t + \Delta t)\), on the left hand side of Eq. (4.03), can be represented as:

\[
\left(\{F\}^t_I + \{\Delta F\}^t_I\right) = [M] \{\ddot{U}\}^{t+\Delta t}
\]  

(4.04)

\[
\left(\{F\}^t_D + \{\Delta F\}^t_D\right) = [C] \{\dot{U}\}^{t+\Delta t}
\]  

(4.05)
\[
\left( \{F\}_S + \{\Delta F\}_S \right) = \{F\}' + [K]'\{\Delta U\}'
\]  
(4.06)

By substitution of Eq. (4.04), (4.05) and (4.06) in Eq. (4.03), we obtain the incremental nodal point equilibrium equation at time t for the nonlinear structural system in the following form:

\[
[M] \{U\}^{t+\Delta t} + [C] \{\dot{U}\}^{t+\Delta t} + [K] \{\Delta U\}^{t+\Delta t} = \{R\}^{t+\Delta t} - \{F\}'
\]  
(4.07)

Where,

- \([M]\) – Structural constant mass matrix;
- \([C]\) – Structural constant damping matrix
- \([K]'\) – Structural tangent stiffness matrix at time \(t\);
- \(\{R\}^{t+\Delta t}\) – Vector of the external loads applying at time \(t+\Delta t\);
- \(\{F\}'\) – Vector of the nodal point forces corresponding to the element stresses at time \(t\);
- \(\{\Delta U\}^{t+\Delta t}\) – Vector of the nodal point accelerations at time \(t+\Delta t\);
- \(\{U\}^{t+\Delta t}\) – Vector of the nodal point velocities at time \(t+\Delta t\);
- \(\{\dot{U}\}^{t+\Delta t}\) – Vector of the nodal point displacements increments between time \(t\) and time \(t+\Delta t\);

Assuming constant structural stiffness matrix during the small increments of time, the solution of equation (4.07) provides approximate solution for the displacement increments \(\{\Delta U\}\), and the total displacements at time \(t+\Delta t\) can be calculated by addition to the known displacement at time \(t\):

\[
\{U\}^{t+\Delta t} = \{U\}' + \{\Delta U\}
\]  
(4.08)

c) Mass Matrix

In the present procedure, constant structural mass matrix is assembled as diagonal (lumped mass analysis approach) considering the contribution from element masses \([M]^{(e)}\) and additional concentrated nodal point masses \([M]^{(a)}\) which can be directly specified. So, the total structural mass matrix is calculated as:

\[
[M] + [M]^{(e)} + [M]^{(a)}
\]  
(4.09)
Considering diagonal form, the structural total mass matrix has been assembled as one dimensional vector in the computation procedure, disregarding zero entries out of diagonal, to reduce the storage requirement in the computer.

d) Damping Matrix

The structural damping matrix \([C]\) is assumed to be assembled as a linear combination of the constant structural matrix \([M]\) and constant (initial) structural stiffness matrix \([K]^L\) as follows:

\[
[C] = \alpha [M] + \beta [K]^L
\]  \hspace{1cm} (4.10)

Where \(\alpha\) and \(\beta\) are Rayleigh damping coefficients.

e) Step by step solution method

The most general solution method for dynamic analysis is an incremental method in which the equilibrium equations are solved at times \(\Delta t\), \(2\Delta t\), \(3\Delta t\), etc. There are a large number of different incremental solution methods. In general, they involve a solution of the complete set of equilibrium equations at each time increment. In the case of nonlinear analysis, it may be necessary to reform the stiffness matrix for the complete structural system for each time step. Also, iteration may be required within each time increment to satisfy equilibrium. As a result of the large computational requirements it can take a significant amount of time to solve structural systems with just a few hundred degrees-of-freedom.

In the present study, we considered the direct integration methods which do not employ uncoupling of the system of equations and can be successfully applied to calculate both linear and nonlinear dynamic response of general structural systems [Ch. 2.01]. Actually in the computer program are included two different direct integration procedures, in the literature known as Wilson- \(\theta\) and Newmark-\(\beta\) method. However, it is of significance to point that with appropriate derivation, both methods are condensed to completely equivalent calculation steps, expressing the difference only in the previously established eleven integration constants.

f) Wilson – \(\theta\) (Theta) Method

Basically, the Wilson \(\theta\) method is an implicit integration scheme derived considering the linear variation of acceleration during an extended time increment, namely from time \(t\) to time \(t + \theta\Delta t\), because of the considered constant \(\theta > 1\). However, when \(\theta = 1\), the method actually
reduces to frequently applied linear acceleration scheme. In the literature it is shown that for unconditional stability we need to use $\theta \geq 1.37$, and to satisfy this condition we presently $\theta = 1.4$. If with the variable $\tau$ we denote increase of time $t \leq t + \theta \Delta t$, the corresponding acceleration is given by:

$$\{\ddot{U}\}^{t+\tau} = \{\ddot{U}\}^{t} + \frac{\tau}{\theta \Delta t} \left( \{\ddot{U}\}^{t+\theta \Delta t} - \{\ddot{U}\}^{t} \right) \quad (4.11)$$

The variation of velocity and displacement can be easily obtained by integration of Eq. (4.11) as following:

$$\{\dot{U}\}^{t+\tau} = \{\dot{U}\}^{t} + \{\dot{U}\}^{t} \tau + \frac{\tau^2}{\theta \Delta t} \left( \{\dot{U}\}^{t+\theta \Delta t} - \{\dot{U}\}^{t} \right) \quad (4.12)$$

$$\{U\}^{t+\tau} = \{U\}^{t} + \{U\}^{t} \tau + \frac{\tau^2}{2 \theta \Delta t} \left( \{U\}^{t+\theta \Delta t} - \{U\}^{t} \right) \quad (4.13)$$

Introducing $\tau = \theta \Delta t$ in Eqs. (4.12) and (4.13) the velocity and displacement at the end of the extended time interval is given by:

$$\{\dot{U}\}^{t+\theta \Delta t} = \{\dot{U}\}^{t} + \frac{\theta \Delta t}{2} \left( \{\dot{U}\}^{t+\theta \Delta t} + \{\ddot{U}\}^{t} \right) \quad (4.14)$$

$$\{U\}^{t+\theta \Delta t} = \{U\}^{t} + \{U\}^{t} \theta \Delta t + \frac{(\theta \Delta t)^2}{6} \left( \{U\}^{t+\theta \Delta t} + 2 \{\dot{U}\}^{t} \right) \quad (4.15)$$

From Eq. (4.15) we can solve for $\{\ddot{U}\}^{t+\theta \Delta t}$ in term of $\{U\}^{t+\theta \Delta t}$

$$\{\ddot{U}\}^{t+\theta \Delta t} = \frac{6}{(\theta \Delta t)^2} \left( \{U\}^{t+\theta \Delta t} + \{U\}^{t} \right) - \frac{6}{\theta \Delta t} \{\dot{U}\}^{t} - 2 \{\ddot{U}\}^{t} \quad (4.16)$$

Now, substituting Eq. (4.16) in Eq. (4.14), the velocity $\{\dot{U}\}^{t+\theta \Delta t}$ can be also expressed in terms of only unknown displacement $\{U\}^{t+\theta \Delta t}$

$$\{\dot{U}\}^{t+\theta \Delta t} = \frac{3}{\theta \Delta t} \left( \{U\}^{t+\theta \Delta t} + \{U\}^{t} \right) - 2 \{\dot{U}\}^{t} - \frac{\theta \Delta t}{2} \{\ddot{U}\}^{t} \quad (4.17)$$
In this method equilibrium is considered at time $t + \theta \Delta t$, and the obtained displacement increments for each extended time interval are subsequently used to calculate the displacements, velocities and the accelerations for time $t + \Delta t$. However, because the accelerations are assumed to vary linearly, a linearly projected load vector is used. The total acceleration and velocities at the end of the extended time interval (4.16) and (4.17), respectively can be simpler expressed through the introduced certain integration constants and vector of incremental displacements, so considering $\theta \Delta t = \tau$,

\[
\{\dot{U}\}^{t+\tau} = a_0 \{\Delta U\} - a_1 \{\dot{U}\}^t - a_3 \{\ddot{U}\}^t
\]  

(4.18)

\[
\{\ddot{U}\}^{t+\tau} = a_1 \{\Delta U\} - a_4 \{\dot{U}\}^t - a_5 \{\dddot{U}\}^t
\]  

(4.19)

where,

\[
a_0 = \frac{6}{\tau^2}; \quad a_1 = \frac{3}{\tau}; \quad a_2 = \frac{6}{\tau} = 2a_1;
\]  

(4.20)

and

\[
\{\Delta U\} = \{U\}^{t+\theta \Delta t} - \{U\}^t
\]  

(4.21)

With the calculated incremental displacements (4.21), the total accelerations at time $t + \theta \Delta t$ are determined from (4.17). To obtain the solution for accelerations, velocities and displacements at time $t + \Delta t$ Eqs. (4.11), (4.12) and (5.13) should be evaluating for time $t + \Delta t$.

\[
\{\ddot{U}\}^{t+\tau} = \frac{\theta - 1}{\theta} \{\dddot{U}\}^t + \frac{1}{\theta} \{\ddot{U}\}^{t+\theta \Delta t}
\]  

(4.22)

\[
\{\dot{U}\}^{t+\theta \Delta t} = \{U\}^t + \frac{\Delta t}{2} \{\dot{U}\}^t + \frac{\Delta t}{2} \{\ddot{U}\}^{t+\theta \Delta t}
\]  

(4.23)

\[
\{U\}^{t+\theta \Delta t} = \{U\}^t + \Delta t \{\dot{U}\}^t + \{\ddot{U}\}^t + \frac{\Delta t^2}{6} \{\dddot{U}\}^{t+\theta \Delta t}
\]  

(4.24)

Substituting Eq.(4.18) into Eq.(4.22) the acceleration at time $t + \Delta t$ can be firstly calculated, and then used to calculate the corresponding velocity and displacement from Eqs.(4.23) and
(4.24), respectively. For the convenience, Eqs. (4.22), (4.23) and (4.24) are expressed in terms of five additional integration constants, used to solve for the accelerations, velocity and displacements at the end of the current time step.

\[
\begin{align*}
\{\ddot{U}\}^{t+\Delta t} &= a_6 \{\Delta U\} - a_7 \{\dot{U}\}^t - a_8 \{\ddot{U}\}^t \\
\{\dot{U}\}^{t+\Delta t} &= \{U\}^t + a_9 \left(\{\dot{U}\}^t + \{\ddot{U}\}^{t+\Delta t}\right) \\
\{U\}^{t+\Delta t} &= \{U\}^t + \{\ddot{U}\}^t \Delta t + a_{10} \left(\{\dddot{U}\}^{t+\Delta t} + 2 \{\ddot{U}\}^t\right)
\end{align*}
\] (4.25) (4.26) (4.27)

Where:

\[
a_6 = \frac{a_0}{\theta}; \quad a_7 = -\frac{a_2}{\theta}; \quad a_8 = 1 - \frac{3}{\theta}; \\
\frac{\Delta t}{2}; \quad a_{10} = \frac{\Delta t^2}{6};
\] (4.28)

g) Newmark – β (Beta) Method

The Newmark’s generalized acceleration method assumes the following approximations for the nodal velocities and displacements for the time \(t+\Delta t\)

\[
\begin{align*}
\{\ddot{U}\}^{t+\Delta t} &= \{\ddot{U}\}^t + \left(1 - \delta\right)\{\dot{U}\}^t + \delta \{\dddot{U}\}^{t+\Delta t} \Delta t \\
\{U\}^{t+\Delta t} &= \{U\}^t + \{\ddot{U}\}^t \Delta t + \left[\frac{1}{2} - \alpha\right] \{\dot{U}\}^t + \{\dddot{U}\}^{t+\Delta t} \Delta t^2
\end{align*}
\] (4.29) (4.30)

Where the parameters \(\alpha\) and \(\delta\) can be selected to obtain the required integration stability and accuracy. When \(\delta = 1/2\) and \(\alpha = 1/6\), the above approximations correspond to the linear acceleration method, or when \(\delta = 1/2\) and \(\alpha = 1/4\), they correspond to the constant acceleration method. For the solution of displacements, velocities and accelerations at time \(t+\Delta t\), besides Eqs. (4.28) and (4.30), the equilibrium equations at time \(t+\Delta t\) have to be additionally included. To express the unknown accelerations and velocities in terms of displacements increments only, we can firstly solve for \(\{U\}^{t+\Delta t}\) from Eq. (4.30), and then substitute the solution into Eq. (4.29). From Eq. (4.30) we have:
\[
\{U\}_{i+\Delta t}^{i+\Delta t} = a_0 \{\Delta U\} - a_2 \{U\}^i - a_3 \{\dot{U}\}^i
\]  
(4.31)

Substituting Eq. (4.31) into Eq. (4.29), we obtain velocities as:

\[
\{\dot{U}\}_{i+\Delta t}^{i+\Delta t} = a_1 \{\Delta U\} - a_4 \{\dot{U}\}^i - a_5 \{\ddot{U}\}^i
\]  
(4.32)

Where the integration constants are:

\[
a_0 = \frac{1}{\alpha \Delta t^2}; \quad a_1 = \frac{\delta}{\alpha \Delta t}; \quad a_2 = \frac{1}{\alpha \Delta t}; \quad a_3 = \frac{1}{2\alpha} - 1; \quad a_4 = \frac{\delta}{\alpha} - 1; \quad a_5 = \left(\frac{\delta}{\alpha} - 2\right) \frac{\Delta t}{2};
\]  
(4.32)

The obtained relations (4.31) and (4.32) for \(\{\dot{U}\}_{i+\Delta t}^{i+\Delta t}\) and \(\{\ddot{U}\}_{i+\Delta t}^{i+\Delta t}\) can be substituted in equilibrium equation to solve for total displacements \(\{U\}_{i+\Delta t}^{i+\Delta t}\) in linear analysis, or to solve for displacement increments \(\{\Delta U\}\) in the nonlinear analysis.

The solution for \(\{\dot{U}\}_{i+\Delta t}^{i+\Delta t}\) can be obtained from Eq. (4.30), or actually from Eq. (4.31) substituting the calculated displacement increments \(\{\Delta U\}\):

\[
\{\dot{U}\}_{i+\Delta t}^{i+\Delta t} = a_6 \{\Delta U\} + a_7 \{\dot{U}\}^i - a_8 \{\ddot{U}\}^i
\]  
(4.33)

where:

\[
a_6 = a_0; \quad a_7 = -a_2; \quad a_8 = -a_3;
\]  
(4.34)

The solution for \(\{\ddot{U}\}_{i+\Delta t}^{i+\Delta t}\) is obtained from Eq. (4.29), substituting the calculated \(\{\dot{U}\}_{i+\Delta t}^{i+\Delta t}\):

\[
\{\ddot{U}\}_{i+\Delta t}^{i+\Delta t} = \{\ddot{U}\}^i + a_9 \{\dot{U}\}^i + a_{10} \{\dddot{U}\}_{i+\Delta t}^{i+\Delta t}
\]  
(4.35)

Where:

\[
a_9 = \Delta t (1 - \delta); \quad a_{10} = \delta \Delta t;
\]  
(4.36)

And finally, the total displacements for the time \(t+\Delta t\) are obtained from Eq. (4.35) substituting the calculated displacement increments,

\[
\{U\}_{i+\Delta t}^{i+\Delta t} = \{U\}^i + \{\Delta U\}
\]  
(4.37)
To use Newmark’s method, in the program two parameters should be specified by the user, i.e. \( \delta \geq 1/2 \) (usually 0.5) and \( \alpha \) which is expressed as:

\[
\alpha = \frac{1}{4} (0.5 + \delta)^2
\]  

(4.38)

and if \( \delta = 0.5, \ \alpha = 0.25 \), the method reduced to constant-average-acceleration scheme or the so-called trapezoidal rule.

\[ h) \ \text{Linear and Nonlinear Dynamic Analysis Procedure} \]

The incremental nodal point dynamic equilibrium equation of a linear system, derived in (4.07) is now written in the following form \[ (4.0).6 \] [St.Po 06]:

\[
[M] \{\ddot{U}\}^{t+\tau} + [C] \{\dot{U}\}^{t+\tau} + [K] \{U\}^t = \{R\}^{t+\tau}
\]  

(4.39)

Assuming that \( \tau = \theta \Delta t \), with \( \theta = 1.4 \), we define the time step size in the Wilson–\( \theta \) Method, while for Newmark – \( \beta \) method we consider \( \theta = 1 \). To calculate the corresponding vector of the nodal point external loads in Wilson – \( \theta \) Method at the end of the extended time interval, a linearly projected load vector assembled for the time \( t+\Delta t \) is used as follows:

\[
\{R\}^{t+\theta \Delta t} = \{R\}^t + \theta \left( \{R\}^{t+\Delta t} - \{R\}^t \right)
\]  

(4.40)

On the other side, the derived expressions in both methods for \( \{\ddot{U}\}^{t+\tau} \) and \( \{\ddot{U}\}^{t+\tau} \) in terms of unknown displacements \( \{U\}^{t+\tau} \) are in the same form as Eqs. (4.18) and (4.19):

\[
\{\ddot{U}\}^{t+\tau} = a_0 \left( \{U\}^{t+\tau} - \{U\}^t \right) - a_2 \{\dot{U}\}^t - a_3 \{U\}^t
\]  

(4.41)

\[
\{\ddot{U}\}^{t+\tau} = a_4 \left( \{U\}^{t+\tau} - \{U\}^t \right) - a_4 \{\dot{U}\}^t - a_5 \{U\}^t
\]  

(4.42)

Substituting expressions (4.40), (4.41), (4.42) in the equilibrium equation (4.39) we have:

\[
\left( [K] - a_0 [M] + a_1 [C] \right) [\Delta U]^{t+\tau} = \{R\}^t + \theta \left( \{R\}^{t+\Delta t} - \{R\}^t \right) + \\
\left[ M \right] \left( a_0 \{U\}^t + a_2 \{\dot{U}\}^t + a_3 \{U\}^t \right) + \\
\left[ C \right] \left( a_4 \{U\}^t + a_4 \{\dot{U}\}^t + a_5 \{U\}^t \right)
\]  

(4.43)

From equation (4.43), it is clear that the structure effective stiffness matrix \( \hat{K} \) and the effective load vector \( \{\hat{R}\}^{t+\tau} \) for the current step have to be calculated as:
\[
\hat{K} = [K] + a_0[M] + a_1[C] \quad (4.44)
\]

\[
\{\hat{R}\}^\tau = \{R\}^\tau - \{F\}^\tau + \theta(\{R\}^{\tau+\Delta\tau} - \{R\}^\tau) + [M]a_0\{U\}^\tau + a_2\{\dot{U}\}^\tau + a_3\{\ddot{U}\}^\tau + [C]a_4\{U\}^\tau + a_5\{\dot{U}\}^\tau \quad (4.45)
\]

Where, the
\[
\hat{K}\{\Delta U\}^\tau = \{\hat{R}\}^\tau \quad (4.46)
\]

And, the displacements for the time \( t \), directly is submitted in the next form as:
\[
\{\Delta U\} = \{U\}^\tau - \{U\}^\tau \quad (4.47)
\]

The incremental nodal point dynamic equilibrium equation of a nonlinear system, derived in (4.07) is now written in the following form:

\[
[M]\{\ddot{U}\}^\tau + [C]\{\dddot{U}\}^\tau + [K]\{\Delta U\} = \{R\}^\tau - \{F\} \quad (4.48)
\]

Where, the
\[
\hat{K}\{\Delta U\} = \{\hat{R}\}^\tau \quad (4.49)
\]

\[
\hat{K} = [K]^\tau + a_0[M] + a_1[C] \quad (4.50)
\]

\[
\{\hat{R}\}^\tau = \{R\}^\tau + \theta(\{R\}^{\tau+\Delta\tau} - \{R\}^\tau) + [M]a_2\{\dot{U}\}^\tau + a_3\{\ddot{U}\}^\tau + [C]a_4\{U\}^\tau + a_5\{\dot{U}\}^\tau + \{F\}^\tau \quad (4.51)
\]

Where \([K]^\tau\) is tangent (nonlinear) structural stiffness matrix assembled for the time \( t \). In case of total nonlinear structure,

\[
[K]^\tau = \sum_{e=1}^{n_{el}} [K]_e^t(e) \quad (4.52)
\]

Where, \( n_{el} \) is the total number of finite elements. In case of partially nonlinear elements (constant part) and nonlinear elements (nonlinear part):

\[
[K]^\tau = [K]^{(LP)} + [K]^{(NP)} \quad (4.53)
\]

After computation of (4.50) and (4.51), the imposed displacement increments in the current solution step can be solved from (4.49), and the corresponding vector of the nodal point
acceleration, velocities and displacements are calculated based on the expressions (4.25), (4.26), (4.27) as the derived relations corresponding to the considered Wilson’s integration scheme.

To calculate the nonlinear dynamic response of the structural system represented by the total nonlinear model or partially nonlinear model [Cho. 05], the following steps have been considered in the computer program:

1. Initial calculation of the structural stiffness matrix;
   
   1.1. Partly nonlinear model: assemble and save linear (constant) part of structure stiffness matrix $\left[K\right]^{(LP)}$ from the contribution of the linear element group;

   1.2. Total nonlinear model: Set up zero entries in the structure global stiffness matrix $\left[K\right]^{(LP)} = 0$.

2. Assemble initial (total linear) structure stiffness matrix $\left[K\right]^{(TL)}$ and save it to be used for assembling of the structure damping matrix $\left[C\right]$ and/or computation of the initial dynamic characteristics (Eigen problem);

3. Assemble the total mass matrix of the structure $\left[M\right]$, and with $\left[K\right]^{(TL)}$ and $\left[M\right]$, compute the structure damping matrix $\left[C\right]$ using the relation (4.10);

4. Specify the initial conditional $\{U\}^0, \{\dot{U}\}^0, \{\ddot{U}\}^0$;

5. Assemble the interpolated ground acceleration matrix $\left[F\right]^*$ for the actual solution step increment $\Delta t$, considering the originally stored ground acceleration records on the respective time step DTF;

6. Set up the following parameters and constants:

   6.1. Wilson’s method: $\theta = 1.4, \tau = \theta \Delta t$ and compute the corresponding integration constants.

   6.2. Newmark’s method: $\theta = 1, \tau = \Delta t$ and compute the corresponding integration constants.

7. Compute the constant part (CP) of the effective structure stiffness matrix:

$$\left[\hat{K}\right]^{CP} = \left[\left[K\right]^{CP} + a_0[M] + a_1[C]\right]$$

(4.54)
8. Start step-by-step computation considering for each step the following sequences:

8.1. Assemble the nodal external force vector, as the inertial forces due to the ground motion \( \{ \dot{U}_{g} \}_{t}^{\text{ext}} \), applying the relation

\[
\{ R \}_{t}^{\text{ext}} = \{ P \}_{t}^{\text{ext}} - [M]B[U]_{t}^{\text{ext}} ;
\]

8.2. Read the saved vector of nodal point forces \( \{ F \} \), which correspond to the element stresses at time \( t \);

8.3. Compute the nonlinear effective load vector \( \{ R \}_{t}^{\text{et}} \), using (4.51);

8.4. Read the saved vector of total strains \( \{ \varepsilon \} \) imposed in nonlinear elements, at the end of the previous step, and assemble the nonlinear part of the structure stiffness matrix \( \hat{K}^{\text{(NP)}} \) from its contributions.

8.5. Assemble the total effective structure stiffness matrix for the current step as:

\[
\hat{K} = \hat{K}^{CP} + \hat{K}^{\text{(NP)}}
\]  
(4.55)

8.6. Decompose the total effective structural stiffness matrix \( \hat{K} \);

8.7. Calculate the unknown displacement increment \( \{ \Delta U \} \), by solving the system of equations in the form (4.49);

8.8. Calculate the nodal point new accelerations, velocities and displacements corresponding to the end of the current solution step. Use (4.25), (4.26), (4.27);

8.9. Using the calculated incremental global displacements \( \{ \Delta U \} \), update the local displacement increment for the nodal points of all the elements and calculate the increments of the local element forces for the linear \( \{ \Delta \bar{S} \}^{L(e)} \) (in case they exist) and nonlinear elements \( \{ \Delta \bar{S} \}^{N(e)} \);

8.10. Compute the incremental element forces in the global coordinate system \( \{ \Delta \bar{S} \}^{L(e)} \) and \( \{ \Delta \bar{S} \}^{N(e)} \), and update the incremental nodal point load vector \( \{ \Delta F \}_{i} \) from the contribution of the linear (in case they exist) and nonlinear elements by

\[
\{ \Delta F \}_{i} = \sum \{ \Delta \bar{S} \}^{L(e)}_{i} + \sum \{ \Delta \bar{S} \}^{N(e)}_{i} ;
\]
8.11. Calculate and save the nodal point load vector corresponding to the imposed total displacements $^r\mathbf{M}\{U\}$, to be used in the next step as $^r\{F\}$;
8.12. Calculate and save the vector of total strains $\{\varepsilon\}$ imposed in the nonlinear elements at the end of the current solution step;
8.13. Repeat steps from 8.1 for the next solution step.

4.1.2. Analysis of Initial Dynamic Characteristics

Mode Shapes and Frequencies- EIGEN Problem Solution

Considering the previously assembled linear structural stiffness matrix $[K]$ and mass matrix $[M]$, the development computer program is capable to compute the initial dynamic characteristics (mode shapes and frequencies) of the modeled structure. The analysis of the initial dynamic characteristics is generally needed to define the Rayleigh damping coefficients $\alpha$ and $\beta$ which are used to assemble the structure damping matrix $[C]$ based on Eq. (4.10).

The inverse vector iteration has been considered as a convenient method to solve for the lower eigenvalues $\lambda_1, \lambda_2, \ldots \lambda_n$ and the corresponding eigenvectors $\{\phi\}_1, \{\phi\}_2, \ldots \{\phi\}_n$ considering the solution of the generalized eigenproblem in the form:

$$[K]\{\phi\}_i = \lambda_i[M]\{\phi\}_i \quad (i = 1, 2, \ldots n)$$

(4.56)

where $[K]$ and $[M]$ are structural initial stiffness and mass matrix, respectively. To compute the first eigenvalue $\lambda_1$ and the corresponding eigenvector $\{\phi\}_1$, the following iterative procedure has been implemented:

1. Assume that $\{Y\}_1 = [M]\{X\}_1$, where $\{X\}_1$ is the selected unit full starting vector, and evaluate for the subsequent iterations $K = 1, 2, \ldots n$ as follows:
2. From $[K]\{\bar{X}\}_{k+1} = \{Y\}_k$, solve for $\{\bar{X}\}_{k+1}$

(4.57)

3. Compute $\{\bar{Y}\}_{k+1} = [M] \{\bar{X}\}_{k+1}$

(4.58)

4. Get $\rho\left(\frac{\{\bar{X}\}_{k+1}^T \{Y\}_k}{\{\bar{X}\}_{k+1}^T \{\bar{Y}\}_{k+1}}\right) \rightarrow \lambda_1$

(4.59)
5. Get
\[
\{Y\}_{k+1} = \frac{\{\overline{Y}\}_{k+1}}{\sqrt{\{X\}^T_{k+1} \{\overline{Y}\}_{k+1}}} \rightarrow [M]\{\Phi\}_i
\]  

(4.60)

6. Check the convergence in each iteration corresponding the calculated eigenvalue \(\lambda_i^{(k)}\) in the previous iteration and the specified tolerance ACC:

\[
\frac{|\lambda_i^{(k+1)} - \lambda_i^{(k)}|}{\lambda_i^{(k+1)}} \leq ACC
\]  

(4.61)

Where, ACC should be specified as 1/10^{2P} or smaller, if \(\lambda_1\) is required to 2P – digit accuracy. Then, the eigenvector \(\{\phi\}_1\) will be accurate to about P or more digits.

If (4.61.57) is not satisfied, proceed with the next iteration and if satisfied, terminate iterations, where:

\[
\lambda_i \approx \rho \left(\{\overline{X}\}_{k+1}\right)
\]  

(4.62)

\[
\{\Phi\}_i \approx \frac{\{\overline{X}\}_{k+1}}{\sqrt{\{X\}^T_{k+1} \{\overline{Y}\}_{k+1}}}
\]  

(4.63)

The subsequent eigenvalues and eigenvectors are calculated applying vector deflation. If the iteration vector is deflated or orthogonalized to all the already calculated eigenvectors (m), the possibility that the iteration will converge to any of the previously calculated is eliminated. Under such conditions, the iteration converge to another eigenvector. In the present procedure the Gram-Schmidt method has been adopted for the vector orthogonalization, employing the following expression:

\[
\{\tilde{X}\}_i = \{X\}_i - \sum_{i=1}^{m} \alpha_i \{\Phi\}_i
\]  

(4.64)

\[
\alpha_i = \{\Phi\}_i^T [M] \{X\}_i, \quad i = 1, 2, ... m
\]  

(4.66)

and vector \(\{\tilde{X}\}_i\) is used as a starting iteration vector instead of \(\{X\}_i\), and because of the provided \(\{X\}^T [M] \{\Phi\}_{m+1} \neq 0\), iteration should converge to the next (m+1) eigenpair.

**4.1.2.1. Linear and Nonlinear Analysis Option**
As stated above, the present version of NORA computer program includes, in total, 10 different analysis options, as we can see in Flow-chart below, originally designed to provide computing of complete any of the following three analysis types:

1. Static step-by-step linear and nonlinear analysis
2. Analysis of the initial dynamic characteristics (Eigen-problem solution)

Since the analytical model of the structure can be composed of linear and nonlinear elements, the specific computation options considered in the static and dynamic analysis are separately listed below.

For computation of the static structural response under the prescribed time-dependent loads, the following three analysis options have been provided in the present computer program:

Option 1: Linear Static Analysis
Option 2: Static Analysis of Structures with local Nonlinearities
Option 3: Nonlinear Static Analysis.

In all the static analysis options, the incremental step-by-step solution procedure was adopted in order to provide structural response analysis due to prescribed time-dependent external loads in any of the global degrees of freedom (in linear analysis), as well as to update the current structural stiffness matrix for the imposed nonlinearities (in the case of nonlinear analysis or analysis of structural systems with local nonlinearities).

4.1.2.2. Analysis of structural initial dynamic characteristics
(Eigen-problem solution)

Option 4: In the case of dynamic linear and nonlinear response analysis, the structural damping matrix is formulated as a linear combination of mass and initial stiffness matrix (Rayleigh damping matrix). In order to define the corresponding Rayleigh damping coefficients, the structural initial characteristics, i.e. the frequencies and mode shapes have to be calculated. For the solution of the generalized Eigenvaleu problem, effective inverse vector iteration method was used in the computer code.
Figure 3.3. Analysis Option Flow-Chart of Developed Computer Program NORA for Nonlinear Earthquake Response Analysis of RC Structures Based on Proposed Stress-Strain Modeling
4.1.2.4. Dynamic linear and nonlinear analysis options

Assuming the possibility of linear and nonlinear dynamic analysis, as well as the possibility for Eigenvalue problem solution, a total of six different dynamic analysis options were originally considered, as follows:

Option 5: Linear Dynamic Analysis
Option 6: Eigenvalue Problem and Linear Dynamic Analysis
Option 7: Dynamic Analysis of Structures with Local Nonlinearities
Option 8: Eigenvalue and Dynamic Analysis with Local Nonlinearities
Option 9: Nonlinear Dynamic Analysis
Option 10: Eigenvalue problem and Nonlinear Dynamic Analysis.

In all of the above listed dynamic analysis options, the dynamic external load can be considered as real earthquake ground excitation, in which case the ground acceleration time history is assumed to be prescribed with discrete values specified at equal (constant) time step. It should be also pointed out that both, namely the horizontal and the vertical components of earthquake ground motions can by applied, and corresponding structural response analysis carried out.
1. GENERAL DESCRIPTION OF THE SELECTED SET OF REPRESENTATIVE MASONRY BUILDINGS FOR THE PRESENT STUDY

1.1. Introduction

The developed general applicable building structures presented in previous chapter, shall now be applied for original seismic vulnerability study of the selected representative existing masonry buildings in Pristina.

Pristina, Capital of Kosova, is known from historic monographies as an old city evolving from ancient Ulpiana, built mainly with small dense housing. Materials used for construction were mainly stone, wood, clay bricks and mud [Ba.La 02]. Today, Pristina is known as a modern built city, even though there are still housing blocks and other buildings constructed in early XX-th century with massive stone or clay brick bearing wall system, bricked with lime mortar.

Having in mind a considerable amount of existing masonry buildings in town, and variety of construction systems and shapes in this building category, this study is developed in a way to serve for future seismic vulnerability assessments at similar structures [Hr.Ri]. Pristina is characterized with a large number of overbuilds on existing buildings, what presents a real challenge for engineers during calculation of building capacity, especially difficulties in calculation of masonry buildings [Pe.Mi.Ri 89].

Possibility of earthquake strikes in our country, more precisely in Pristina, which theoretically, as per available data (from Seismological Report of Kosova), can be of a large intensity. Also, number of habitants in these buildings is not small, therefore economic consequences can be considerable. From the above it can be roughly estimated that this existing building category is most vulnerable from possible earthquake strikes, therefore the need for seismic vulnerability assessment for these buildings is necessary.
1.2. General Description of full set of representative masonry buildings in Prishtina

Development of the city, as time goes by, is embryonic, form the core (center) towards outskirts. Old historic buildings are concentrated mainly in the city centre, including religious cult buildings, museums, public schools and many residential buildings that are mainly with a small footprint and limited levels. Development of the city with new buildings is on the outskirts and includes good quality, mainly high reinforced concrete structures.

1.2.1. General Description of the All marked Buildings

In the city centre there are zones of residential buildings that are concentrated in blocks, constructed mainly with structural masonry walls. Apart of these blocks, there are isolated buildings constructed in the same system, with masonry walls. Among the large number of existing buildings in the city, we have marked 55 buildings for analysis in this study. Basic criteria for selection of buildings are: representation of a large number of buildings that can be grouped in a typical structure, variety of building usages, number of story’s, footprint dimensions.

From the marked of 55 buildings, which were inspected and measured on the site, selected are and unified 15 separate representative buildings for detailed analysis for the purpose of the present study.

In general, all buildings have a wooden roof structure. Roof structures are regular with main clasical timber trusses that rest on load baring walls. Above the trusses, there are purlins and ribbons. Roof covering is usually clay tiles.

1.2.2. General Description of selected buildings

Starting from the criteria to represent a large number of buildings that can be grouped in a typical structure [Ca. 99], below give are brief descriptions of building groups selected for analysis.


Figure 1.1. shows the layout of Block No.1 of residential buildings in Pristina. Similarity of the buildings is evident in the layout that simplifies creation of the representative typical building for analysis.
Block No. 1 consists of 14 separate buildings with typical floorplan shown in Figure 1.2. This floorplan is characterized with the symmetry along y axis. Along the perimeter of the building located are load baring walls in both directions - x and y, with a thickness of 35cm. Also in the interior there are load baring walls on both directions. These walls are capable to absorb all vertical and horizontal impacts on the building. As a result of the building dimensions, the capacity of the building may be lower along the shorter direction. This can be proved with the stiffness of load baring walls along both directions when vulnerability direction is determined.

Also there are non-structural partition walls in the interior of the building, that are usually thinner. Considering their thickness, non-structural elements have small influence in the overall stiffness of the building, because of their small capacity, even though we will include them in the analysis [Pe.Me.Ch 04] .

The building has the basement, ground floor, first floor, second floor and attic.
All 14 buildings of Block No. 1 will be represented with one unified typical building, named as “Building no. 11 – Residential building, “bloc #1” Nazim Gafurri str.”

Building mezzanine structures in Block No.1 are with timber elements shown in Figure 1.3. These structural elements are anchored in load baring walls, meaning they transmit horizontal impacts on structural elements.

Considerable negative phenomena, that took place lately, not only in private housing buildings, but also in residential building blocks in the city, is overbuild or renovation of buildings (change of destination of the storey or parts of the building from residential to commercial areas).
b. **Building #15, (in our analysis), part of Block No.2**

A good example of overbuilding in Pristina, are buildings in Block No.2. Figure 1.4. shows layout of Block No. 2:

![Block No.2 Layout](image1)

*Figure 1.4. Building block No.2, in Prishtina*

Block No.2 consists of 7 separate buildings, constructed with masonry load baring walls shown in figure 1.5. This floor plan is characterized with a very low amount of non-structural elements compared to structural elements. Symmetry of the floor plan along axis y is evident, and it is in favor of the building as far as center of rigidity and center of mass is concerned. Load baring walls along the perimeter are weakened with window openings, especially along x axis the openings are of a larger number. Along the y axis structural walls are placed in 7 planes and should have considerable stiffness.

![Floor Plan](image2)

*Figure 1.5. Floor plane of typical building on bloc #2*
Mezzanine structure of this building could be a benefit to stability, as it is with prefabricated reinforced concrete beams type “avramenko” that hold the reinforced concrete slab as shown in figure 1.6.

The reason for analysis of residential buildings grouped in Block No.2 is overbuilding shown in figure 1.7. that was constructed in 2000, where vertical load in the structural elements was considerably raised.

Figure 1.6. Typical concrete floor construction, type “Avramenko”

Figure 1.7. Building #15, (in our analysis), part of Block No.2.

In the list of analyzed buildings this one is shown as “Building no. 15 – Residential Building, Nazim Gafurri str. “Block #2”

c. Building No. 3 – Residential Building, Migjeni str.

Based on the number of similar buildings that was one of the criteria for selection of buildings to be analyzed in this study, figure 1.8. shows residential building block in Qafa complex.
Residential building block in Qafa complex consists of 6 identical buildings with a floor plan shown in Figure 1.9.

As seen in the building floor plan, structural walls are positioned in both axis – x and y. Another characteristic of this floor plan is that perimeter walls along y axis have large openings that have impact in the overall stiffness of the building. There is symmetry along y axis that is in favor of absorption of horizontal impacts. Also floor plan dimensions relation ly/lx=1.58 is favorable in absorption of earthquake impacts.

Mezzanine structures in the Qafa residential building block are with “Avramenko” type semi-prefabricated structure reinforced concrete beams and monolith slab, shown in Figure 1.6. This mezzanine structure is highly presented in existing buildings in the city, especially among the buildings constructed in mid XX century. Feature of this mezzanine structure are prefabricated beams, and the thin monolith concrete slab (usually ~5cm thick), therefore this system is treated as a ribbed slab that transmits the load on one direction, the direction of the beams. This structure was widely used also because of simple boarding.
This residential building block is constructed in 1930s, buildings have basement, ground floor and two floors. In the list of analyzed buildings, this one is shown as “Building no. 3 – Residential Building, Migjeni str.”

d. Building No. 2 – Residential Building, Fehmi Agani str.

Another residential building block located in the city centre is the one in “Small coffee bars” complex.

![Figure 1.10. Residential Building block in “small coffee bars” complex](image)

Typical floor plan of buildings consisting this block is presented in Figure 1.11.

![Figure 1.11. Typical building floor plan in “small coffee bar” complex](image)

Feature of this building, other than representation of 5 buildings, is the high variation of number of structural walls between basement and ground floor. In the y axis direction there is symmetry, as opposite to x axis, where asymmetry is high, that should result disfavorable in horizontal impact absorption along this axis.

Mezzanine structure, “avramenko” ribbed reinforced concrete structure, is shown in figure 1.6. Non-structural partition walls in the floor plans are of a low number compared to the
structural walls. In the list of analyzed buildings, this one is shown as “Building no. 2 – Residential Building, st. Fehmi Agani”.

e. **Building no. 14, Residential building, Sylejman Vokshi str.**

Buildings that apart from their representation of a number of buildings are characterized also with high dimensions relation lx/ly, and are part of residential building block in “Collegium Cantorum” quarter, are shown in Figure 1.12.

![Figure 1.12. Residential Building block in “Collegium Cantorum” quarter](image)

Floor plan of this typical building, which in the list of analyzed buildings is named as “Building no. 14, Residential building, Sylejman Vokshi str., “Collegium Cantorum” quarter”, is shown in Figure 1.13.

![Figure 1.13. Typical building floor plan in block, “Collegium Cantorum” quarter](image)

Special feature of this floor plan is dimensions ratio lx/ly=3.032, that results with different stiffness of the building along axis x and y. From this dimensions ratio of the building base, we will note what the dynamic response of the structure will be, and how preferable this ratio of the base sides is in active earthquake zones. Mezzanine structure is of a type shown in Figure 1.6.
A residential building, as far as representation of numerous buildings is concerned, is also "Building no. 7 – Residential Building, Sylejman Vokshi str.", which is characterized with a unique possible phenomena – partial overbuild over half of the base. There are two such buildings close to each other. It will be interesting to observe the behavior of the structure and its elements under the horizontal impacts of an earthquake, considering that the building has 4 levels. Figure 1.14. presents the perspective view of the building, and its floor plan is shown in Figure 1.15.

**Figure 1.14. Perspective of buildings**

From the floor plan we can see the position of structural walls along axis, but no non-structural partition walls can be observed. On all levels, structural walls have the thickness of 50cm, what ensures good stiffness considering the height and nature of such buildings.

**Figure 1.15. Floor plan of building**

Overbuild is on half of the building base, including the staircase. With the non-symmetric increase of building mass, there will surely be a drop of building response from horizontal impacts along y axis. Mezzanine structure is same as the one shown in Figure 1.6. Roof
structure on the un-overbuilt area has been renovated and has a larger slope. The roof cover is with clay roof tiles.

**g. Building No. 9 – Residential building, Qamil Hoxha str.**

A building system, with a regular symmetry, unweakened structural walls – without any new openings along the perimeter, is represented with “Building No. 9 – Residential building, Qamil Hoxha str.”. Special feature of this building is that along x axis, structural walls are 50cm thick, but along y axis, structural walls are thinner, with 25cm, Figure 1.16.

![Figure 1.16. Floor plane of building](image)

Reason for selection of this building type for analysis, is in the fact that a large number of such buildings are constructed not only in the city, but also in other surrounding areas like Fushe Kosova, Obiliq etc.

![Figure 1.17. Part of city Pristina, Residential Buildings, type #9.](image)
h. **Building no. 1 – FCA – Architectural Department Building**

In the category of buildings selected for their usage – buildings for educational use [Ba.Po 08], we have selected the building housing Faculty of Civil Engineering and Architecture (FCA) – Architectural department building.

While treating the category of school buildings, as part of public buildings with a large flow of students during the day, this chosen masonry building is considered paramount among a large number of University buildings with a same system.

In the analysis named as “Building No. 1 – FCA – Building of Architectural Department”, the building has a floor plan as shown in Figure 1.20, and a system with constructive walls on both directions x and y.
A feature of the FCA building, apart from its usage, is the fact that the structural elements along the perimeter have constant thickness of 50cm, and inner structural walls are 38cm thick. Numerous non-structural partition walls in the attic are 12cm thick.

Mezzanine slab, a specific feature in treated buildings, is a reinforced concrete slab with constant thickness and rests on both directions on structural walls.

Also the “sandwich” type roof cover is specific compared to roof covers of other buildings selected for analysis, which is characterized with a low weight and high load baring ability.

### i. Building no. 4 – Secondary School “7 September”, Hile Mosi str.

Among numerous school buildings in Pristina constructed with masonry system, is Public School named “Building no. 4 – Secondary School “7 September”, Hile Mosi str.”. Upon construction, the building was used as a political school, and now is serves as a secondary professional school owned by Municipality of Pristina. It was renovated several times, but there are no structural changes. The building consists of basement, ground and first floor. Ground and first floors areas are used for teaching, and basement serves for storage.

The reason for selection of this school lies in its floor plan – organization of structural walls and form of the base that does not meet the conditions for building center of mass and center of rigidity.
Floor plan of the building presented in Figure 1.21. shows that structural walls on the perimeter of the building along x axis have large openings and there is no symmetry. This form will be a special case in the study from the fact that there is no symmetry along any of the axis x or y.

![Figure 1.22. Perspective of Secondary School](image)

While structural walls present a large percentage in the building, non-structural partition walls are a few in number.

**j. Building no. 6 – Residential Building, Ilir Konushevci str. (ex city clinic center)**

In the category of buildings that have changed their destination during their exploitation, that is the interest of the analysis in this study, having in mind a large number of such building, we have selected “Building no. 6 – Residential Building, Ilir Konushevci str. (ex city clinic center)”.

Upon construction, in 1936, it was used as city clinic center – hospital, but was later converted to a residential building. Change of destination of the building results with demolition of many partition walls as well as their relocation, that in reality worsens the building’s behavior for all cases of external impacts. Figure 1.23. shows floor plan of the top floor as it is today.

![Figure 1.23. Floor plan of Residential Building (ex clinic center of city)](image)
Figure 1.24. façade, structural wall in perimeter of building.

Figure 1.24. shows the front façade wall which is in fact a structural wall. In general, looking at the façade wall we can observe a large number of openings. Along y axis, structural walls are numerous and are located in equal distances, and there is symmetry along this axis. Structural elements along x axis are with a large number of openings. Stiffness of the structure, structural walls for directions x and y is large compared to the ones along x axis, even despite base dimensions ratio.

**k. Building no. 5 – Residential Building, Ilir Konushevci str. (behind Health Station)**

“Building no. 5 – Residential Building, Ilir Konushevci str. (behind Health Station)” is also categorized in the group of buildings with the changed destination [Be.Fa.Me 05]. Special feature of this building is that its base has an irregular shape, as shown in figure 1.25.

Figure 1.25. Floor plane of Residential Building (behind of Health Station)

Apart form the irregular shape of the building’s base, what can be seen in a number of constructions in town, it can also be observed that structural walls have different thicknesses.
Figure 1.26 shows perspective view of the building where wall openings and levels can be seen.

![Figure 1.26. Perspective Residential Buildings](image_url)

Structural wall positioned centrally along y axis is 89cm thick, and all other structural walls have a smaller thickness.

A category of buildings attractive for the analysis, grouped by number and relatively equal dimensions ratio, are buildings numbered 8, 10, 12 and 13. Figure 1.27 shows mentioned buildings, which are located close to each other.

We expect to get interesting results for response of these buildings from horizontal impacts of earthquakes, considering their small height and dimensions.

![Figure 1.27. Plan view of the area indicating the location of private houses](image_url)
Building #8, with a base shown in figure 1.28, presents a case of buildings with small base dimensions.

![Figure 1.28, Base plan of Building #8](image_url)

Building #10 with a base shown in figure 1.29, is a case of individual housing with small base dimensions, constructed in masonry system.

![Figure 1.29, Base plan of Building #10](image_url)

Building #12, with a base shown in figure 1.30, presents the case of individual houses with symmetry along y axis, constructed with masonry walls along both axes. The building has three floors.

![Figure 1.30, Base plan of Building #12](image_url)

Building #13, with a base shown in figure 1.31, is a case of individual house constructed with masonry walls on both directions. The building has two floors.
Table 1.1, Specifications of Representative Sets of 15 Masonry Buildings for the Present Study.

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<thead>
<tr>
<th>Address Buildings</th>
<th>Number of Buildings</th>
<th>Usability</th>
<th>Storey</th>
<th>Dimensions of Base</th>
<th>Form Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Lx (m)</td>
<td>Ly (m)</td>
</tr>
<tr>
<td>B No. 1</td>
<td>1</td>
<td>Education Buildings</td>
<td>2</td>
<td>47.4</td>
<td>11.95</td>
</tr>
<tr>
<td>B No. 2</td>
<td>5</td>
<td>Residential Buildings</td>
<td>3</td>
<td>2.9</td>
<td>9.20</td>
</tr>
<tr>
<td>B No. 3</td>
<td>6</td>
<td>Residential Buildings</td>
<td>4</td>
<td>18.53</td>
<td>11.74</td>
</tr>
<tr>
<td>B No. 4</td>
<td>1</td>
<td>Secondary School</td>
<td>3</td>
<td>26.60</td>
<td>14.60</td>
</tr>
<tr>
<td>B No. 5</td>
<td>1</td>
<td>Residential Buildings</td>
<td>2</td>
<td>27.80</td>
<td>17.00</td>
</tr>
<tr>
<td>B No. 6</td>
<td>1</td>
<td>Residential Buildings</td>
<td>5</td>
<td>25.11</td>
<td>10.95</td>
</tr>
<tr>
<td>B No. 7</td>
<td>1</td>
<td>Residential Buildings</td>
<td>4</td>
<td>21.10</td>
<td>9.50</td>
</tr>
<tr>
<td>B No. 8</td>
<td>2</td>
<td>Residential Buildings</td>
<td>3</td>
<td>18.50</td>
<td>10.00</td>
</tr>
<tr>
<td>B No. 9</td>
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<td>Residential Buildings</td>
<td>3</td>
<td>22.96</td>
<td>10.00</td>
</tr>
<tr>
<td>B No. 10</td>
<td>1</td>
<td>Private House</td>
<td>3</td>
<td>12.00</td>
<td>8.50</td>
</tr>
<tr>
<td>B No. 11</td>
<td>14</td>
<td>Residential Buildings</td>
<td>5</td>
<td>20.68</td>
<td>11.49</td>
</tr>
<tr>
<td>B No. 12</td>
<td>1</td>
<td>Private House</td>
<td>3</td>
<td>18.00</td>
<td>13.20</td>
</tr>
<tr>
<td>B No. 13</td>
<td>2</td>
<td>Private House</td>
<td>2</td>
<td>9.20</td>
<td>10.40</td>
</tr>
<tr>
<td>B No. 14</td>
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<td>Residential Buildings</td>
<td>3</td>
<td>42.50</td>
<td>14.00</td>
</tr>
<tr>
<td>B No. 15</td>
<td>7</td>
<td>Residential Buildings</td>
<td>4</td>
<td>15.50</td>
<td>10.40</td>
</tr>
</tbody>
</table>
Chapter 2

GENERAL REVIEW OF THE SEISMICITY OF KOSOVO, THE CITY OF PRISHTINA AND DESCRIPTION OF THE SELECTED REPRESENTATIVE EARTHQUAKE RECORDS USED FOR THE PRESENT SEISMIC VULNERABILITY ANALYSIS OF REPRESENTATIVE BUILDINGS

2.1. General Description of Seismicity of Kosova

Kosovo represents an active seismic zone. Kosovo territory is covered with many areas of seismic sources, which present active separations or active separation areas that cause earthquakes. Active separations, that go deep under the surface, are places where earthquakes are born, and in particular the so-called seismotectonic joints, crossing points of active separations with different alignment directions, is where powerful earthquakes are expected.

In practice, methods of probability theory and statistical mathematics are used to determine the locations of future earthquakes and their possible impact on the surface.

The main problem in the description of earthquakes is related to the existence of sufficient data on earthquakes, but it must be said that there is limited information for Kosovo. With the combination of seismological data and those obtained on the basis of geological criteria, a seismic model for Kosovo and seismic activity of its main seismic sources is defined.

2.2. Seismology model of Kosova – seismology sources

The complex mechanism of seismic sources and the spread of seismic waves, represents a problem which has been given special attention recently. Determining the mechanism of sources, release of energy, type and borders of seismic sources with the aim to define seismic phenomena in the source, have enabled creation of mathematic model of seismic sources. Even though seismic sources present random processes, considering present experience results that, theoretically there are three different models of seismic sources in relation to the region: punctual, linear and planar.

Map of seismic sources for the territory of Kosovo is a combination of results from epicenters map, map of maximal observed strengths and maximum magnitude defined for each seismic source, geological mapping, statistical mapping of seismic activity, and neo-tectonic and
seismo-tectonic map of Kosovo and surrounding regions [Dum. 1.04]. These data represent the basic documentation needed.

Present knowledge on Kosovo and region seismology is such that allow classification of 15 seismic sources, of whom eight sources are linear and seven are planar (Fig. 2.1.).

![Seismo-tectonic Map of Kosovo and Surrounding Regions](image)

Fig. 2.1. Maximal Observed Strengths Map, Period 360 – 1950

Definition of boundaries of a seismic source is done based on concentration of epicenters in its interior and based on layout of active separations and relevant morfostructures. Geometric determination of seismic sources is difficult, especially because of the following factors: complexity of tectonic structures, main seismic areas neither begin nor end within Kosovo, there are seismic areas that lie outside the territory of Kosovo, seismic sources present processes that are unexpected in relation to time and space.

Here we can mention that, linear sources 1, 2, 3, 4 and planar sources 9 and 10 are determined based on geological criteria, which are based on the results of seismo-tectonic research.
2.3. Maximum magnitude of seismology sources

In engineering terms, registration of powerful earthquakes is of special interest, because they cause greater destruction effects on structures. When talking about the strength of earthquakes, we should separate notions magnitude and intensity. For the needs of this study we will use magnitude as a measure for the force of the quake. Magnitude as a measure for earthquakes is related to the amount of seismic energy released from the epicenter to the surface.

For some areas of seismic sources, with particular interest is to define maximum magnitude earthquakes, which represent the greater seismic risk in the seismic source, so one of the main seismic parameters.

2.3.1. Seismic risk maps of Kosova

Seismic risk at any point on Earth surface is presented as the superficial effect of the ground shaking, expressed through the maximum seismic intensity in the face (I) or maximum haste of land (%g), conditioned by all the seismic sources (epicenters) around this point.
In all methodologies used today in the world, seismic risk is defined as the probability that in a given point on the Earth surface, for a certain period of time $T$, can be felt or observed an earthquake intensity $I$ or maximum acceleration $A_{\text{max}}$. For example, in a certain city $X$, within a period of next $T=100$ years, the possibility of an earthquake with intensity $I_0 = IX$ degrees has the calculated probability 70%.

Higher the probability, longer will be time $T$, within which the specific earthquake is expected to happen. In this aspect seismic risk is more understandable, meaning it presents the period in which the specific earthquake is expected to hit. Therefore, seismic risk requires the recognition of two main elements that are mentioned above: the maximum possible energy in epicenters $M_{\text{max}}$ and its fading from the source to the construction site.

Seismic risk for the territory of Kosovo is determined by the maximum values of haste and intensity of the characteristic repeat time periods: 50, 100, fig.2.1, 200, 500, Fig. 2.2, and 1000 years with the probability of these events of 63%, according to the formula:

$$P(M,T) = 1 - e^{-N(M)T}$$

Where,

$N(M)$- presents report magnitude frequency, while $T$ is the time for which we want to calculate probability.

Map of seismic risk, ie spread of haste and maximum intensity, are calculated according to the following procedure:

- The territory of Kosovo is divided into a network of 0.10 in latitude and 0.10 in longitude.
- For each section point is calculated: haste and maximum intensity of soil shaking for different repetition periods.
- Through Interpolation with geometric methods of calculated values for haste and intensity were acquired seismic hazard maps of Kosovo. It is worth to emphasize that isolines in these maps represent only bordering lines with certain haste and intensity of soil, which does not represent at the same time the borders of any seismic source. Restrictive lines are characterized with certain numerical values for different periods of repetition.
2.3.2. Maps presenting spread of Earthquake Intensity

Besides seismic hazard maps of Kosovo, in which the oscillation of the land are expressed through the maximum expected haste in %g, as laid above, there are also compiled seismic hazard maps, in which soil shaking is presented through maximal expected intensity in forefront. These maps present distribution of the maximum intensity expected at the forefront in the territory of Kosovo for repetition earthquake periods of 100 and 500 years. The input data used for compilation of these maps are the data reflected on maps of haste, with probability of 63%. Distribution of the maximum expected intensity of earthquakes for the repetition period of 100 years is shown in Fig.2.3.

![Maximum Seismic Intensity Map](image)

*Fig. 2.3. Propagation of the maximum seismic intensity for territory of Kosova, Encore period 100 year*

From the map of maximum expected intensity in Kosovo, the repetition period of 500 years, that should be used in designing massive structures in cities and villages, as recommended in Eurocode 8, show that there are only two Zones, namely zone Ferizaj -Viti-Gjilane and Kopaonik zone where future earthquakes may occur with the expected maximum intensity IX
degrees MSK-64, while in all the rest of the territory of Kosovo are expected earthquakes with maximum anticipated intensity VIII degrees MSK-64.

Fig. 2.4. Propagation of the maximum seismic intensity for territory of Kosovo, Encore period 500 year

Map of seismic sources has served as the basis for calculation of seismic risk for different periods of time. Comparison of results obtained shows that the dimensions of the seismic sources have major impact on the end results – maximum soil haste. It should be noted that with the help of the map of seismic sources, compiled taking into consideration the criteria of geological seismity of Kosovo, effort was made to find more objective results for seismic risk in Kosovo.

The values of maximum projected haste, for certain regions and locations, for the repetition period of 500 years, should be read in relevant maps, as these haste values, as envisioned in Eurokod 8, are available to consider during design and construction of the massive structures common in cities and villages.
Fig. 2.5. Propagation of the maximum accelerations for territory of Kosova, Encore period 500 year

Fig. 2.6. Map of the Tectonic Disjunction of territory Kosova.

Thus, for average ground, haste values, that should be taken into consideration during design and construction for several major residential centers are as follows (see Fig. 27 - Map of
seismic hazard in Kosovo, for average ground, repetition period 500 years): Gjakova and Peja – 0.25 g, Prizren – 0.20 g, Kaçanik - 0.25-0.30 g, Gjilane - 0.20-0.25 g, Pristina Podujeva – 0.15 g, Mitrovica - 0.15-0.20 g.

2.4. Description of the selected three Earthquake Records, Used for the Present Study.

Applying the realistically defined mathematical model of the building and the defined time history of acceleration as input excitation, the structural response can be defined by using a computer program for nonlinear dynamic analysis [Dum. 2.04]. Based on the obtained response and using a corresponding program, it is possible to define damage level of the structure to that excitation level.

However, it is well known that the intensity of dynamic response of structures depends strictly on the frequency content of the input excitation.

For the purpose of a more realistically presentation of the dominant frequency range of the expected earthquake motions in the considered case, it is necessary that one should have very good knowledge on the regional and especially the local soil conditions. The local soil the final modification of the input seismic wave. There are two solutions for solving this problem, i.e., making a proper selection of the input excitation. The first solution is to perform detailed, additional experimental and analytical investigations for exact quantification of the effects of the local soil conditions. The second solution is to select an acceleration time history recorded at soil of similar characteristics on the basic of the known tectonics of the site and evaluation of the local soil conditions. In the considered case, a set of three earthquake round, involving a wide frequency range was selected for the proposed study [St. 90]. The following representative excitations were selected.

1. Ulcinj-Albatros N-S 1979 (Montenegro);

2. El Centro SOOE 1940 (USA); and,

3. Pristina Synthetics (Artificial).

Large number of individual non-linear seismic response analysis of the selected buildings (in total 15 buildingsx66 analysis = 990 non-linear analysis, case) under real earthquake excitations is realized with computed program NORA-2009 (Nonlinear Response Analysis, program) developed for such special study purposes. For each building element we have
created the idealized hysteretic model which is harmonized in the generalized analytical model developed for integral buildings separately for both principal directions x and y.

Fig. 2.7. shows acceleration records for three earthquake motions that are used for non-linear analysis. In order to implement the dynamic analysis we have adopted gradation of maximal purposes acceleration (PGA - Peak Ground Acceleration) in eleven different levels from 0.025g to 0.50g. Considering this we have conducted eleven non-linear dynamic response analysis for each earthquake, and having three earthquakes and two directions (x and y) we head to realize 66 analysis for each building.

1) Ulqin – Albatros earthquake

2) El – Centro Earthquake, SOOE component
3) Pristina synthetic – artificial earthquake

Fig. 2.7. Acceleration diagrams for earthquakes used in the analysis: (1) Ulqin – Albatros 1979, (2) El-Centro 1940 and (3) Pristina synthetic – artificial earthquake.
Chapter 3

THEORETICAL ANALYSIS OF SEISMIC VULNERABILITY AND DAMAGE PROPAGATION OF THE SELECTED 15 REPRESENTATIVE MASONRY BUILDINGS IN PRISHTINA

3.1. Seismic Vulnerability Analysis of Building No. 1 in Longitudinal Direction-x and Transversal Direction-y

3.1.1. Description of basic characteristics of the building structural system

FCA Building – Architecture is constructed in 1918. Initially it was used as military barracks, but later, during 1960’s was used as university building – Technical Faculty. Today it is property of Prishtina University and is used by the Faculty of Civil Engineering and Architecture – Architectural department.

![Building No. 1: Architectural department of the Faculty of Civil Engineering and Architecture](image)

The building was renovated several times in the past (latest renovation took place in 2002). At this time the original clay tile roof cover was replaced with sandwich type metal sheets.

![First floor plan, identical to ground floor plan](image)

*Fig. 3.1.1. Building No. 1: Architectural department of the Faculty of Civil Engineering and Architecture*

*Fig. 3.1.2. Building No. 1: First floor plan, identical to ground floor plan*
Floor plan of the building with dimensions (46.95 x 11.50)m, shown in Fig. 3.1.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are three linear load baring walls, 5.75m apart, and on the latitudinal direction, along “y” axis, there are seven linear walls with different distances among each other. The building consists of ground floor (3.52m high) + first floor (3.54m high) + attic. Connection points of load baring walls on two directions are strengthened with reinforced concrete non-structural columns. All structural walls are bricked with solid clay bricks with dimensions 25x12x6 cm joined with mortar and have a constant width of 38cm. There are partition walls as non-structural elements on each floor. Structural wall sections with parapets and spandrels are treated as non-structural elements. Mezzanine construction is massive reinforced concrete slab 18cm thick that rests on both directions on 38x38cm concrete beams. Roof structure, monolith timber trusses rest on longitudinal walls along “x” axis. Timber purlins rest on main trusses and hold the corrugated sheet roof cover.

3.1.2. Seismic Vulnerability Analysis of Building No. 1 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 1 in Longitudinal Direction-x and Structural Dynamic Characteristics

Fig. 3.1.3 Building No.1: Part of Individual Wall Segments C-C Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Mathematical model used for vulnerability analysis of Building No. 1 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements [Br.Me UK].
The formulated non-linear mathematical model is defined as “shear type”, formulated based on systematic implementation of “multi componential” concept. In Fig. 3.1.4, shown is the formulated mathematical model of the building consisting of two concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively [Ch.Dum 08].

In Fig. 3.1.5, and Fig. 3.1.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 1 for Longitudinal Direction-x**

Representative force-displacement envelope curves for structural and non-structural elements for all building stories are defined with three characteristic points C (cracking point), Y (yielding point) and U (ultimate point). The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in table. 3.1.1. To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.1.7.
Tab. 3.1.1 Displacement Envelope Curve for initial stiffness $K_0$ and Envelope Curves for points C, Y and U

<table>
<thead>
<tr>
<th>Storey</th>
<th>Elements</th>
<th>Initial stiffness $K_0$ [kN/cm]</th>
<th>Cracking point deformation and force</th>
<th>Yielding point deformation and force</th>
<th>Ultimate point deformation and force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dc (cm)</td>
<td>Fc (kN)</td>
<td>Dy (cm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>S.E. 3</td>
<td>1773319</td>
<td>0.147</td>
<td>2608.87</td>
<td>0.942</td>
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<tr>
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<td>132.53</td>
<td>0.444</td>
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<td>2545371</td>
<td>0.079</td>
<td>2025.87</td>
<td>0.726</td>
</tr>
<tr>
<td></td>
<td>N.E. 2</td>
<td>311139</td>
<td>0.022</td>
<td>68.56</td>
<td>0.201</td>
</tr>
</tbody>
</table>

![Figure 3.1.7](image.png)

**Fig. 3.1.7 Envelope curves for structural behavior.**

c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 1 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

The computed maximum or “Pick-Response” relative storey displacements of Building No. 1 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.1.2. In the mentioned table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Prishtina Synthetic earthquake record (EQR).

Tab. 3.1.2. Relative displacements in building storeys, gained from the non-linear dynamic response analysis formed in the “multi componential” analytical model

<table>
<thead>
<tr>
<th>NP</th>
<th>Eqi - Ulcinj – Albatros N-S</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Real Displacement – Direction-x (cm)</td>
</tr>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>1</td>
<td>0.079</td>
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<tr>
<td>2</td>
<td>0.058</td>
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</table>
### EQI – El-Centro

<table>
<thead>
<tr>
<th>NP</th>
<th>Real Displacement – Direction-x (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>1</td>
<td>0.118</td>
</tr>
<tr>
<td>2</td>
<td>0.085</td>
</tr>
</tbody>
</table>

### EQI – Prishtina Synthetic

<table>
<thead>
<tr>
<th>NP</th>
<th>Real Displacement – Direction-x (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>1</td>
<td>0.071</td>
</tr>
<tr>
<td>2</td>
<td>0.052</td>
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</tbody>
</table>

**Fig. 3.1.8. Computed Pick Relative Storey Displacements of Building No. 1 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x**

**Fig. 3.1.9. Computed Pick Relative Storey Displacements of Building No. 1 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x**
Fig 3.1.10. Computed Pick Relative Storey Displacements of Building No. 1 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 1 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form.

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 1 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.1.8., Fig. 3.1.9., and Fig. 3.1.10., respectively.

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 1 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 1 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.1.3. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.1.3. Computed Maximum (“Peak-Response”) Inter-story drift (ISD) of Building No. 1 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>NP</th>
<th>Index of inter-storey drift, displacement (%) – Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05g</td>
<td>0.10g</td>
</tr>
<tr>
<td>1</td>
<td>0.224</td>
<td>0.455</td>
</tr>
<tr>
<td>2</td>
<td>0.159</td>
<td>0.324</td>
</tr>
</tbody>
</table>
e) The predicted Seismic Vulnerability Functions of Building No. 1, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The computed maximum earthquake response basic and/or representative parameters, namely earthquake inter-story drift demands (ISD), from all of these analyses are presently considered as basic indicating data for further evaluation of building vulnerability characteristics and development of resulting average vulnerability functions.

Basic relations established between the increasing input earthquake intensity parameter (PGA) and the resulting inter-story drifts (ISD), based on data for all stories and all three earthquake motion types are presented in separate tables. This complete set of the established ISD-PGA basic relations, along with the adopted damage criteria and specified respective element specific loss functions (as described in previous, part I, chapter 3) are further implemented to determine the expected levels of building specific loss as well as to derive theoretical vulnerability functions of building structural (SE) and nonstructural (NE) elements for the increasing intensities of seismic loads.

The predicted direct analytical vulnerability functions of the integral Building No. 1 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.1.11, Fig. 3.1.12, Fig. 3.1.13 and Fig. 3.1.14.) [Dum. 00], [Dum. 02]. In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural
and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.1.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 1 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.1.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 1 in Direction-x Under El-Centro earthquake

Fig. 3.1.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 1 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.1.3. Seismic Vulnerability Analysis of Building No. 1 Transversal Direction-y

a) **Formulation of Non-Linear Mathematical Model of Building No. 1 in Transversal Direction-y and Structural Dynamic Characteristics**

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in transverse y-direction. The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 1 in transversal direction, Fig. 3.1.15.

![Wall frame in 1-1 & 6-6 axis](image)

**Fig. 3.1.15 Building No 1: Part of Individual Wall Segments 1-1 and 6-6, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-y**

The formulated non-linear mathematical model which is used for vulnerability analysis of Building No. 1 in direction-y includes separately non-linear behavior characteristics of
structural and non-structural elements consequently in all existing building stories [Mi.Ri.Po.Zd. 94].

In fact, for this study purposes, the formulated non-linear mathematical model is defined as “shear type”, formulated based on systematic implementation of “multi component” concept. In Fig. 3.1.16, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.1.17, and Fig. 3.1.18, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

### b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 1 for Transversal Direction-\(y\)

Representative force-displacement envelope curves for structural and non-structural elements for all building stories are defined with three characteristic points C (cracking point), Y (yielding point) and U (ultimate point) [Dum. 03]. The calculated initial stiffness \(K_0\), and respective force and displacement values for above specified points are presented in table. 3.1.4. To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.1.19.
**Figure. 3.1.19 Envelope curves for structural behavior.**

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 1 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” relative storey displacements of Building No. 1 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.1.5. In the mentioned table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Prishtina Synthetic earthquake record (EQR).

**Tab. 3.1.5. Computed Maximum (“Pick-Response”) Relative Storey Displacements of Building No. 1 Under Different Earthquake Intensity Levels in Transversal Direction-y**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Elements</th>
<th>Initial stiffness K₀ [kN/cm]</th>
<th>Cracking point deformation and force</th>
<th>Yielding point deformation and force</th>
<th>Ultimate point deformation and force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dc (cm)</td>
<td>Fe (kN)</td>
<td>Dy (cm)</td>
</tr>
<tr>
<td>1</td>
<td>S.E. 3</td>
<td>1280460</td>
<td>0.074</td>
<td>949.67</td>
<td>0.676</td>
</tr>
<tr>
<td></td>
<td>N.E. 4</td>
<td>41912</td>
<td>0.209</td>
<td>87.58</td>
<td>0.705</td>
</tr>
<tr>
<td>2</td>
<td>S.E. 1</td>
<td>1315585</td>
<td>0.043</td>
<td>561.28</td>
<td>0.389</td>
</tr>
<tr>
<td></td>
<td>N.E. 2</td>
<td>903071</td>
<td>0.039</td>
<td>348.30</td>
<td>0.352</td>
</tr>
</tbody>
</table>

**Tab. 3.1.4. Computed Non-Linear Force-Displacement Envelope Curves for Structural and Non-Structural Elements of Building No. 1 for transversal direction-y**

<table>
<thead>
<tr>
<th>Storey</th>
<th>Elements</th>
<th>Initial stiffness K₀ [kN/cm]</th>
<th>Cracking point deformation and force</th>
<th>Yielding point deformation and force</th>
<th>Ultimate point deformation and force</th>
</tr>
</thead>
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<td>Dc (cm)</td>
<td>Fe (kN)</td>
<td>Dy (cm)</td>
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<td>0.074</td>
<td>949.67</td>
<td>0.676</td>
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<td>0.209</td>
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<td>N.E. 2</td>
<td>903071</td>
<td>0.039</td>
<td>348.30</td>
<td>0.352</td>
</tr>
</tbody>
</table>
Fig 3.1.20. Computed Pick Relative Storey Displacements of Building No. 1 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y

Fig 3.1.21. Computed Pick Relative Storey Displacements of Building No. 1 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y
To obtain full evidence in the most important response parameters of Building No. 1 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 1 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.1.20., Fig. 3.1.21., and Fig. 3.1.22., respectively.

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 1 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 1 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.1.6. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.1.6. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 1 Under Different Earthquake Intensity Levels in Transversal Direction-y

<table>
<thead>
<tr>
<th>EQR-1: Ulcinj – Albatros N-S</th>
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<tbody>
<tr>
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<tr>
<td>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</td>
</tr>
<tr>
<td>0.025g</td>
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</table>
**EQR-2: El-Centro**

<table>
<thead>
<tr>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
<th>0.50g</th>
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<tbody>
<tr>
<td>0.025g</td>
<td>0.369</td>
<td>0.918</td>
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<td>3.503</td>
<td>5.304</td>
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<td>2.923</td>
<td>4.646</td>
<td>6.692</td>
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<td>11.585</td>
<td>13.981</td>
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<td>17.901</td>
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**EQR-3: Prishtina Synthetic**

<table>
<thead>
<tr>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
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<tbody>
<tr>
<td>0.025g</td>
<td>0.281</td>
<td>0.781</td>
<td>2.531</td>
<td>4.358</td>
<td>4.520</td>
<td>3.324</td>
<td>1.429</td>
<td>1.483</td>
<td>1.878</td>
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<td>2.602</td>
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<tr>
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<td>1.970</td>
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<td>3.843</td>
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<td>0.984</td>
<td>1.190</td>
<td>1.486</td>
<td>1.753</td>
<td>2.049</td>
</tr>
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</table>

e) The Predicted Seismic Vulnerability Functions of Building N0. 1, Under The Effect of Three Selected Earthquakes in Transversal Direction-y

The computed maximum earthquake response basic and/or representative parameters, namely earthquake inter-story drift demands (ISD), from all of these analyses are presently considered as basic indicating data for further evaluation of building vulnerability characteristics and development of resulting average vulnerability functions. Basic relations established between the increasing input earthquake intensity parameter (PGA) and the resulting inter-story drifts (ISD), based on data for all stories and all three earthquake motion types are presented in separate tables. This complete set of the established ISD-PGA basic relations, along with the adopted damage criteria and specified respective element specific loss functions (as described in previous chapters) are further implemented to determine the expected levels of building specific loss as well as to derive theoretical vulnerability functions of building structural (SE) and nonstructural (NE) elements for the increasing intensities of seismic loads.

The predicted direct analytical vulnerability functions of the integral Building N0 1 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.1.23., Fig. 3.1.24. and Fig. 3.1.25.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.1.23. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No.1. in Direction-y Under Ulcinj- Albatros earthquake

Fig. 3.1.24. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No.1. in Direction-y Under El-Centro earthquake

Fig. 3.1.25. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No.1. in Direction-y Under Prishtina Synthetic earthquake
3.1.4. Comparative Presentation of Damage Propagation Through SE&NE of Masonry Building No. 1. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results on the building vulnerability under three Earthquakes (EQ=1, Ulcinj-Albatros, EQ-2, El-Centro & EQ-3, Prishtina Synthetic), behaviour of SE and NE can be described as follows:

1. On the longitudinal direction x, NE collapse prior to SE, and on the other direction, is the opposite – SE collapse before NE, [Hi.Sh.Gh 04].

2. In building No.1, regardless of its overall stiffness, collapse takes place always on the first floor and simultaneously on SE and NE.

Fig. 3.1.26. Comparative Presentation of Cumulative Seismic Vulnerability Functions Masonry Building No.1. in Direction- \( y \) For Three Considered Earthquakes
<table>
<thead>
<tr>
<th>EQ=1 (B1x)</th>
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<tr>
<td>EQI9 = 0.40G</td>
<td>EQI10 = 0.45G</td>
<td>EQI11 = 0.50G</td>
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<tr>
<td><img src="image10" alt="Diagram" /></td>
<td><img src="image11" alt="Diagram" /></td>
<td><img src="image12" alt="Diagram" /></td>
</tr>
</tbody>
</table>

*Figure 3.1.27. Damage Propagation Troudh SE & NE of Masonry Building No. 1. for Ulcinj - Albatros Earthquake in Longitudinal Direction-x*
<table>
<thead>
<tr>
<th>EQ = 2 (B1x)</th>
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<th>EQI2 = 0.05G</th>
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*Figure 3.1.28: Damage Propagation Through SE & NE of Masonry Building No. 1 for El-Centro Earthquake in Longitudinal Direction-x*
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<thead>
<tr>
<th>EQI1</th>
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<th>EQI3</th>
<th>EQI4</th>
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<th>EQI7</th>
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<td>S.E.</td>
<td>S.E.</td>
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Figure 3.1.29. Damage Propagation Through SE & NE of Masonry Building No. 1 for Prishtina synthetic Earthquake in Longitudinal Direction-x.
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</tbody>
</table>

*Figure 3.1.30. Damage Propagation Trough SE & NE of Masonry Building No. 1 for Ulcinj-Albatros Earthquake in Transversal Direction-y*
<table>
<thead>
<tr>
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*Figure 3.1.31. Damage Propagation Through SE & NE of Masonry Building No. 1, for El-Centro Earthquake in Transversal Direction-y*
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*Figure 3.1.32. Damage Propagation Through SE & NE of Masonry Building No. 1 for Prishtina synthetic Earthquake in Transversal Direction-y*
3.1.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 1. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 1, the following general conclusions can be derived:

1. Building collapse happens on PGA = 0.15g in referent direction Y. This is because of the present different story stiffness and storey strength for directions X and Y. Stiffness and strength is greater in referent direction X, therefore collapse of the building happens faster in the direction-y with the lower stiffness and strength.

2. From the comparison of results for three earthquakes used in the analysis, it can be easily noticed that the Pristina synthetic-artificial earthquake is with a lower intensity compared to the other two earthquakes. Despite different intensities of earthquake strikes, the building collapses on the same values of PGA. From this we can say that the building does not bare large displacements – it collapses for considerable low displacement values.

3. It is computed that in all cases of three earthquakes, the collapse takes place due to induced progressive damage and collapse of the first story first. As a result it will affect other elements producing in last stage the total or partial collapse of the building.

4. Total loss is 3.88% from the total building cost in the collapse moment in case of Pristina synthetic-artificial earthquake acting in referent direction-y, meanwhile structural elements take part in this loss with 2.28% and non-structural elements with 1.43%. These specific mode of structural collapse with produced low damage and cost of loss in the pre-collapse moment show that these buildings do not have sufficient capacity to absorb earthquake strikes, especially the ones with the epicenter close to the building or so called strong local earthquakes.
3.2. Seismic Vulnerability Analysis of Building No. 2 in Longitudinal Direction-x and Transversal Direction-y

3.2.1. Description of basic characteristics of the building structural system

During the whole period, the building was used as collective apartment building. Following the public building privatization, these buildings now are privately-owned by occupants. The building was renovated a few times in the past. It has the basement, ground and first floor. It is important to mention that in these buildings, ground floor is converted into business areas.

![Building No. 2: Residential Building No. 2, Fehmi Agani str.](image)

During conversion large portions of partition walls were removed with the intention to have large internal areas, but also braking large openings on load-baring walls, thus changing the structural system. Assessment is made taking into consideration present condition of the building.

![Floor plan of the building with dimensions (20.45 x 8.75)m](image)

Floor plan of the building with dimensions (20.45 x 8.75)m, shown in Fig. 3.2.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.
On the longitudinal direction, along “x” axis, there are three linear load baring walls, 4.30m and 4.45 apart, and on the latitudinal direction, along “y” axis, there are seven linear walls with different distances among each other (4.47, 4.83)m. The building consists of basement floor (2.67m high) ground and first floor (3.00m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). All structural walls are bricked with solid clay bricks with dimensions 25x12x6 cm joined with mortar and have a constant width of 38cm. Structural wall sections with parapets and spandrels are treated as non-structural elements.

3.2.2. Seismic Vulnerability Analysis of Building No. 2 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 2 in Longitudinal Direction-x and Structural Dynamic Characteristics

Fig. 3.2.3 Building No. 2: Part of Individual Wall Segments A-A, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Mathematical model used for vulnerability analysis of Building No. 2 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements [On.Gu.Ri. 98].

Fig. 3.2.4. Building No. 2: Non-Linear MC Model for Direction-x

Fig. 3.2.5. Building No. 2: Mode Shape-1, Direction-x; \( T_{1x} = 0.283 \) sec

Fig. 3.2.6. Building No. 2: Mode Shape-2, Direction-x; \( T_{2x} = 0.093 \) sec
In Fig. 3.2.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively [Ri. 88].

In Fig. 3.2.5, and Fig. 3.2.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 2 for Longitudinal Direction-x

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.2.7.

![Figure. 3.2.7 Envelope curves for structural behavior.](image)

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 2 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 2 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 2 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.2.8., Fig. 3.2.9., and Fig. 3.2.10., respectively.
Fig. 3.2.8. Computed Pick Relative Storey Displacements of Building No. 2 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig. 3.2.9. Computed Pick Relative Storey Displacements of Building No. 2 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig. 3.2.10. Computed Pick Relative Storey Displacements of Building No. 2 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 2 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 2 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.2.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.2.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 2 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<tbody>
<tr>
<td>0.025g</td>
<td>0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.097 0.183 0.473 0.990 1.473 2.020 2.847 4.313 5.500 7.307 8.490</td>
</tr>
<tr>
<td>1</td>
<td>0.449 0.933 2.232 3.906 5.558 8.094 12.228 16.442 19.603 23.592 26.322</td>
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</table>

<table>
<thead>
<tr>
<th>EQI - El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<tbody>
<tr>
<td>0.025g</td>
<td>0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
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<tr>
<td>3</td>
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</tr>
<tr>
<td>2</td>
<td>0.347 0.720 1.437 2.660 3.747 5.083 7.280 9.973 12.977 14.957 17.307</td>
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<tr>
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<td>0.453 0.933 1.891 3.416 4.985 6.517 9.333 13.075 17.199 20.056 23.449</td>
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</table>

<table>
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<tr>
<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
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<td>0.025g</td>
<td>0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.063 0.137 0.257 0.673 1.137 1.557 1.973 2.357 3.350 4.323 5.693</td>
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<tr>
<td>2</td>
<td>0.243 0.513 1.163 2.457 3.777 5.237 6.867 8.767 10.963 12.840 15.730</td>
</tr>
<tr>
<td>1</td>
<td>0.318 0.674 1.506 3.112 4.914 6.899 9.067 11.622 14.581 17.363 21.498</td>
</tr>
</tbody>
</table>

e) The predicted Seismic Vulnerability Functions of Building No. 2, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

Basic relations established between the increasing input earthquake intensity parameter (PGA) and the resulting inter-story drifts (ISD), based on data for all stories and all three earthquake motion types are presented in separate tables. This complete set of the established ISD-PGA basic relations, along with the adopted damage criteria and specified respective element specific loss functions are further implemented to determine the expected levels of building specific loss as well as to derive theoretical vulnerability functions of building structural (SE) and nonstructural (NE) elements for the increasing intensities of seismic loads.

The predicted direct analytical vulnerability functions of the integral Building No. 2 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.2.11, Fig. 3.2.12, and Fig. 3.2.13.). In this case, based on the gathered statistical information on participation of
structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building [Tr. Mi, Ol 05].

Fig. 3.2.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 2 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.2.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 2 in Direction-x Under El-Centro earthquake
3.2.3. Seismic Vulnerability Analysis of Building No. 2 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 2 in Transversal Direction-y and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in transverse y-direction. The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 2 in transversal direction, Fig. 3.2.14.

Mathematical model used for vulnerability analysis of Building No. 2 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.
In Fig. 3.2.15, shown is the formulated mathematical model of the building consisting of three concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.2.16, and Fig. 3.2.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 2 for Transversal Direction-\( \text{y} \)**

The calculated initial stiffness \( K_0 \), and respective force and displacement values for above specified points are presented in graphical form in Fig. 3.2.18.

---

**Figure. 3.2.18, Envelope curves for structural behavior.**
c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 2 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)

To obtain full evidence in the most important response parameters of Building No. 2 in transversal \(y\)-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 2 in transverse \(y\)-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.2.19., Fig. 3.2.20., and Fig. 3.2.21., respectively.

![Graph](image1)

**Fig 3.2.19. Computed Pick Relative Storey Displacements of Building No. 2 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-\(y\)**

![Graph](image2)

**Fig 3.2.20. Computed Pick Relative Storey Displacements of Building NO. 2 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-\(y\)**
Fig 3.2.21. Computed Pick Relative Storey Displacements of Building NO. 2 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 2 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 2 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.2.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.2.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 2 Under Different Earthquake Intensity Levels in Transversal Direction-y

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
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<td></td>
<td>0.025g</td>
</tr>
<tr>
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<table>
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<th>EQI – El-Centro</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
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<tr>
<td>1</td>
<td>0.345</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>EQI – Pristina Synthetic</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>3</td>
<td>0.080</td>
</tr>
<tr>
<td>2</td>
<td>0.170</td>
</tr>
</tbody>
</table>
e) The Predicted Seismic Vulnerability Functions of Building No. 2, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 2 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.2.22., Fig. 3.2.23. and Fig. 3.2.24.) [We.El.Br. 04]. In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.2.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 2. in Direction-y Under Ulcinj- Albatros earthquake

Fig. 3.2.23 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 2. in Direction-y Under El-Centro earthquake
Fig. 3.2.24  The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 2. in Direction-y Under Prishtina Synthetic earthquake

3.2.4. Comparative Presentation of Damage Propagation Trough SE&NE of Masonry Building No. 2. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results on the building vulnerability under the impact of three earthquakes (EQ-1, Ulcinj-Albatros, EQ-2 El-Centro, and EQ-3 Pristina Synthetic), for SE and NE it can be concluded that:

1. Collapse of SE for longitudinal direction x and transversal direction y takes place simultaneously for the PGA = 0.25g;
2. For Building No.2, regardless of its stiffness, collapse always takes place on the second level simultaneously in SE and NE;
3. Level of damage in the collapse peak is higher in Ne (4.24%) compared to SE (1.69%).

[Graph showing predicted cumulative seismic vulnerability function]
3.2.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 2. 
Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 2, the following general conclusions can be derived:

(1) Building collapse happens on PGA = 0.25g in longitudinal x and transversal y direction. This is because of the very similar story stiffness for different directions x and y.

(2) For all three earthquake impacts on Building No.2, as well as for both orthogonal directions x and y, collapse takes place on the second level. The reason for the simultaneous collapse of SE and NE at this level stands in the fact that on the first level walls have larger stiffness (width of the walls is 45cm) and on the other levels walls are thinner (38cm wide), and overall load on the second level is larger than on the third (last) level.

(3) Total loss is 5.94% from the total building cost in the collapse moment in case of Pristina synthetic-artificial earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 1.69% and non-structural elements with 4.24%. Those specific mode of structural collapse with produced low damage and cost of loss in the pre-collapse moment show that these buildings do not have sufficient capacity to absorb earthquake strikes, especially the ones with the epicenter close to the building or so called strong local earthquakes.
3.3. Seismic Vulnerability Analysis of Building No. 3 in Longitudinal Direction-x and Transversal Direction-y

3.3.1. Description of basic characteristics of the building structural system

Building serves as an apartment building for collective housing. Following privatization of buildings, these buildings are privately owned by occupants. The building was renovated a few times in the past. It consists of Basement, ground and two floors. It is important to mention that in these buildings, ground floor is converted into business areas.

Fig. 3.3.1. Building No. 3: Residential Building No. 3, Migjeni str.

During conversion large portions of partition walls were removed with the intention to have large internal areas, but also braking large openings on load-baring walls, thus changing the structural system. Assessment is made taking into consideration present condition of the building.

Fig. 3.3.2. Building No. 3: Floor plan

Floor plan of the building with dimensions (18.53 x 11.74)m, shown in Fig. 3.3.2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.
On the longitudinal direction, along “x” axis, there are four linear load baring walls, 4.62m, 1.56 and 4.65 apart, and on the transversal direction - y, there are six linear walls with different distances among each other (4.25, 3.05)m. The building consists of basement floor (2.38m high) ground and for all stories (3 x 3.40m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). Load-baring walls are made of stones and bricks. Walls at the basement level are with stones and are 45cm thick, and brick walls with a thickness of 38cm are on all other stories. Unit brick dimensions are 25x12x6.5cm and are usually bricked with cement plaster. Walls are properly interconnected during bricking.

3.3.2. Seismic Vulnerability Analysis of Building No. 3 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 3 in Longitudinal Direction-x and Structural Dynamic Characteristics

Mathematical model used for vulnerability analysis of Building No. 3 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Diagram](image-url)

**Fig. 3.3.3 Building No. 3: Part of Individual Wall Segments A-A, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x**

Mathematical model used for vulnerability analysis of Building No. 3 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.
In Fig. 3.3.4, shown is the formulated mathematical model of the building consisting of four concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.3.5, and Fig. 3.3.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 3 for Longitudinal Direction-x

The initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in graphical form in Fig. 3.3.7.

As seen in Fig. 3.3.7, it is special for this building that participation of SE is much larger compared to NE. This leads to the fact that stiffness of NE is much lower, therefore their participation in the absorption of horizontal earthquake forces is low.

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 3 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 3 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form [Xie. 05].

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 3 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.3.8., Fig. 3.3.9., and Fig 3.3.10., respectively.
Fig 3.3.8. Computed Pick Relative Storey Displacements of Building No. 3 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.3.9. Computed Pick Relative Storey Displacements of Building No. 3 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.3.10. Computed Pick Relative Storey Displacements of Building No. 3 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 3 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 3 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.3.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.3.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 3 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
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<td></td>
</tr>
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</tr>
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<th>EQI – El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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</thead>
<tbody>
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<table>
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</table>

e) The predicted Seismic Vulnerability Functions of Building No. 3, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 3 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.3.11, Fig. 3.3.12, and Fig. 3.3.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this
particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

**Fig. 3.3.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 3 in Direction-x Under Ulqin – Albatros earthquake**

**Fig. 3.3.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 3 in Direction-x Under El-Centro earthquake**

**Fig. 3.3.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 3 in Direction-x Under Prishtina Synthetic – artificial Earthquake**
3.3.3. Seismic Vulnerability Analysis of Building No. 3 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 3 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 3 in transversal direction, Fig. 3.3.14.

Mathematical model used for vulnerability analysis of Building No. 3 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.3.15, shown is the formulated mathematical model of the building consisting of four concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.3.16, and Fig. 3.3.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) **Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 3 for Transversal Direction-\(y\)**

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.3.18.

---

![Figure. 3.3.18, Envelope curves for structural behavior.](image)

---

c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 3 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)**

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 3 in transverse \(y\)-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.3.19., Fig. 3.3.20., and Fig 3.3.21., respectively.

---

![Figure. 3.2.19. Computed Pick Relative Storey Displacements of Building No. 3 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-\(y\)](image)
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 3 Under Different Earthquake Intensity Levels in Transversal Direction-\( y \)

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 3 under different earthquake intensity levels in transversal direction-\( y \) are presented in Table 3.3.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.3.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 3 Under Different Earthquake Intensity Levels in Transversal Direction-\( y \)**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-( y )</th>
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<td>4</td>
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### EQI – El-Centro

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<th>0.10g</th>
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### EQI – Prishtina Synthetic

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<td>13.25</td>
<td>10.50</td>
<td>15.22</td>
<td>18.15</td>
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</table>

### e) The Predicted Seismic Vulnerability Functions of Building No. 3, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 3 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.3.22., Fig. 3.3.23. and Fig. 3.3.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Graph](image_url)

**Fig. 3.3.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No.3. in Direction-y Under Ulcinj- Albatros earthquake**
3.3.4. Comparative Presentation of Damage Propagation Through SE & NE of Masonry Building No. 3. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results for building vulnerability under the three earthquake impacts (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro & EQ-3 Prishtina Synthetic), we can conclude the following on the behavior of SE and NE within the structure:

1. For the longitudinal direction x, SE and NE collapse for the same PGA values (PGA=0.15g), in the case of El-Centro and Pristina Synthetic earthquakes;
2. Regardless of the overall stiffness of the whole building, it is interesting to observe that the collapse of SE and NE always takes place simultaneously on two levels (second and third level);

Damage degree in the collapse peak for NE is larger (3.86%) compared to SE (2.93%), which are calculated for the impact of Pristina Synthetic earthquake.
<table>
<thead>
<tr>
<th>EQ = 1 (G0)</th>
<th>EQ1 = 0.05G</th>
<th>EQ2 = 0.05G</th>
<th>EQ3 = 0.10G</th>
<th>EQ4 = 0.15G</th>
<th>EQ5 = 0.20G</th>
<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
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</table>

Damage Propagation Troudh SE & NE of Masonry Building No. 3. for El-Centro Earthquake in Transversal Direction-y

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G
3.3.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 3. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 3, the following general conclusions can be derived:

(1) Figures 3.3.7 and 3.3.8 show that, for capacity diagrams of SE and NE, for the longitudinal direction x and transversal y, participation of SE in the structure capacity is higher that NE. Also in these diagrams we can see stiffness variation in two orthogonal directions, where building stiffness along direction y is much higher than along the other direction.

(2) Displacements of separate structural elements on different levels are different and indicative. SE of the top level have much smaller displacements compared to SE of the lowest level. This principle leads to the collapse of the elements floor-by-floor.

(3) Building collapse happens on PGA = 0.15g in longitudinal x-direction. This is because of the present different story stiffness and storey strength for directions X and Y. Stiffness and strength is greater in referent transversal direction y, therefore collapse of the building happens faster in the longitudinal x-direction with the lower stiffness and strength.

(4) In two cases of earthquake impact for building No.3, along the longitudinal direction x, collapse takes place on the second and third level simultaneously. The reason for this is that walls on the first level are stiffer (they are 45cm thick) and on the other levels they are thinner (38cm thick). Another reason for simultaneous collapse is in the weakening of the structural walls that occurred later with new openings. (5)

(5) Total loss is 6.79% from the total building cost in the collapse moment in case of Pristina synthetic-artificial earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.93% and non-structural elements with 3.86%.
3.4. Seismic Vulnerability Analysis of Building No. 4 in Longitudinal Direction-x and Transversal Direction-y

3.4.1. Description of basic characteristics of the building structural system

The building was always used as a public school. Immediately after construction, it was used as a political school, but today it serves as a public professional secondary school owned by Prishtina Municipality. It has been renovated several times in the past, but without any structural changes to it. It consists of Basement, ground and first floor. Ground and first floor consist of classrooms and other accompanying areas, and basement is used for storage.

![Building No. 4: Secondary School “7 November” No. 4, Hile Mosi str.](image)

**Fig. 3.4.1. Building No. 4: Secondary School “7 November” No. 4, Hile Mosi str.**

Floor plan of the building with dimensions (26.0 x 14.60)m, shown in Fig. 3.4.2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.

**Fig. 3.4.2. Building No. 4, Floor plan**

Structural system is a system with load-baring walls on both orthogonal directions. Basement walls are with stone and have a thickness of 45cm. Other load baring walls are with bricks.
and are 38cm thick. Brick dimensions are 25x12x6.5cm and are bricked with cement plaster. On the longitudinal direction, along “x” axis, there are five linear load bearing walls, with different distance, and on the transversal direction - y, there are four linear walls with different distances among each other (7.95, 9.65, 7.95)m. The building consists of basement floor (2.98m high) ground and first floor (3.76m high). Connection points of load bearing walls on two directions are strengthened with our self. Load-bearing walls are made of stones and bricks. Walls at the basement level are with stones and are 45cm thick, and brick walls with a thickness of 38cm are on all other stories. Unit brick dimensions are 25x12x6.5cm and are usually bricked with cement plaster. Walls are properly interconnected during bricking.

### 3.4.2. Seismic Vulnerability Analysis of Building No. 4 Longitudinal Direction-x

#### a) Formulation of Non-Linear Mathematical Model of Building No. 4 in Longitudinal Direction-x and Structural Dynamic Characteristics

![Mathematical model](image)

Mathematical model used for vulnerability analysis of Building No. 4 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Mode Shapes](image)

In Fig. 3.4.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear...
stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.4.5, and Fig. 3.4.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 4 for Longitudinal Direction-x**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in graphical form in Fig. 3.4.7.

![Figure 3.3.7 Envelope curves for structural behavior.](image)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 4 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 4 in longitudinal x-direction, the computed maximum or "Pick-Response" relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 4 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.4.8., Fig. 3.4.9., and Fig 3.4.10., respectively.
Fig 3.4.8. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.4.9. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.4.10. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
**d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 4 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 4 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.4.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

*Tab. 3.4.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 4 Under Different Earthquake Intensity Levels in Longitudinal Direction-x*

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
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</table>

**e) The predicted Seismic Vulnerability Functions of Building No. 4, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.**

The predicted direct analytical vulnerability functions of the integral Building No. 4 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.4.11, Fig. 3.4.12, and Fig. 3.4.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.4.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 4 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.4.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 4 in Direction-x Under El-Centro earthquake

Fig. 3.4.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 4 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.4.3. Seismic Vulnerability Analysis of Building No. 4 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 4 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 4 in transversal direction, Fig. 3.4.14.

Mathematical model used for vulnerability analysis of Building No. 4 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.4.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.4.16, and Fig. 3.4.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 4 for Transversal Direction-y

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.4.18.

![Figure 3.4.18, Envelope curves for structural behavior.](image)

To obtain full evidence in the most important response parameters of Building No. 4 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 4 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.4.19., Fig. 3.4.20., and Fig 3.4.21., respectively.

![Fig 3.4.19. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y](image)
Fig 3.4.20. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y

Fig 3.4.21. Computed Pick Relative Storey Displacements of Building No. 4 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 4 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 4 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.4.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (%) in Transversal Direction-y</th>
</tr>
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<tbody>
<tr>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
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<td>0.127 0.225 0.581 1.034 1.661 1.960 2.178 2.329 2.493 2.534 2.691</td>
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EQI – El-Centro

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<td>9.551</td>
<td>12.223</td>
<td>15.431</td>
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</tbody>
</table>

EQI – Prishtina Synthetic

<table>
<thead>
<tr>
<th>Maximum Ground Acceleration (G)</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
<th>0.50g</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.087</td>
<td>0.174</td>
<td>0.309</td>
<td>0.792</td>
<td>1.359</td>
<td>1.671</td>
<td>1.725</td>
<td>1.584</td>
<td>1.282</td>
<td>1.047</td>
<td>1.050</td>
</tr>
<tr>
<td>2</td>
<td>0.247</td>
<td>0.497</td>
<td>1.056</td>
<td>1.923</td>
<td>2.941</td>
<td>4.324</td>
<td>6.636</td>
<td>7.793</td>
<td>8.434</td>
<td>7.479</td>
<td>6.875</td>
</tr>
<tr>
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<td>0.332</td>
<td>0.670</td>
<td>1.545</td>
<td>2.923</td>
<td>4.410</td>
<td>6.237</td>
<td>8.628</td>
<td>10.146</td>
<td>11.157</td>
<td>11.000</td>
<td>11.644</td>
</tr>
</tbody>
</table>

**e) The Predicted Seismic Vulnerability Functions of Building No. 4, Under the Effect of Three Selected Earthquakes in Transversal Direction-y**

The predicted direct analytical vulnerability functions of the integral Building No. 4 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.4.22., Fig. 3.4.23. and Fig. 3.4.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Graph](image)

**Fig. 3.4.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 4. in Direction-y Under Ulcinj- Albatros earthquake**
3.4.4. Comparative Presentation of Damage Propagation Trough SE&NE of Masonry Building No. 4. in Case of Three Considered Earthquakes in Directions - x & y

Calculated results for building vulnerability show that under the impact of three earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro and EQ-3 Prishtina Synthetic), SE and NE of the structure behave as follows:

1. For the longitudinal direction x, as can be seen in the stiffness diagrams shown in figure 3.4.7, and for transversal direction y, shown in figure 3.4.18, it is clearly visible that the stiffness along longitudinal direction x is much higher, and as a result collapse takes place along the transversal direction x for the PGA = 0.20g, and for the longitudinal direction at PGA=0.45g.

2. Regardless of the overall stiffness of the building as a whole, collapse takes place simultaneously in SE and NE on the second level.

Damage level at the collapse peak for NE is larger (2.43%) compared to SE (1.98%), which occurs under the impact of Pristina Synthetic earthquake.
### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for El-Centro Earthquake in Transversal Direction-y

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**

### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for Prishtina Synthetic Earthquake in Transversal Direction-y

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**

### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for El-Centro Earthquake in Longitudinal Direction-x

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**

### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for Prishtina Synthetic Earthquake in Longitudinal Direction-x

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**

### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for El-Centro Earthquake in Direction-x

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**

### Damage Propagation Troudh SE & NE of Masonry Building No. 4. for Prishtina Synthetic Earthquake in Direction-x

- **EQI1 = 0.025G**
- **EQI2 = 0.05G**
- **EQI3 = 0.10G**
- **EQI4 = 0.15G**
- **EQI5 = 0.20G**
- **EQI6 = 0.25G**
- **EQI7 = 0.30G**
- **EQI8 = 0.35G**
- **EQI9 = 0.40G**
- **EQI10 = 0.45G**
- **EQI11 = 0.50G**
3.4.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 4. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 4, the following general conclusions can be derived:

(1) In the Fig 3.4.7 and Fig 3.4.18 for capacity diagrams of SE and NE, along orthogonal directions x and y, it can be observed that participation of SE in the overall capacity of the building is larger than of NE. Also, from these diagrams we can see variations of building stiffness along both orthogonal directions, where the building stiffness along longitudinal direction x is much higher than along the other direction. This is a result of different dimensions of the base, where the sides ratio is lx/ly = 26,0 / 14,0.

(2) Displacements of separate structural elements on different levels are different and depend on the overall stiffness of the building for orthogonal directions. This can be also observed through results for Ulcinj-Albatros earthquake, where displacement at the top level along the longitudinal direction x is 2.869 cm (for PGA = 0.45g), and along the y direction the value is 3.474 cm (for PGA = 0.25g).

(3) Building collapse takes place on PGA = 0.20g in transversal y-direction. This is because of the present different story stiffness and storey strength for directions X and Y. Stiffness and strength is greater in referent longitudinal direction x, therefore collapse of the building happens faster in the transversal y-direction with the lower stiffness and strength. As a result of the large stiffness along the x direction, it can be seen in the damage propagation under El-Centro and Pristina Synthetic earthquakes that theoretically the building does not collapse, even though the total collapse occurs in PGA = 0.20g.

(4) In two cases of earthquake strikes on Building No.4, along the transversal direction y, collapse takes place on the second level. The reason for collapse of SE and NE on the second level is that walls of the first level are stiffer (thickness is 45cm), and on the other levels, walls are thinner (38cm thick).

(5) Total loss is 4.41% from the total building cost in the collapse moment in case of Pristina synthetic-artificial earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 1.98% and non-structural elements with 2.43%.
3.5. Seismic Vulnerability Analysis of Building No. 5 in Longitudinal Direction-x and Transversal Direction-y

3.5.1. Description of basic characteristics of the building structural system

The Building in previously used as a Town Clinic Center, but later converted for residential use. This conversion included removal and re-positioning of several partition walls. New partition walls in general are positioned same in all levels. Building has ground and first floor. Assessment is made taking into consideration present condition of the building.

During conversion large portions of partition walls were removed with the intention to have large internal areas, but also braking large openings on load-baring walls, thus changing the structural system.

Floor plan of the building with dimensions (27.80 x 17.00)m, shown in Fig. 3.5.2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are five linear load baring walls, 5.36m, 2.18 and 9.0 apart, and on the transversal direction - y, there are six linear walls with different
distances among each other. The building consists of basement floor (3.07m high) ground and first floor (3.40m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). Walls are of bricks and have thickness of 38cm on all levels. Brick dimensions 25x12x6.5cm and are bricked with cement plaster. It is important to mention that the central wall along axis “4-4” is 89cm thick and no structural joint is visible along this wall. Walls are properly interconnected during bricking (without bond beams). Mezzanine structures are with wooden beams and in toilets there are concrete slabs (constructed later).

3.5.2. Seismic Vulnerability Analysis of Building No. 5 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 5 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction. Mathematical model used for vulnerability analysis of Building No. 5 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.5.4, shown is the formulated mathematical model of the building consisting of two concentrated masses and of two principal elements for each storey representing non-linear
stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.5.5, and Fig. 3.5.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 5 for Longitudinal Direction-x**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.5.7.

![Figure. 3.5.7 Envelope curves for structural behavior.](image)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 5 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 5 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 5 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.5.8., Fig. 3.5.9., and Fig 3.5.10., respectively.

![Fig 3.5.8. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x](image)
**Fig 3.5.9. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x**

**Fig 3.5.10. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x**

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 5 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 5 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.5.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.5.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 5 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>2</td>
<td>0.251</td>
</tr>
<tr>
<td>1</td>
<td>0.385</td>
</tr>
</tbody>
</table>
e) The predicted Seismic Vulnerability Functions of Building No. 5, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 5 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.5.11, Fig. 3.5.12, and Fig. 3.5.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

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**Fig. 3.5.11.** The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 5 in Direction-x Under Ulqin – Albatros earthquake
3.5.3. Seismic Vulnerability Analysis of Building No. 5 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 5 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 5 in transversal direction, Fig. 3.5.14.
In Fig. 3.5.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.5.16, and Fig. 3.5.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 5 for Transversal Direction-\( y \)**

The calculated initial stiffness \( K_0 \), and respective force and displacement values for above specified points are presented in Fig. 3.5.18.

*Figure. 3.5.18, Envelope curves for structural behavior.*
c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 5 Under Different Earthquake Intensity Levels in Transversal Direction-y**

To obtain full evidence in the most important response parameters of Building No. 5 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 5 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.5.19., Fig. 3.5.20., and Fig 3.5.21., respectively.

![Graph](image1.png)

**Fig 3.5.19. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y**

![Graph](image2.png)

**Fig 3.5.20. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y**
Fig. 3.5.21. Computed Pick Relative Storey Displacements of Building No. 5 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y

\textit{d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 5 Under Different Earthquake Intensity Levels in Transversal Direction-y}

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 5 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.5.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

\textit{Tab. 3.5.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 5 Under Different Earthquake Intensity Levels in Transversal Direction-y}

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>2</td>
<td>0.085 0.173 0.345 0.521 0.831 1.238 1.866 2.456 3.062 3.847 4.541</td>
</tr>
<tr>
<td>1</td>
<td>0.115 0.232 0.465 0.697 1.056 1.574 2.353 3.118 3.971 5.103 5.997</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – El-Centro</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>2</td>
<td>0.107 0.218 0.436 0.704 1.111 1.492 1.883 2.417 3.013 3.502 3.984</td>
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<td>0.147 0.297 0.594 0.994 1.521 2.091 2.709 3.282 4.118 4.829 5.571</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>2</td>
<td>0.068 0.137 0.270 0.384 0.505 0.932 1.336 1.837 2.508 2.945 3.625</td>
</tr>
<tr>
<td>1</td>
<td>0.091 0.182 0.365 0.515 0.656 1.071 1.682 2.406 3.171 3.926 4.735</td>
</tr>
</tbody>
</table>
e) The Predicted Seismic Vulnerability Functions of Building No. 5, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No 5 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.5.22., Fig. 3.5.23. and Fig. 3.5.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.5.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 5. in Direction-y Under Ulcinj-Albatros earthquake

Fig. 3.5.23 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 5. in Direction-y Under El-Centro earthquake
Based on the results of building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro & EQ-3 Prishtina Synthetic), the following conclusions can be made for behavior of SE and NE:

1. Stiffness along the y direction is larger than stiffness along the x direction. As a result, collapse takes place along the direction with the lower stiffness – along the longitudinal direction x. This collapse takes place in SE and NE simultaneously for PGA = 0.15g. Collapse progress of SE and NE along the longitudinal direction x starting from PGA = 0.025g and onward is almost immediate.

2. Participation of NE in the overall stiffness of the building along both orthogonal directions is small compared to SE.

3. Regardless of the overall stiffness of the building, collapse takes place simultaneously in SE and NE on the second level.

Damage level of SE and NE is close to equal (2.17% SE and 2.19% NE).
<table>
<thead>
<tr>
<th>EQ = 1 (B4y)</th>
<th>EQI1 = 0.025G</th>
<th>EQI2 = 0.05G</th>
<th>EQI3 = 0.10G</th>
<th>EQI4 = 0.15G</th>
<th>EQI5 = 0.20G</th>
<th>EQI6 = 0.25G</th>
<th>EQI7 = 0.30G</th>
<th>EQI8 = 0.35G</th>
<th>EQI9 = 0.40G</th>
<th>EQI10 = 0.45G</th>
<th>EQI11 = 0.50G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage Propagation Troudh SE &amp; NE of Masonry Building No. 5. for Pristina Synthetic Earthquake in Direction-x</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Damage Propagation Troudh SE &amp; NE of Masonry Building No. 5. for Pristina Synthetic Earthquake in Direction-y</td>
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<tr>
<td>Damage Propagation Troudh SE &amp; NE of Masonry Building No. 5. for Pristina Synthetic Earthquake in Direction-z</td>
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</tr>
</tbody>
</table>
3.5.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 5. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 5, the following general conclusions can be derived:

(1) It can be seen in Fig. 3.5.1 and Fig. 3.5.18, for capacity diagrams of SE and NE, along the longitudinal direction x and transversal direction y, that participation of NE in the overall capacity of the building is almost negligible. Also in these diagrams we can notice the stiffness variation along both orthogonal directions, where building stiffness along y direction is much higher than along the other direction. This is a result of the building base shape with a large difference of the side dimensions, where stiffness of the central wall along y direction is very high.

(2) Displacements of separate structural elements through different levels are different and dependent on the overall stiffness of the building for each direction. This comparison can also be made looking at the calculated results under the impact of Pristina Synthetic earthquake, where displacement of the top level along x direction is 2.359cm, and for the other direction 1.98cm (for PGA = 0.20g).

(3) Building collapse happens on PGA = 0.15g in longitudinal x-direction. This is because of the present different story stiffness and storey strength for directions X and Y. From the large stiffness along the y direction, it can be seen also from the damage propagation for earthquakes Ulcinj-Albatros and Pristina Synthetic, building collapses at the last PGA values even though there is a total destruction under PGA = 0.15g.

(4) Under the impact of Ulcinj-Albatros earthquake along the longitudinal direction x, there is an immediate collapse of the building on two levels in SE and NE, and in the case of El-Centro earthquake, again along the longitudinal direction x, collapse also takes place on the second level. Difference in the damage propagation of SE and NE from level to level is very small, as the building consists of only two levels, what can be seen in the figure of damage propagations that the differences through levels are very small.

(5) Total loss is 4.36% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.17% and non-structural elements with 2.19%.
3.6. Seismic Vulnerability Analysis of Building No. 6 in Longitudinal Direction-x and Transversal Direction-y

3.6.1. Description of basic characteristics of the building structural system

It is a residential building. Initially it was used as a Clinic Center – Hospital, but was later modified for residential use. This conversion included removal and re-positioning of several partition walls. New partition walls in general are positioned same in all levels. Building has ground floor, two floors and attic. Assessment was made taking into consideration current condition of the building.

![Building No. 6: Residential Building, Ilir Konushevc str. (ex city clinic center).](image)

**Fig. 3.6..1. Building No. 6: Residential Building, Ilir Konushevc str. (ex city clinic center).**

**Fig. 3.6..2. Building No. 6: Floor plan**

Floor plan of the building with dimensions (25.11 x 10.95)m, shown in Fig. 3.6..2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are three linear load baring walls, 4.67m, and 5.83m apart, and on the transversal direction - y, there are eight linear walls with different distances among each other. The building consists of ground floor (2.72m high) ground and
for all stories (4 x 3.05m high). Load-bearing walls are made of stones and bricks. Connection points of load-bearing walls on two directions are strengthened with mortar (masonry, connected). Basement walls are with stone and have a thickness of 38cm. Other load-bearing walls are with bricks and are 38cm thick too. Brick dimensions are 25x12x6.5cm and are bricked with cement plaster. Unit brick dimensions are 25x12x6.5cm and are usually bricked with cement plaster.

3.6.2. Seismic Vulnerability Analysis of Building No. 6 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 6 in Longitudinal Direction-x and Structural Dynamic Characteristics

Mathematical model used for vulnerability analysis of Building No. 6 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

Fig. 3.6.3 Building No. 6: Part of Individual Wall Segments A-A, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Mathematical model used for vulnerability analysis of Building No. 6 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

Fig. 3.6.4. Building No. 6: Non-Linear MC Model for Direction-x

Fig. 3.6.5. Building No. 6: Mode Shape-1, Direction-x; $T_{1x}=0.482$ sec

Fig. 3.6.6. Building No. 6: Mode Shape-2, Direction-x; $T_{2x}=0.164$ sec
In Fig. 3.6.4, shown is the formulated mathematical model of the building consisting of five concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.6.5, and Fig. 3.6.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 6 for Longitudinal Direction-x**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.6.7.

![Figure 3.6.7 Envelope curves for structural behavior.](image)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 6 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 6 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 6 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.6.8., Fig. 3.6.9., and Fig 3.6.10., respectively.
Fig 3.6.8. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.6.9. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.6.10. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) **Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 6 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 6 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.6.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.6.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 6 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-x</th>
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<tr>
<td>0.025g</td>
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<tr>
<td>0.025g</td>
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<tr>
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<table>
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<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-x</th>
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</thead>
<tbody>
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<td>0.221</td>
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<td>4</td>
<td>0.682</td>
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</table>

e) **The predicted Seismic Vulnerability Functions of Building No. 6, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.**

The predicted direct analytical vulnerability functions of the integral Building No. 6 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.6.11, Fig. 3.6.12, and Fig. 3.6.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is
the cost ratio of 65% for structural elements and 35% for non-structural elements. Through
the adapted ratio, defined are loss functions for structural and non-structural elements. In this
particular case adopted is uniform cost distribution of structural and non-structural elements
throughout the height of the building.

Fig. 3.6.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE
and NE) of masonry building No. 6 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.6.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE
and NE) of masonry building No. 6 in Direction-x Under El-Centro earthquake

Fig. 3.6.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE
and NE) of masonry building No. 6 in Direction-x Under Prishtina Synthetic, Earthquake
3.6.3. Seismic Vulnerability Analysis of Building No. 6 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 6 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 6 in transversal direction, Fig. 3.6.14.

Mathematical model used for vulnerability analysis of Building No. 6 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

Fig. 3.6.15. Building No. 6: Non-Linear MC Model for Direction-y

Fig. 3.6.16. Building No. 6: Mode Shape-1, Direction-y; \( T_{1y}=0.435 \) sec

Fig. 3.6.17. Building No. 6: Mode Shape-2, Direction-y; \( T_{2y}=0.143 \) sec

In Fig. 3.6.15, shown is the formulated mathematical model of the building consisting of five concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.6.16, and Fig. 3.6.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 6 for Transversal Direction-y

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.6.18.

![Force-Displacement Graph](image)

*Figure. 3.6.18, Envelope curves for structural behavior.*

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 6 Under Different Earthquake Intensity Levels in Transversal Direction-y

To obtain full evidence in the most important response parameters of Building No. 6 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 6 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.6.19., Fig. 3.6.20., and Fig 3.6.21., respectively.
Fig 3.6.19. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y

Fig 3.6.20. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y

Fig 3.6.21. Computed Pick Relative Storey Displacements of Building No. 6 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 6 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 6 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.6.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.6.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 6 Under Different Earthquake Intensity Levels in Transversal Direction-y

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
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<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
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<td>2.961</td>
<td>3.446</td>
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e) The Predicted Seismic Vulnerability Functions of Building No. 6, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 6 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.6.22., Fig. 3.6.23. and Fig. 3.6.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through
the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.6.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 6. in Direction-y Under Ulcinj-Albatros earthquake

Fig. 3.6.23 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 6. in Direction-y Under El-Centro earthquake

Fig. 3.6.24 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 6. in Direction-y Under Prishtina Synthetic earthquake
<table>
<thead>
<tr>
<th>EQ1 (B6)</th>
<th>EQ1 = 0.025G</th>
<th>EQ2 = 0.05G</th>
<th>EQ3 = 0.10G</th>
<th>EQ4 = 0.15G</th>
<th>EQ5 = 0.20G</th>
<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
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Damage Propagation Troudh SE & NE of Masonry Building No. 6. for Ulcinj - Albatros Earthquake in Longitudinal Direction-x

EQ11 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G

Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQ1 (B6)</th>
<th>EQ1 = 0.025G</th>
<th>EQ2 = 0.05G</th>
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</table>

Damage Propagation Troudh SE & NE of Masonry Building No. 6. for El - Centro Earthquake in Transversal Direction-y

Tranversal Direction-y
3.6.4. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 6. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 6, the following general conclusions can be derived:

(1) In Fig. 3.6.7. and Fig. 3.6.18, for capacity diagrams of SE and NE, along orthogonal directions x and y, it is visible that participation of NE in the overall building capacity is very small and without impact. Also in these diagrams we can notice the variation of stiffness along two orthogonal directions, where building stiffness along transversal direction y is much higher than along the other direction x. This is a result of the shape of the building base that has different side dimensions, but also because of the large stiffness of the central wall along the y direction. It is important to mention that even though stiffness of the building along transversal direction y is higher, collapse takes place simultaneously along both directions for PGA = 0.15g.

(2) Displacements of separate structural elements on different levels are different and dependent also on the overall stiffness for the respective directions. Since the building has 5 storeys, there is not a large variation of displacements for the respective directions, where in case of the Ulcinj-Albatros earthquake impact, displacement is 3.902cm along the x direction, and 2.346cm along the y direction, for PGA = 0.15g. This could be the reason of simultaneous collapse of the building along both directions for the same PGA value.

(3) Along the longitudinal direction x, total collapse takes place under the impact of three earthquakes for the same PGA values, but along the transversal direction y collapse takes place under El-Centro earthquake impact. As far as level of damages is concerned, they are different for each of the earthquake scenarios.

(4) Total loss is 3.42% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 1.42% and non-structural elements with 2.02%. Collapse takes place for small values of damages.
3.7. Seismic Vulnerability Analysis of Building No. 7 in Longitudinal Direction-x and Transversal Direction-y

3.7.1. Description of basic characteristics of the building structural system

Building serves as an apartment building for collective housing. Following privatization of buildings, these buildings are privately owned by occupants. The building was renovated a few times in the past. It consists of Basement, ground and three floors.

Figure 3.7.1. Building No. 7: Residential Building No. 7, Sylejman Vokshi str.

![Building No. 7: Residential Building No. 7, Sylejman Vokshi str.](image)

Fig. 3.7.2. Building No. 7: Floor plan

Floor plan of the building with dimensions (21.10 x 9.50)m, shown in Fig. 3.7.2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.
On the longitudinal direction, along “x” axis, there are three linear load baring walls, and on the transversal direction - y, there are nine linear walls with different distances among each other. The building consists of basement floor (3.01m high) ground and for all stories (3 x 3.30m high). Load-baring walls are made of stones and bricks. Walls at the basement level are with stones and are 60cm thick, and brick walls with a thickness of 50cm are on all other levels. Brick dimensions are 25x12x6.5cm and are bricked with cement plaster. Walls are properly interconnected during bricking.

3.7.2. Seismic Vulnerability Analysis of Building No. 7 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 7 in Longitudinal Direction-x and Structural Dynamic Characteristics

Mathematical model used for vulnerability analysis of Building No. 7 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.7.4, shown is the formulated mathematical model of the building consisting of four concentrated masses and of two principal elements for each storey representing non-linear
stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.7.5, and Fig. 3.7.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 7 for Longitudinal Direction-x**

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.7.7.

![Figure. 3.7.7 Envelope curves for structural behavior.](image)

**Figure. 3.7.7 Envelope curves for structural behavior.**

c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 7 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 7 in longitudinal x-direction, the computed maximum or "Pick-Response" relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 7 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.7.8., Fig. 3.7.9., and Fig. 3.7.10., respectively.
Fig 3.7.8. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.7.9. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.7.10. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 7 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 7 under different earthquake intensity levels in longitudinal direction-x are presented in Table 3.7.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.7.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 7 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<tr>
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<table>
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<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<td>0.382</td>
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</table>

e) The predicted Seismic Vulnerability Functions of Building No. 7, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 7 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.7.11, Fig. 3.7.12, and Fig. 3.7.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements.
Fig. 3.7.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 7 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.7.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 7 in Direction-x Under El-Centro earthquake

Fig. 3.7.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 7 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.7.3. Seismic Vulnerability Analysis of Building No. 7 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 7 in Transversal Direction-y and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in transverse y-direction. The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 7 in transversal direction, Fig. 3.7.14.

![Fig. 3.7.14 Building No. 7: Part of Individual Wall Segments I-I, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-y](image)

Mathematical model used for vulnerability analysis of Building No. 7 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Fig. 3.7.15. Building No. 7: Non-Linear MC Model for Direction-y](image)

In fact, for this study purposes, the formulated non-linear mathematical model is defined as “shear type”, formulated based on systematic implementation of ”multi component” concept.
In Figure 3.7.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with four principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Figure 3.7.16 and Figure 3.7.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 7 for Transversal Direction-y**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.7.18.

![Figure 3.7.18, Envelope curves for structural behavior.](image)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 7 Under Different Earthquake Intensity Levels in Transversal Direction-y**

To obtain full evidence in the most important response parameters of Building No. 7 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 7 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.7.19., Fig. 3.7.20., and Fig 3.7.21., respectively.
Fig 3.7.19. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y

Fig 3.7.20. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y

Fig 3.7.21. Computed Pick Relative Storey Displacements of Building No. 7 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 7 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 7 under different earthquake intensity levels in transversal direction-\(y\) are presented in Table 3.7.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Storey Drift ISD (‰) in Transversal Direction-(y)</th>
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</thead>
<tbody>
<tr>
<td>0.025g</td>
<td>0.226 0.399 0.907 1.482 1.797 1.997 2.817 5.186 8.741 11.987 11.967</td>
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<tr>
<td>0.05g</td>
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<tr>
<td>0.10g</td>
<td>0.606 1.288 3.730 6.297 9.506 11.888 15.476 27.033 31.409 38.164 46.415</td>
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<td>0.15g</td>
<td>0.661 1.358 3.970 6.779 10.242 12.758 16.388 28.055 32.624 39.515 48.279</td>
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<th>EQI – El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-(y)</th>
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</thead>
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<td>0.467 0.903 2.670 4.355 4.064 4.961 5.667 6.194 6.718 10.861 8.685</td>
</tr>
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<table>
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<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-(y)</th>
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<td>0.15g</td>
<td>0.452 0.933 3.130 6.109 9.642 12.264 14.348 16.809 17.333 18.370 18.003</td>
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</table>

e) The Predicted Seismic Vulnerability Functions of Building No. 7, Under the Effect of Three Selected Earthquakes in Transversal Direction-\(y\)

The predicted direct analytical vulnerability functions of the integral Building No 7 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.7.22., Fig. 3.7.23. and Fig. 3.7.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements.
Fig. 3.7.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 7. in Direction-y Under Ulcinj-Albatros earthquake

Fig. 3.7.23. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 7. in Direction-y Under El-Centro earthquake

Fig. 3.7.24. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 7. in Direction-y Under Prishtina Synthetic earthquake
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<td>0.50G</td>
</tr>
</tbody>
</table>

**Damage Propagation Troudh SE & NE of Masonry Building No. 7. for El-Centro Earthquake in Longitudinal Direction-x**

**Damage Propagation Troudh SE & NE of Masonry Building No. 7. for Prishtina Synthetic Earthquake in Transversal Direction-y**
3.7.4. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 7. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 7, the following general conclusions can be derived:

(1) From the capacity diagrams of SE and NE shown in Fig. 3.7.7 and Fig. 3.7.18, along both orthogonal directions, building stiffness is very similar along both directions. Participation of NE in the overall building capacity is very low and without considerable impact on the overall capacity.

(2) Displacements of separate structural elements on various levels are different and depend also on the overall building stiffness for the respective directions. Since building consists of four levels, displacements are similar for both orthogonal directions, where in the case of Ulcinj-Albatros earthquake impact, at the collapse peak displacement along the y direction is 2.237 cm and along the x direction is 2.073 cm for PGA = 0.15 g along both orthogonal directions.

(3) Along transversal direction y, collapse takes place under the impact of Ulcinj-Albatros and El-Centro earthquakes, for PGA = 0.15 g. Collapse takes place on the second level.

(4) Total loss is 3.87% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.10% and non-structural elements with 1.77%. Collapse takes place for small values of building damages.
3.8. Seismic Vulnerability Analysis of Building No. 8 in Longitudinal Direction-x and Transversal Direction-y

3.8.1. Description of basic characteristics of the building structural system

During the whole period, the building was used as collective apartment building. Following the public building privatization, these buildings now are privately-owned by occupants. The building was renovated a few times in the past. It has the basement, ground and first floor. It is important to mention that in these buildings, ground floor is converted into business areas.

![Building No. 8: Residential Building No. 8, Ymer Alishani str. behind “kurrizi”](image)

**Figure 3.8.1. Building No. 8: Residential Building No. 8, Ymer Alishani str. behind “kurrizi”**

![Floor plan](image)

**Figure 3.8.2. Building No. 8: Floor plan**

Floor plan of the building with dimensions (18.50 x 10.00)m, shown in Figure 3.8.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are three linear load baring walls, 4.92m and 4.60 apart, and on the latitudinal direction, along “y” axis, there are five linear walls with different distances among each other (3.50 and 4.50)m. The building consists of basement floor (2.63m high) ground and first floor (3.10m high). Connection points of load baring walls
on two directions are strengthened with our self (masonry, connected). Load-bearing walls are made of stones and bricks. Walls at the basement level are with stones and are 50cm thick, and brick walls with a same thickness are on all other levels. Structural wall sections with parapets and spandrels are treated as non-structural elements.

3.8.2. Seismic Vulnerability Analysis of Building No. 8 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 8 in Longitudinal Direction-x and Structural Dynamic Characteristics

Mathematical model used for vulnerability analysis of Building No. 8 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Figure 3.8.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.
In Figure 3.8.5, and Figure 3.8.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 8 for Longitudinal Direction-x

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.8.7.

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 8 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 8 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 8 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.8.8., Fig. 3.8.9., and Fig. 3.8.10., respectively.
Fig 3.8.9. Computed Pick Relative Storey Displacements of Building No. 8 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.8.10. Computed Pick Relative Storey Displacements of Building No. 8 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 8 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 8 under different earthquake intensity levels in longitudinal direction-x are presented in Table 3.8.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.8.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 8 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (%) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.122   0.240 0.863 1.319 2.122 2.468 3.445 4.205 4.962 6.688 7.502</td>
</tr>
<tr>
<td>2</td>
<td>0.200   0.403 1.323 2.248 3.558 5.645 7.939 11.168 14.429 18.400 20.300</td>
</tr>
<tr>
<td>1</td>
<td>0.245   0.490 1.539 2.726 4.152 6.274 10.590 16.332 22.181 26.765 29.655</td>
</tr>
<tr>
<td>EQI – El-Centro</td>
<td>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</td>
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<td>-------------------------------------------------------------</td>
</tr>
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<td>0.206</td>
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</table>

**e) The predicted Seismic Vulnerability Functions of Building No. 8, Under the Effect of Three Selected Earthquake in Longitudinal Direction-x.**

The predicted direct analytical vulnerability functions of the integral Building No. 8 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.8.11, Fig. 3.8.12, and Fig. 3.8.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Figure 3.8.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 8 in Direction-x Under Ulqin – Albatros earthquake](image-url)
Figure 3.8.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 8 in Direction-x Under El-Centro earthquake

Fig. 3.8.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 8 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.8.3. Seismic Vulnerability Analysis of Building No. 8 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 8 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 8 in transversal direction, Fig. 3.8.14.

![Diagram](image)

*Figure 3.8.14 Building No. 8: Part of Individual Wall Segments 3-3, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-y*

Mathematical model used for vulnerability analysis of Building No. 8 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Diagram](image)

*Figure 3.8.15. Building No. 8: Non-Linear MC Model for Direction-y*

In Figure 3.8.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Figure 3.8.16, and Figure 3.8.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

![Diagram](image)

*Figure 3.8.16. Building No. 8: Mode Shape-1, Direction-y; T₁y=0.214 sec*

*Figure 3.8.17. Building No. 8: Mode Shape-2, Direction-y; T₂y=0.075 sec*

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 8 for Transversal Direction-y

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.8.18.
c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 8 Under Different Earthquake Intensity Levels in Transversal Direction-y**

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 8 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.8.19., Fig. 3.8.20., and Fig 3.2.21., respectively.

*Fig 3.8.19. Computed Pick Relative Storey Displacements of Building No. 8 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y*
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 8 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 8 under different earthquake intensity levels in transversal direction-y are presented in Table 3.8.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.8.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 8 Under Different Earthquake Intensity Levels in Transversal Direction-y**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.103 0.205 0.437 0.882 1.426 2.209 2.597 3.171 3.848 4.658 5.924</td>
</tr>
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<td>2</td>
<td>0.161 0.326 0.655 1.200 2.226 3.194 4.048 4.877 6.223 7.506 8.900</td>
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<td>0.187 0.377 0.755 1.316 2.429 3.545 4.490 5.513 7.129 8.726 10.426</td>
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The predicted direct analytical vulnerability functions of the integral Building No 8 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.8.22., Fig. 3.8.23. and Fig. 3.8.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Figure 3.8.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 8. in Direction-y Under Ulcinj- Albatros earthquake](image)

**Figure 3.8.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 8. in Direction-y Under Ulcinj- Albatros earthquake**
3.8.4. Comparative Presentation of Damage Propagation Through SE&NE of Masonry Building No. 8. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results for the building vulnerability under three Earthquakes (EQ=1, Ulcinj-Albatros, EQ-2, El-Centro & EQ-3, Prishtina Synthetic), behaviour of SE and NE within the structure can be described as follows:

1. Stiffness along the transversal direction y is much higher than stiffness along direction x. From this it can be concluded that the collapse takes place along the longitudinal direction x. Collapse accrues in SE and NE simultaneously for PGA = 0.25g.
2. Participation of NE in the overall building stiffness along both orthogonal directions is very small compared to SE participation.
3. Regardless of the overall building stiffness, collapse takes place simultaneously in SE and NE on the second level.
4. Damage propagation in SE is larger than the one in NE (3.32% SE and 2.74% NE).
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Figure 3.8.25: Damage Propagation Through SE & NE of Masonry Building No. 8. for Ulcinj - Albatros Earthquake in Longitudinal Direction - x

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G

Figure 3.8.26: Damage Propagation Through SE & NE of Masonry Building No. 8. for El-Centro Earthquake in Longitudinal Direction - x

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G

Figure 3.8.27: Damage Propagation Through SE & NE of Masonry Building No. 8. for Pristina Synthetic Earthquake in Longitudinal Direction - x

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G

Figure 3.8.28: Damage Propagation Through SE & NE of Masonry Building No. 8. for Pristina Synthetic Earthquake in Transversal Direction - y

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G

Figure 3.8.29: Damage Propagation Through SE & NE of Masonry Building No. 8. for Pristina Synthetic Earthquake in Transversal Direction - y

EQI1 = 0.025G EQI2 = 0.05G EQI3 = 0.10G EQI4 = 0.15G EQI5 = 0.20G EQI6 = 0.25G EQI7 = 0.30G EQI8 = 0.35G EQI9 = 0.40G EQI10 = 0.45G EQI11 = 0.50G
3.8.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 8. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 8, the following general conclusions can be derived:

(1) As seen in Fig. 3.8.7 and Fig. 3.8.18 for capacity diagrams of SE and NE along longitudinal and transversal directions participation of NE in the overall building capacity is negligible. Also the diagrams we can see the variation of stiffness along the orthogonal directions, where building stiffness along the transversal direction y is much higher than along the other direction. This is a result of the building base shape. Based on this variation, collapse takes place along the longitudinal direction x with the smaller stiffness.

(2) Displacements of separate structural elements for each level are different and depend on the overall building stiffness for orthogonal directions. Displacements can be easily compared also by viewing the numeric results under the impact of Ulcinj-Albatros earthquake, where on the top level displacement along the x direction is 1.945cm, and along the y direction is 1.099cm (for PGA = 0.25g).

(3) Building collapse happens on PGA = 0.25g in longitudinal x-direction. This is because of the present different story stiffness and storey strength for directions X and Y. Coming from the large stiffness along the y direction, as can be seen in the damage propagation results for the El-Centro and Pristine Synthetic earthquakes, building collapses under final PGA values, even though it reaches the total destruction for PGA = 0.25g.

(4) Under the impact of Ulcinj-Albatros earthquake, collapse takes place along the longitudinal direction x in SE and NE simultaneously on the second level.

(5) Total loss is 6.07% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 3.32% and non-structural elements with 2.74%.
3.9. Seismic Vulnerability Analysis of Building No. 9 in Longitudinal Direction-x and Transversal Direction-y

3.9.1. Description of basic characteristics of the building structural system

It is a residential building for collective housing. Following the privatization process of public buildings, it is now privately owned by occupants. Building consists of Ground and two floors. As in many above mentioned cases, in this building also ground floor areas are modified for commercial use. This conversion included removal of partition walls, as well as braking large openings on load-baring walls. Assessment is made taking into consideration current condition of the building.

Figure 3.9.1. Building No. 9: Residential Building No. 9, Qamil Hoxha str.

Figure 3.9.2. Building No. 9: Floor plan

Floor plan of the building with dimensions (22.96 x 10.00)m, shown in Figure 3.9.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.
On the longitudinal direction, along “x” axis, there are four linear load bearing walls, 4.0m, 2.0m and 3.50 apart, and on the latitudinal direction, along “y” axis, there are six linear walls with different distances among each other (4.40, 3.90 and 4.50)m. The building consists of basement floor (3.15m high) ground (3.35m high) and first floor (3.30m high). Connection points of load bearing walls on two directions are strengthened with our self (masonry, connected). Walls are of clay bricks and have thickness of 50cm on all levels. Brick dimensions 25x12x6.5cm and are bricked with cement plaster. Walls are properly interconnected during bricking (without bond beams). Structural wall sections with parapets and spandrels are treated as non-structural elements.

3.9.2. Seismic Vulnerability Analysis of Building No. 9 Longitudinal Direction- x

a) Formulation of Non-Linear Mathematical Model of Building No. 9 in Longitudinal Direction- x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

Mathematical model used for vulnerability analysis of Building No. 9 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.
In Figure 3.9.4, shown is the formulated mathematical model of the building consisting of two concentrated masses and of three principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Figure 3.9.5, and Figure 3.9.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 9 for Longitudinal Direction-x**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.9.7.

![Figure 3.9.7 Envelope curves for structural behavior.](image)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 9 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 9 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 9 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.9.8., Fig. 3.9.9., and Fig. 3.9.10., respectively.
Fig 3.9.8. Computed Pick Relative Storey Displacements of Building No. 9 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.9.9. Computed Pick Relative Storey Displacements of Building No. 9 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.9.10. Computed Pick Relative Storey Displacements of Building No. 9 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 9 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 9 under different earthquake intensity levels in longitudinal direction-x are presented in Table 3.9.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.9.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 9 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<tr>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
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<table>
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<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<td>1 0.155 0.306 0.642 1.252 2.048 2.882 3.591 4.470 5.452 6.576 8.585</td>
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</tr>
</tbody>
</table>

e) The predicted Seismic Vulnerability Functions of Building No. 9, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 9 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.9.11, Fig. 3.9.12, and Fig. 3.9.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Figure 3.9.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 9 in Direction-x Under Ulqin – Albatros earthquake

Figure 3.9.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 9 in Direction-x Under El-Centro earthquake

Fig. 3.9.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 9 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.9.3. Seismic Vulnerability Analysis of Building No. 9 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 9 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 9 in transversal direction, Fig. 3.9.14.

Figure 3.9.14 Building No. 9: Part of Individual Wall Segments 2-2, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-y

Mathematical model used for vulnerability analysis of Building No. 9 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Figure 3.9.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Figure 3.9.16, and Figure 3.9.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 9 for Transversal Direction-y

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.9.18.

![Figure 3.9.18, Envelope curves for structural behavior.](image)

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 9 Under Different Earthquake Intensity Levels in Transversal Direction-y

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 9 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.9.19., Fig. 3.9.20., and Fig 3.9.21., respectively.

![Fig 3.9.19. Computed Pick Relative Storey Displacements of Building No. 9 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y](image)
The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 9 under different earthquake intensity levels in transversal direction-\(y\) are presented in Table 3.8.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.9.2.** Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 9 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)

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<tr>
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EQI – El-Centro

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EQI – Prishtina Synthetic

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e) The Predicted Seismic Vulnerability Functions of Building No. 9, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 9 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.9.22., Fig. 3.9.23. and Fig. 3.9.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Figure 3.9.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 9. in Direction-y Under Ulcinj- Albatros earthquake](image_url)

*Figure 3.9.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 9. in Direction-y Under Ulcinj- Albatros earthquake*
3.9.4. Comparative Presentation of Damage Propagation Through SE & NE of Masonry Building No. 9. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results of the building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro & EQ-3 Prishtina Synthetic), behaviour of SE and NE can be described as follows:

1. Building stiffness along longitudinal direction x is higher than along the other direction. This results with the fact that collapse takes place along the direction with the lower stiffness – transversal direction y; for all PGA values up to the total collapse, SE and NE suffer same damage propagation through all earthquake stages.

2. Participation of NE in the overall building stiffness along both orthogonal directions is small compared to SE.

3. Regardless of the overall building stiffness, collapse takes place simultaneously on SE and NE on the first level.

4. Damage propagation of SE is larger than NE (3.16% SE and 2.78 NE).
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<th>EQ2 = 0.09G</th>
<th>EQ3 = 0.10G</th>
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<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
<th>EQ11 = 0.50G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage</td>
<td>Propagation to the West, S. E. &amp; N.E.</td>
<td></td>
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<table>
<thead>
<tr>
<th>EQ1 (B4y)</th>
<th>EQ2 = 0.025G</th>
<th>EQ3 = 0.09G</th>
<th>EQ4 = 0.10G</th>
<th>EQ5 = 0.15G</th>
<th>EQ6 = 0.20G</th>
<th>EQ7 = 0.25G</th>
<th>EQ8 = 0.30G</th>
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<th>EQ12 = 0.50G</th>
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<th>EQ3 = 0.10G</th>
<th>EQ4 = 0.15G</th>
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<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
<th>EQ11 = 0.50G</th>
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<tbody>
<tr>
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<td>Propagation to the West, S. E. &amp; N.E.</td>
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<th>EQ4 = 0.15G</th>
<th>EQ5 = 0.20G</th>
<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
<th>EQ11 = 0.50G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage</td>
<td>Propagation to the West, S. E. &amp; N.E.</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQ4 (B4x)</th>
<th>EQ1 = 0.025G</th>
<th>EQ2 = 0.09G</th>
<th>EQ3 = 0.10G</th>
<th>EQ4 = 0.15G</th>
<th>EQ5 = 0.20G</th>
<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
<th>EQ11 = 0.50G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Damage</td>
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<table>
<thead>
<tr>
<th>EQ4 (B4y)</th>
<th>EQ1 = 0.025G</th>
<th>EQ2 = 0.09G</th>
<th>EQ3 = 0.10G</th>
<th>EQ4 = 0.15G</th>
<th>EQ5 = 0.20G</th>
<th>EQ6 = 0.25G</th>
<th>EQ7 = 0.30G</th>
<th>EQ8 = 0.35G</th>
<th>EQ9 = 0.40G</th>
<th>EQ10 = 0.45G</th>
<th>EQ11 = 0.50G</th>
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<tbody>
<tr>
<td>Damage</td>
<td>Propagation to the West, S. E. &amp; N.E.</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

**Damage Propagation Troudh SE & NE of Masonry Building No. 9. for El-Centro Earthquake in Longitudinal Direction-x**

**Damage Propagation Troudh SE & NE of Masonry Building No. 9. for Prishtina Synthetic Earthquake in Longitudinal Direction-x**

**Damage Propagation Troudh SE & NE of Masonry Building No. 9. for Ulcinj - Albatros Earthquake in Transversal Direction-y**

EQI1 = 0.025G, EQI2 = 0.05G, EQI3 = 0.10G, EQI4 = 0.15G, EQI5 = 0.20G, EQI6 = 0.25G, EQI7 = 0.30G, EQI8 = 0.35G, EQI9 = 0.40G, EQI10 = 0.45G, EQI11 = 0.50G
3.9.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 9. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 9, the following general conclusions can be derived:

(1) In Fig. 3.9.7 and Fig. 3.9.18 for capacity diagrams of SE and NE along the longitudinal and transversal directions, it is visible that participation of NE in the overall building stiffness is negligible. Also these diagrams show the variation of stiffness along both orthogonal directions, where stiffness along the longitudinal direction x is much higher than the one along the transversal direction y. As a result, building collapses along the direction with the lower stiffness (y), for PGA = 0.25g.

(2) Displacements of separate structural elements on each level are different and depend on the overall building stiffness along orthogonal directions. Displacements can be easily compared by viewing the numeric results under the Ulcinj-Albatros earthquake on the top level along the longitudinal direction x the displacement is 1.201cm, and along the direction y it is 2.064cm (for PGA = 0.25g).

(3) Building collapse happens on PGA = 0.25g in longitudinal x-direction. This is because of the present different storey stiffness and storey strength for directions x and y. Large stiffness along the longitudinal direction x can be visible that under the El-Centro and Pristine Synthetic earthquakes building collapses for final PGA values, even though it reaches total destruction for PGA – 0.25g along the transversal direction y under the impact of Ulcinj-Albatros earthquake.

(4) Under the impact of Ulcinn-Albatros earthquake along the longitudinal direction x collapse takes place on the second level in SE and NE.

(5) Total loss is 5.94% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 3.16% and non-structural elements with 2.78%.
3.10. Seismic Vulnerability Analysis of Building No. 10 in Longitudinal Direction-\(x\) and Transversal Direction-\(y\)

3.10.1. Description of basic characteristics of the building structural system

It is a private house. Building consists of Ground and two floors. It consists of Basement, ground and first floor. It was constructed around 1950. It is important to mention that this building has a small footprint and corresponds to a large number of individual housing buildings in Pristine with similar structure.

Fig. 3.10.1. Building No. 10: Private House No. 10,Ymer Alishani str.

Fig. 3.10.2. Building No. 10: Floor plan

Floor plan of the building with dimensions (12.00 x 8.50)m, shown in Fig. 3.10.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

On the longitudinal direction, along “\(x\)” axis, there are three linear load baring walls, 4.5m, and 3.50 apart, and on the latitudinal direction, along “\(y\)” axis, there are six linear walls with
different distances among each other. The building consists of basement floor (2.80m high) ground and first floor (3.10m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). Walls at the basement level are with stones and are 60cm thick, and brick walls with a thickness of 38cm are on all other levels. Brick dimensions 25x12x6.5cm and are bricked with cement plaster. Walls are properly interconnected during bricking (without bond beams). Structural wall sections with parapets and spandrels are treated as non-structural elements.

3.10.2. Seismic Vulnerability Analysis of Building No. 10 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 10 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

![Diagram showing structural wall and earthquake forces](image)

Fig. 3.10.3 Building No. 10: Part of Individual Wall Segments A-A, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Mathematical model used for vulnerability analysis of Building No. 10 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Diagram showing non-linear MC model for direction-x](image)

Fig. 3.10.4. Building No. 10: Non-Linear MC Model for Direction-x

![Diagram showing mode shapes](image)

Fig. 3.10.5. Building No. 10: Mode Shape-1, Direction-x; $T_{1x}=0.187$ sec

Fig. 3.10.6. Building No. 10: Mode Shape-2, Direction-x; $T_{2x}=0.072$ sec
In Fig. 3.10.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.10.5, and Fig. 3.10.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

\[ \text{b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 10 for Longitudinal Direction-x} \]

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.10.7.

\[ \text{c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 10 Under Different Earthquake Intensity Levels in Longitudinal Direction-x} \]

To obtain full evidence in the most important response parameters of Building No. 10 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 10 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.10.8., Fig. 3.10.9., and Fig. 3.10.10., respectively.
Fig. 3.10.8. Computed Pick Relative Storey Displacements of Building No. 10 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig. 3.10.9. Computed Pick Relative Storey Displacements of Building No. 10 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig. 3.10.10. Computed Pick Relative Storey Displacements of Building No. 10 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 10 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 10 under different earthquake intensity levels in longitudinal direction-x are presented in Table 3.10.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.10.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 10 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.043 0.089 0.179 0.254 0.364 0.521 12.971 18.707 18.286 17.857 14.571</td>
</tr>
<tr>
<td>2</td>
<td>0.100 0.203 0.406 0.635 0.919 5.371 12.252 17.581 17.284 16.861 13.926</td>
</tr>
<tr>
<td>1</td>
<td>0.126 0.255 0.510 0.777 1.216 5.587 12.481 17.855 17.574 17.171 14.229</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>3</td>
<td>0.057 0.111 0.214 0.286 0.361 1.704 4.636 5.543 5.282 5.843 8.786</td>
</tr>
<tr>
<td>2</td>
<td>0.135 0.268 0.555 0.900 1.258 2.239 4.732 5.706 5.458 5.916 8.432</td>
</tr>
<tr>
<td>1</td>
<td>0.171 0.345 0.694 1.181 1.716 2.532 5.006 5.942 5.632 6.106 8.610</td>
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</table>

<table>
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<tr>
<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
<td>3</td>
<td>0.039 0.079 0.154 0.204 0.279 1.650 4.675 2.596 0.443 0.186 0.207</td>
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<tr>
<td>2</td>
<td>0.090 0.181 0.358 0.471 0.800 2.126 4.794 3.116 1.042 0.526 0.581</td>
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<tr>
<td>1</td>
<td>0.113 0.226 0.448 0.587 0.958 2.448 4.955 3.481 1.216 0.661 0.732</td>
</tr>
</tbody>
</table>

e) The predicted Seismic Vulnerability Functions of Building No. 10, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 10 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.10.11, Fig. 3.10.12, and Fig. 3.10.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.10.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 10 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.10.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 10 in Direction-x Under El-Centro earthquake

Fig. 3.10.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 10 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.10.3. Seismic Vulnerability Analysis of Building No. 10 Transversal Direction-\(y\)

a) Formulation of Non-Linear Mathematical Model of Building No. 10 in Transversal Direction-\(y\) and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 10 in transversal direction, Fig. 3.10.14.

![Diagram](image)

*Fig. 3.10.14 Building No. 10: Part of Individual Wall Segments 1-1, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-\(y\)*

Mathematical model used for vulnerability analysis of Building No. 10 in direction-\(y\) is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Diagram](image)

*Fig. 3.10.15. Building No. 10: Non-Linear MC Model for Direction-\(y\)*

In Fig. 3.10.15, shown is the formulated mathematical model of the building consisting of three concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.10.16, and Fig. 3.10.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 10 for Transversal Direction-y

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.10.18.

![Envelop curves for structural behavior](image1)

Fig. 3.10.18, Envelope curves for structural behavior.

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 10 Under Different Earthquake Intensity Levels in Transversal Direction-y

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 10 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.10.19., Fig. 3.10.20., and Fig. 3.10.21., respectively.

![Computation Pick Relative Storey Displacements of Building No. 10 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y](image2)

Fig 3.9.19. Computed Pick Relative Storey Displacements of Building No. 10 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y
**d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 10 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)**

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 10 under different earthquake intensity levels in transversal direction-\(y\) are presented in Table 3.8.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.10.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 10 Under Different Earthquake Intensity Levels in Transversal Direction-\(y\)**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
<th>0.50g</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.046</td>
<td>0.089</td>
<td>0.164</td>
<td>0.393</td>
<td>0.732</td>
<td>1.118</td>
<td>1.343</td>
<td>1.411</td>
<td>1.654</td>
<td>1.807</td>
<td>2.264</td>
</tr>
<tr>
<td>2</td>
<td>0.106</td>
<td>0.213</td>
<td>0.465</td>
<td>0.939</td>
<td>1.706</td>
<td>2.632</td>
<td>3.690</td>
<td>4.500</td>
<td>6.765</td>
<td>8.806</td>
<td>11.274</td>
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<tr>
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<td>0.271</td>
<td>0.629</td>
<td>1.229</td>
<td>2.165</td>
<td>3.368</td>
<td>4.719</td>
<td>7.545</td>
<td>9.439</td>
<td>17.335</td>
<td>19.348</td>
</tr>
</tbody>
</table>
e) The Predicted Seismic Vulnerability Functions of Building No. 10, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 10 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.10.22., Fig. 3.10.23. and Fig. 3.10.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.10.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 10. in Direction-y Under Ulcinj-Albatros earthquake
3.10.4. Comparative Presentation of Damage Propagation Through SE & NE of Masonry Building No. 10. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results for building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro and EQ-3 Prishtina Synthetic), behaviour of SE and NE within the structure can be explained as follows:

1. Along longitudinal direction x, collapse takes place in SE and NE for PGA = 0.25g.
2. In Building No.10, regardless of the overall building stiffness, collapse takes place always on the first level and simultaneously in SE and NE.
3. Damage propagation at the collapse peak is larger in NE (2.25%) compared to SE (3.45%).
Longitudinal Direction- x

Transverse Direction- y

Damage Propagation Troudh SE & NE of Masonry Building No. 10. for Ulcinj - Albatros Earthquake in

EQI1 = 0.025G  EQI2 = 0.05G  EQI3 = 0.10G  EQI4 = 0.15G  EQI5 = 0.20G  EQI6 = 0.25G  EQI7 = 0.30G  EQI8 = 0.35G  EQI9 = 0.40G  EQI10 = 0.45G  EQI11 = 0.50G

Damage Propagation Troudh SE & NE of Masonry Building No. 10. for Pristina Synthetic Earthquake in
3.10.4. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 10. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 10, the following general conclusions can be derived:

1. Capacity diagrams of SE and NE shown in Fig. 3.10.7 and Fig. 3.10.18 along the longitudinal direction x, show that overall building stiffness is larger along the transversal direction y. Participation of NE in the overall building capacity is very small and without impact in the overall stiffness.

2. Displacements of separate structural elements at various levels are different and depend on the overall building stiffness for the respective directions. As building consists of three levels. Displacements are not very different along orthogonal directions, where under the Ulcinj-Albatros earthquake impact at the collapse peak, displacements along the transversal direction y is 1.044cm and along the x direction is 1.732cm for PGA = 0.25g. This can be the reason of total building collapse for small PGA differences along orthogonal directions x and y.

3. Along the transversal direction y collapse takes place under the impact of Ulcinj-Albatros and El-Cedro earthquake impacts, for PGA = 0.25g. Building collapses on the first level.

4. Total loss is 3.45% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.25% and non-structural elements with 3.45%. Collapse takes place for low values of damage propagation.
3.11. Seismic Vulnerability Analysis of Building No. 11 in Longitudinal Direction-x and Transversal Direction-y

3.11.1. Description of basic characteristics of the building structural system

It is a residential building, used for public housing. Following the privatization process of public buildings, it is now privately owned by occupants. These buildings were renovated several times in the past. There is a considerable number of this type of buildings around Pristina – totally 14. They consist of (B+G+2+A). Initially these buildings were used as dormitory, with single-room apartment without toilets inside (Toilets were constructed outside the building). At a later stage, the building was modified into larger apartments with toilets. Assessment is made taking into consideration current condition of the building.

Fig. 3.11.1. Building No. 11: Residential Building, Building No. 11, “Bloc No. 1” Nazim Gafurri str.

Fig. 3.11.2. Building No. 11: Floor plan
Floor plan of the building with dimensions (20.68 x 11.49)m, shown in Fig. 3.11.2, has an orthogonal shape with load bearing constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.

On the longitudinal direction, along "x" axis, there are three linear load bearing walls, and on the transversal direction - y, there are five linear walls with different distances among each other. The building consists of ground floor (3.31m high) ground and for all stories (4 x 3.19m high). Load-bearing walls are made of stones and bricks. Connection points of load bearing walls on two directions are strengthened with our self (masonry, connected). Walls at the basement level are with stones and are 50cm thick, and brick walls with a thickness of 38cm are on all other levels. Brick dimensions are 25x12x6.5cm and are bricked with cement plaster.

3.11.2. Seismic Vulnerability Analysis of Building No. 11 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 11 in Longitudinal Direction-x and Structural Dynamic Characteristics

Fig. 3.11.3 Building No. 11: Part of Individual Wall Segments 3-3, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Fig. 3.11.4. Building No. 11: Non-Linear MC Model for Direction-x

Fig. 3.11.5. Building No. 11: Mode Shape-1, Direction-x; $T_{1x}=0.372$ sec

Fig. 3.11.6. Building No. 11: Mode Shape-2, Direction-x; $T_{2x}=0.128$ sec
Mathematical model used for vulnerability analysis of Building No. 11 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.11.4, shown is the formulated mathematical model of the building consisting of five concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.11.5, and Fig. 3.11.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 11 for Longitudinal Direction-x**

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.11.7.

![Fig. 3.11.7 Envelope curves for structural behavior.](image_url)

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 11 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

To obtain full evidence in the most important response parameters of Building No. 11 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 11 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.11.8., Fig. 3.11.9., and Fig 3.6.10., respectively.
Fig 3.11.8. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig 3.11.9. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.11.10. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
**d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 11 Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 11 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.11.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.11.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 11, Under Different Earthquake Intensity Levels in Longitudinal Direction-x**

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e) The predicted Seismic Vulnerability Functions of Building No. 11, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 11 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.11.11, Fig. 3.11.12, and Fig. 3.11.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through
the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.11.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 11 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.11.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 11 in Direction-x Under El-Centro earthquake

Fig. 3.11.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 11 in Direction-x Under Prishtina Synthetic, Earthquake
3.11.3. Seismic Vulnerability Analysis of Building No. 11 Transversal Direction-\(\text{y}\)

**a) Formulation of Non-Linear Mathematical Model of Building No. 11 in Transversal Direction-\(\text{y}\) and Structural Dynamic Characteristics**

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 11 in transversal direction, Fig. 3.11.14.

![Building No. 11: Part of Individual Wall Segments 1-1, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-\(\text{y}\)](image)

Mathematical model used for vulnerability analysis of Building No. 11 in direction-\(\text{y}\) is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Fig. 3.11.15. Building No. 11: Non-Linear MC Model for Direction-\(\text{y}\)](image)

![Fig. 3.11.16. Building No. 11: Mode Shape-1, Direction-\(\text{y}\); \(T_{1y}=0.522\) sec](image)

![Fig. 3.11.17. Building No. 11: Mode Shape-2, Direction-\(\text{y}\); \(T_{2y}=0.175\) sec](image)

In Fig. 3.11.15, shown is the formulated mathematical model of the building consisting of five concentrated masses interconnected with two principal elements for each storey representing...
non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.11.16, and Fig. 3.11.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 11 for Transversal Direction-y**

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.11.18.

*Fig. 3.11.18, Envelope curves for structural behavior.*

c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 11 Under Different Earthquake Intensity Levels in Transversal Direction-y**

To obtain full evidence in the most important response parameters of Building No. 11 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 11 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.11.19., Fig. 3.3.20., and Fig 3.6.21., respectively.
Fig 3.11.19. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y

Fig 3.11.20. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y

Fig 3.11.21. Computed Pick Relative Storey Displacements of Building No. 11 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 11 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 11 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.11.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

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<td>Computed Inter-Story Drift ISD (%) in Transversal Direction-y</td>
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e) The Predicted Seismic Vulnerability Functions of Building No. 11, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 11 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.11.22., Fig. 3.11.23. and Fig. 3.11.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through
the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

Fig. 3.11.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 11. in Direction-y Under Ulcinj- Albatros earthquake

Fig. 3.11.23 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 11. in Direction-y Under El-Centro earthquake

Fig. 3.11.24 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 11. in Direction-y Under Prishtina Synthetic earthquake
Damage Propagation Troubleshooting of Masonry Building No. 11 for El-Centro Earthquake in

**Longitudinal Direction-x**

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**Transverse Direction-y**

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3.11.4. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 11. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 11, the following general conclusions can be derived:

(1) Capacity diagrams of SE and NE shown in Fig. 3.10.7 and Fig. 3.10.18 along the longitudinal direction x, show that overall building stiffness is larger along the transversal direction y. Participation of NE in the overall building capacity is very small and without impact in the overall stiffness.

(2) Displacements of separate structural elements at various levels are different and depend on the overall building stiffness for the respective directions. As building consists of three levels. Displacements are not very different along orthogonal directions, where under the Ulcinj-Albatros earthquake impact at the collapse peak, displacements along the transversal direction y is 1.044cm and along the x direction is 1.732cm for PGA = 0.25g. This can be the reason of total building collapse for small PGA differences along orthogonal directions x and y.

(3) Along the transversal direction y collapse takes place under the impact of Ulcinj-Albatros and El-Cedro earthquake impacts, for PGA = 0.25g. Building collapses on the first level.

(4) Total loss is 3.45% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.25% and non-structural elements with 3.45%. Collapse takes place for low values of damage propagation.

3.12.1. Description of basic characteristics of the building structural system

The building serves for private housing. It consists of (B+G+1). It is important to mention that this building has a small footprint and corresponds to a large number of individual housing buildings in Pristine with similar structure.

Fig. 3.121. Building No. 12: Private House, Building No. 12, Ymer Alishani str.

Floor plan of the building with dimensions (18.00 x 13.20)m, shown in Fig. 3.12.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are three linear load baring walls, and on the latitudinal direction, along “y” axis, there are five linear walls with different distances among each other 4.40m. The building consists of basement floor (2.38m high) ground and first floor (3.10m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). Load-baring walls are made of stones and...
bricks. Walls at the basement level are with stones and are 38cm thick, and brick walls with a thickness of 38cm are on all other levels.

3.12.2. Seismic Vulnerability Analysis of Building No. 12 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 12 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

Mathematical model used for vulnerability analysis of Building No. 12 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.12.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively.

In Fig. 3.12.5, and Fig. 3.12.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 12 for Longitudinal Direction-x

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.12.7.

![Envelope curves for structural behavior.](image1)

Figure. 3.12.7 Envelope curves for structural behavior.

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 12 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 12 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.12.8., Fig. 3.12.9., and Fig. 3.12.10., respectively.

![Computed Pick Relative Storey Displacements of Building No. 12 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x](image2)

Fig 3.12.8. Computed Pick Relative Storey Displacements of Building No. 12 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x
Fig 3.12.9. Computed Pick Relative Storey Displacements of Building No. 12 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.12.10. Computed Pick Relative Storey Displacements of Building No. 12 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Longitudinal Direction-x

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 12 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 12 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.12.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.12.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 12 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Storey Drift ISD (%ø) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>3</td>
<td>0.101</td>
</tr>
<tr>
<td>2</td>
<td>0.174</td>
</tr>
<tr>
<td>1</td>
<td>0.219</td>
</tr>
</tbody>
</table>
e) The predicted Seismic Vulnerability Functions of Building No. 12, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 12 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.12.11, Fig. 3.12.12, and Fig. 3.12.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Figure 3.12.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 12 in Direction-x Under Ulqin – Albatros earthquake](image)

a) Formulation of Non-Linear Mathematical Model of Building No. 12 in Transversal Direction-y and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in transverse y-direction. The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 12 in transversal direction, Fig. 3.12.14.
Mathematical model used for vulnerability analysis of Building No. 12 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.12.15, shown is the formulated mathematical model of the building consisting of three concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.12.16, and Fig. 3.12.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 12 for Transversal Direction-y**

The computed maximum or “Pick-Response” relative storey displacements of Building No. 12 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.12.5. In the mentioned table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Prishtina Synthetic earthquake record (EQR).
c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 12 Under Different Earthquake Intensity Levels in Transversal Direction-y**

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 12 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.12.19., Fig. 3.12.20., and Fig 3.12.21., respectively.

![Figure 3.12.18](image1)

**Figure. 3.12.18, Envelope curves for structural behavior.**

![Figure 3.12.19](image2)

**Fig 3.12.19. Computed Pick Relative Storey Displacements of Building No. 12 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y**
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 12 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 12 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.12.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.12.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 12 Under Different Earthquake Intensity Levels in Transversal Direction-y**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (%‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g</td>
</tr>
<tr>
<td>3</td>
<td>0.063</td>
</tr>
<tr>
<td>2</td>
<td>0.123</td>
</tr>
<tr>
<td>1</td>
<td>0.155</td>
</tr>
</tbody>
</table>
The predicted direct analytical vulnerability functions of the integral Building No 12 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.12.22., Fig. 3.12.23. and Fig. 3.12.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

**EQI – El-Centro**

<table>
<thead>
<tr>
<th>Maximum Ground Acceleration (G)</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
<th>0.50g</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.071</td>
<td>0.143</td>
<td>0.286</td>
<td>0.466</td>
<td>0.815</td>
<td>1.248</td>
<td>1.597</td>
<td>1.887</td>
<td>2.319</td>
<td>2.651</td>
<td>2.929</td>
</tr>
<tr>
<td>2</td>
<td>0.145</td>
<td>0.290</td>
<td>0.584</td>
<td>0.919</td>
<td>1.397</td>
<td>1.987</td>
<td>2.497</td>
<td>2.952</td>
<td>3.552</td>
<td>4.000</td>
<td>4.681</td>
</tr>
<tr>
<td>1</td>
<td>0.190</td>
<td>0.384</td>
<td>0.768</td>
<td>1.223</td>
<td>1.890</td>
<td>2.732</td>
<td>3.435</td>
<td>4.310</td>
<td>4.981</td>
<td>5.658</td>
<td>6.539</td>
</tr>
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</table>

**EQI – Prishtina Synthetic**

<table>
<thead>
<tr>
<th>Maximum Ground Acceleration (G)</th>
<th>0.025g</th>
<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
<th>0.45g</th>
<th>0.50g</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.034</td>
<td>0.067</td>
<td>0.134</td>
<td>0.214</td>
<td>0.412</td>
<td>0.769</td>
<td>1.139</td>
<td>1.475</td>
<td>1.878</td>
<td>2.214</td>
<td>2.618</td>
</tr>
<tr>
<td>2</td>
<td>0.065</td>
<td>0.129</td>
<td>0.258</td>
<td>0.406</td>
<td>0.710</td>
<td>1.168</td>
<td>1.684</td>
<td>2.194</td>
<td>2.739</td>
<td>3.252</td>
<td>3.771</td>
</tr>
<tr>
<td>1</td>
<td>0.081</td>
<td>0.161</td>
<td>0.326</td>
<td>0.513</td>
<td>0.865</td>
<td>1.419</td>
<td>2.106</td>
<td>2.919</td>
<td>3.735</td>
<td>4.468</td>
<td>5.197</td>
</tr>
</tbody>
</table>

*e) The Predicted Seismic Vulnerability Functions of Building No. 12, Under the Effect of Three Selected Earthquakes in Transversal Direction-y*

The predicted direct analytical vulnerability functions of the integral Building No 12 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.12.22., Fig. 3.12.23. and Fig. 3.12.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
From the calculated results of building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro and EQ-3 Prishtina Synthetic), behaviour of SE and NE can be described as follows:

1. Along the longitudinal direction x, collapse of Se and NE takes place for PGA = 0.30g.
2. In Building No.12, regardless of the overall building stiffness, collapse takes place on the first and second level simultaneously for SE and NE.
3. Damage propagation at the collapse peak is higher in NE (10.98%) than in SE (6.65%). This level of usage of Building No.10 can be considered very high compared to the other buildings.
Damage Propagation Troudh SE & NE of Masonry Building No. 12. for Ulcinj-Albatros Earthquake in Direction-x

**EQI1** = 0.05G  **EQI2** = 0.10G  **EQI3** = 0.15G  **EQI4** = 0.20G  **EQI5** = 0.25G  **EQI6** = 0.30G  **EQI7** = 0.35G  **EQI8** = 0.40G  **EQI9** = 0.45G  **EQI10** = 0.50G

Damage Propagation Troudh SE & NE of Masonry Building No. 12. for El-Centro Earthquake in Direction-x

**EQI1** = 0.025G  **EQI2** = 0.05G  **EQI3** = 0.10G  **EQI4** = 0.15G  **EQI5** = 0.20G  **EQI6** = 0.25G  **EQI7** = 0.30G  **EQI8** = 0.35G  **EQI9** = 0.40G  **EQI10** = 0.45G  **EQI11** = 0.50G

Damage Propagation Troudh SE & NE of Masonry Building No. 12. for Prishtina Synthetic Earthquake in Direction-x

**EQI1** = 0.025G  **EQI2** = 0.05G  **EQI3** = 0.10G  **EQI4** = 0.15G  **EQI5** = 0.20G  **EQI6** = 0.25G  **EQI7** = 0.30G  **EQI8** = 0.35G  **EQI9** = 0.40G  **EQI10** = 0.45G  **EQI11** = 0.50G

Damage Propagation Troudh SE & NE of Masonry Building No. 12. for Prishtina Synthetic Earthquake in Direction-y

**EQI1** = 0.025G  **EQI2** = 0.05G  **EQI3** = 0.10G  **EQI4** = 0.15G  **EQI5** = 0.20G  **EQI6** = 0.25G  **EQI7** = 0.30G  **EQI8** = 0.35G  **EQI9** = 0.40G  **EQI10** = 0.45G  **EQI11** = 0.50G

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 12, the following general conclusions can be derived:

1. As seen in capacity diagrams of SE and NE shown in Fig. 3.12.7 and Fig. 3.12.18, along the transversal direction y, overall building stiffness is larger along the other direction x. Participation of NE in the overall building capacity is very small and without impact on the building stiffness.

2. Displacements of separate structural elements on various levels are different and depend on the overall building stiffness along the respective directions. Under the impact of Ulcinj-Albatros earthquake at the collapse peak, displacement along x direction is 3.385cm, and along the y direction it is 1.935cm for PGA = 0.30g. These displacements are large compared to the ones previously analysed.

3. Along the y direction collapse takes place under the impact of Ulcinj-Albatros earthquake for PGA = 0.30g. The building collapses simultaneously on the second and third level.

4. Total loss is 17.63% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 10.98% and non-structural elements with 6.65%. Building is resistant to the PGA valus, and has satisfactory response.

3.13.1. Description of basic characteristics of the building structural system

The building serves for private housing. It consists of (B+G+1). It is important to mention that this building has a small footprint and corresponds to a large number of individual housing buildings in Pristina with similar structure.

![Building No. 13: Private House, Ymer Alishani str.](image)

**Fig. 3.13.1. Building No. 13: Private House, Ymer Alishani str.**

**Fig. 3.13.2. Building No.1: First floor plan, identical to ground floor plan**

Floor plan of the building with dimensions (9.20 x 10.40)m, shown in Fig. 3.13.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

On the longitudinal direction, along “x” axis, there are three linear load baring walls, also on the latitudinal direction, along “y” axis, there are three linear walls with different distances among each other. The building consists of ground floor (2.84m high) + first floor (3.10m high). Connection points of load baring walls on two directions are strengthened with reinforced concrete non-structural columns. All structural walls are bricked with solid clay.
bricks with dimensions 25x12x6 cm joined with mortar and have a constant width of 38cm. There are partition walls as non-structural elements on each floor. Structural wall sections with parapets and spandrels are treated as non-structural elements.

3.13.2. Seismic Vulnerability Analysis of Building No. 13 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 13 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

![Image](image_url)

Fig. 3.13.3 Building No.13: Part of Individual Wall Segments A-A Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-x

Mathematical model used for vulnerability analysis of Building No. 13 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Image](image_url)

Fig. 3.13.4. Building No. 13: Non-Linear MC Model for Direction-x

In Fig. 3.13.4, shown is the formulated mathematical model of the building consisting of two concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.13.5, and Fig. 3.13.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 13 for Longitudinal Direction-x

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.13.7.

![Fig. 3.13.7 Envelope curves for structural behavior.](image)

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 13 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 13 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 13 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.13.8., Fig. 3.13.9., and Fig. 3.13.10., respectively.

![Fig. 3.13.8. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x](image)
Fig 3.13.9. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig 3.13.10. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x

d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 13 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 13 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.13.3. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.13.3. Computed Maximum (“Peak-Response”) Inter-storey drift (ISD) of Building No. 13 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>NP</th>
<th>Index of inter-storey drift, displacement (‰) – Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EQI - Ulcinj – Albatros N-S</td>
</tr>
<tr>
<td>1</td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>2</td>
<td>0.068 0.132 0.268 0.400 0.555 0.755 0.974 1.197 1.429 1.755 2.129</td>
</tr>
</tbody>
</table>
e) The predicted Seismic Vulnerability Functions of Building No. 13, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 13 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.13.11, Fig. 3.13.12, Fig. 3.13.13 and Fig. 3.13.14.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

a) Formulation of Non-Linear Mathematical Model of Building No. 13 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 13 in transversal direction, Fig. 3.13.15.
The formulated non-linear mathematical model which is used for vulnerability analysis of Building No. 13 in direction-y includes separately non-linear behavior characteristics of structural and non-structural elements consequently in all existing building stories.

In Fig. 3.13.15, shown is the formulated mathematical model of the building consisting of two concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.13.16, and Fig. 3.13.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

**b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 13 for Transversal Direction-y**

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.13.19.

**c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 13 Under Different Earthquake Intensity Levels in Transversal Direction-y**

To obtain full evidence in the most important response parameters of Building No. 13 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form.
Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 13 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.13.19, Fig. 3.13.20, and Fig. 3.13.21., respectively.

Fig. 3.13.19. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y

Fig. 3.13.20. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y

Fig. 3.13.21. Computed Pick Relative Storey Displacements of Building No. 13 Under Different Intensity Levels of Pristins-Synthetic Earthquake in Transversal Direction-y
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building NO. 13 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building NO. 13 under different earthquake intensity levels in transversal direction-y are presented in Tab. 3.13.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.13.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 13 Under Different Earthquake Intensity Levels in Transversal Direction-y

<table>
<thead>
<tr>
<th>EQR-1: Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
<td>1 0.049 0.102 0.204 0.384 0.606 0.937 1.254 1.458 1.944 2.373 3.630</td>
</tr>
<tr>
<td></td>
<td>2 0.061 0.123 0.255 0.439 0.665 0.994 1.316 1.535 2.035 2.494 3.729</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQR-2: El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
<td>1 0.067 0.134 0.254 0.616 0.954 1.394 1.613 2.215 2.426 3.433 3.683</td>
</tr>
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<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
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<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
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<td>2 0.068 0.135 0.268 0.400 0.594 0.890 1.171 1.477 1.784 2.042 2.629</td>
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e) The Predicted Seismic Vulnerability Functions of Building NO. 133, Under The Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 13 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.13.22., Fig. 3.13.23. and Fig. 3.13.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.13.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building N0.1. in Direction-y Under Ulcinj-Albatros earthquake

Fig. 3.13.23. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 13. in Direction-y Under El-Centro earthquake

Fig. 3.13.24. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 13. in Direction-y Under Prishtina Synthetic earthquake
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<th>EQI8 = 0.35G</th>
<th>EQI9 = 0.40G</th>
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<th>EQI11 = 0.50G</th>
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<tbody>
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<td><strong>EQI2 = 0.05G</strong></td>
<td><strong>EQI3 = 0.10G</strong></td>
<td><strong>EQI4 = 0.15G</strong></td>
<td><strong>EQI5 = 0.20G</strong></td>
<td><strong>EQI6 = 0.25G</strong></td>
<td><strong>EQI7 = 0.30G</strong></td>
<td><strong>EQI8 = 0.35G</strong></td>
<td><strong>EQI9 = 0.40G</strong></td>
<td><strong>EQI10 = 0.45G</strong></td>
<td><strong>EQI11 = 0.50G</strong></td>
</tr>
</tbody>
</table>

**Damage Propagation Troudh SE & NE of Masonry Building No. 13. for Pristina synthetic Earthquake in Longitudinal Direction-x**

**Damage Propagation Troudh SE & NE of Masonry Building No. 13. for El-Centro Earthquake in Transversal Direction-y**

**Damage Propagation Troudh SE & NE of Masonry Building No. 13. for Ulcinj - Albatros Earthquake in Transversal Direction-y**
3.13.4. Comparative Presentation of Damage Propagation Through SE&NE of Masonry Building No. 13. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results of building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro and EQ-3 Prishtina Synthetic), behaviour of SE and NE can be explained as follows:

1. Along the transversal direction y, collapse of SE and NE takes place for PGA = 0.45g, under the impact of El-Centro earthquake.
2. In Building No.13, regardless of its overall stiffness, collapse takes place on the first level simultaneously on SE and NE along the transversal direction y, and along the longitudinal direction it resists all the way the earthquake impacts.
3. Damage propagation at the collapse peak is larger in NE (5.05%) than in SE (2.68%), and for Building No.13 it is considered that this level of usage is high compared to the other analysed buildings. This is because this building can be treated as small.

3.13.5. General Remarks on Predicted Seismic Vulnerability of Masonry Building No.13. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 13, the following general conclusions can be derived:

1. Building collapse happens on PGA = 0.45g in referent direction Y. For this PGA value we can conclude that this building has small base dimensions and along the longitudinal direction it resists all the way, but along the transversal direction y it collapses under high PGA values.
2. Under the impact of El-Cedro earthquake at the collapse peak, displacement along the longitudinal direction x is 2.626cm, and along the other direction is 1.106cm for PGA = 0.45g. As can be seen, even though displacements are larger along the longitudinal direction x, collapse takes place along the transversal direction y.
3. Along the transversal direction y collapse takes place under the impact of El Centro earthquake for PGA = 0.45g. The building collapses simultaneously on the first level.
4. Total loss is 7.72% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.68% and non-structural elements with 5.05%. The building resists for PGA values, and it has satisfactory response for the level of use.

3.14.1. Description of basic characteristics of the building structural system

The building serves for residential use, for public housing. Following the privatization process of public buildings, it is now privately owned by occupants. The building was renovated several times in the past. It consists of (B+G+1).

It is important to mention that these buildings were constructed by prisoners of WWII.

Fig. 3.14.1. Building No. 14: Residential Building, No. 14, Sylejman Vokshi str.

Floor plan of the building with dimensions (42.00 x 13.50)m, shown in Fig. 3.14.2, has an orthogonal shape with load baring constructive walls on both directions, and partition walls as non-structural elements.

Fig. 3.14.2. Building No. 14: Floor plan
On the longitudinal direction, along “x” axis, there are four linear load baring walls, 5.81m, and 1.88m apart, and on the latitudinal direction, along “y” axis, there are eleven linear walls with different distances among each other. The building consists of basement floor (2.90m high) ground and first floor (3.50m high). Connection points of load baring walls on two directions are strengthened with our self (masonry, connected). Walls are of bricks and have thickness of 50cm on all levels. Walls are properly interconnected during bricking (without bond beams). Structural wall sections with parapets and spandrels are treated as non-structural elements.


a) Formulation of Non-Linear Mathematical Model of Building No. 14 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

Mathematical model used for vulnerability analysis of Building No. 14 in direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.
In Fig. 3.14.4, shown is the formulated mathematical model of the building consisting of three concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.14.5, and Fig. 3.14.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 14 for Longitudinal Direction-x

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.14.7.

![Figure. 3.14.7 Envelope curves for structural behavior.](image)

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 14 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 14 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 14 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.14.8., Fig. 3.14.9., and Fig. 3.14.10., respectively.
Fig. 3.14.8. Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig. 3.14.9. Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig. 3.14.10. Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 14 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 14 under different earthquake intensity levels in longitudinal direction-x are presented in Tab. 3.14.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).


<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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<tr>
<td>0.025g</td>
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<table>
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<table>
<thead>
<tr>
<th>EQI – Pristina Synthetic</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-x</th>
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</thead>
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</table>

e) The predicted Seismic Vulnerability Functions of Building No. 14, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 14 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.14.11, Fig. 3.14.12, and Fig. 3.14.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost radio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.14.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 14 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.14.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 14 in Direction-x Under El-Centro earthquake

Fig. 3.14.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 14 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.10.3. Seismic Vulnerability Analysis of Building No. 14 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 14 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 14 in transversal direction, Fig. 3.14.14.

Mathematical model used for vulnerability analysis of Building No. 14 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

In Fig. 3.14.15, shown is the formulated mathematical model of the building consisting of three concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.14.16, and Fig. 3.14.17, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.
b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 14 for Transversal Direction-y

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.14.18.

![Graph of Force vs. Displacement](image)

*Fig. 3.14.18, Envelope curves for structural behavior.*

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 14 Under Different Earthquake Intensity Levels in Transversal Direction-y

Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 14 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.14.19., Fig. 3.14.20., and Fig. 3.14.21., respectively.

![Graph of Displacement vs. Maximum Ground Acceleration](image)

*Fig 3.14.19, Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-y*
**Fig 3.14.20. Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-y**

**Fig 3.14.21. Computed Pick Relative Storey Displacements of Building No. 14 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-y**

**d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 14 Under Different Earthquake Intensity Levels in Transversal Direction-y**

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 14 under different earthquake intensity levels in transversal direction-y are presented in Table 3.14.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

**Tab. 3.14.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 14 Under Different Earthquake Intensity Levels in Transversal Direction-y**

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
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<th>0.05g</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.25g</th>
<th>0.30g</th>
<th>0.35g</th>
<th>0.40g</th>
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<th>0.50g</th>
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### EQI – El-Centro

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<th>0.15g</th>
<th>0.20g</th>
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<th>0.30g</th>
<th>0.35g</th>
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### EQI – Prishtina Synthetic

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### e) The Predicted Seismic Vulnerability Functions of Building No. 14, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building No. 14 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.14.22., Fig. 3.14.23. and Fig. 3.14.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.

![Ulcinj-Albatros Earthquake](image-url)

**Fig. 3.14.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 14. in Direction-y Under Ulcinj-Albatros earthquake**
3.14.4. Comparative Presentation of Damage Propagation Trough SE&NE of Masonry Building No. 14. in Case of Three Considered Earthquakes in Directions - x & y

From the calculated results of building vulnerability under three Earthquakes (EQ-1 Ulcinj-Albatros, EQ-2 El-Centro and EQ-3 Prishtina Synthetic), behaviour of SE and NE can be described as follows:

1. Along the longitudinal direction x, collapse of SE and NE takes place for PGA = 0.15g under the impact of Ulcinj-Albatros earthquake.
2. Regardless to the overall building stiffness, collapse takes place on the second level simultaneously in SE and NE along the longitudinal direction x, and along the transversal direction y, building resists up to PGA = 0.30G.
3. Damage propagation at the collapse peak is approximately equal in NE (2.71%) and SE (2.68%).
<table>
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<th>EQI2</th>
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<th>EQI8</th>
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<td>0.50</td>
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</table>

Damage Propagation Troudh SE & NE of Masonry Building No. 14 for Ulcinj - Albatros Earthquake in

Longitudinal Direction-x

Transversal Direction-y

Prishtina Synthetic Earthquake in

Transversal Direction-y

Image source: [Image](image_url)

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 14, the following general conclusions can be derived:

1. Building collapse happens on PGA = 0.15g in referent direction x. This takes place since along the longitudinal direction x the building is much longer than along the other direction (Fig. 3.14.2) what can also be seen in the capacity diagrams for the corresponding directions (Fig. 3.14.7 and Fig. 3.14.18) because along the transversal direction y there is a larger number of supporting walls than along the longitudinal direction x.

2. Under the impact of Ulcinj-Albatros earthquake, at the collapse peak displacements for direction x are 2.228cm and along the direction y they are 0.715 for PGA = 0.15g. As can be seen displacements at the immediate collapse peak are quite small.

3. Total loss is 5.39% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 2.68% and non-structural elements with 2.71%. The building resists earthquake impacts with PGA values, and performs satisfactory towards the level of usage.
3.15. Seismic Vulnerability Analysis of Building No. 15 in Longitudinal Direction-\(x\) and Transversal Direction-\(y\)

3.15.1. Description of basic characteristics of the building structural system

Building serves as an apartment building for collective housing. These buildings were renovated several times in the past. There is a considerable number of this type of buildings around Pristine – totally 7. They consist of (B+G+2+A). It has to be mentioned that an additional floor was added to these buildings in year 2000. Assessment includes current condition of the building.

\[\text{Floor Plan of the building with dimensions (15.50 x 10.40)m, shown in Fig. 3.15.2, has an orthogonal shape with load baring constructive walls on longitudinal - x and transversal - y directions, and partition walls as non-structural elements.}\]
On the longitudinal direction, along “x” axis, there are three linear load baring walls, and on the transversal direction - y, there are nine linear walls with different distances among each other. The building consists of basement floor (2.67m high) ground and for all storyes (3 x 3.31m high). Load-baring walls are made of stones and bricks. Walls at the basement level are with stones and are 50cm thick, and brick walls with a thickness of 38cm are on all other levels. Brick dimensions are 25x12x6.5cm and are bricked with cement plaster. Walls are properly interconnected during bricking.

3.15.2. Seismic Vulnerability Analysis of Building No. 15 Longitudinal Direction-x

a) Formulation of Non-Linear Mathematical Model of Building No. 15 in Longitudinal Direction-x and Structural Dynamic Characteristics

Based on in-site building inspection, component descriptions, measurement and respective office work defined are appropriate data (including geometrical and material characteristics) of all structural and non-structural elements acting in longitudinal x-direction.

Mathematical model used for vulnerability analysis of Building No. 15 in longitudinal direction-x is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.
In Fig. 3.15.4, shown is the formulated mathematical model of the building consisting of four concentrated masses and of two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Fig. 3.15.5, and Fig. 3.15.6, presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 15 for Longitudinal Direction-x

The calculated initial stiffness $K_0$, and respective force and displacement values for above specified points are presented in Fig. 3.15.7.

---

![Image of envelope curves for structural behavior.](image)

Fig. 3.15.7 Envelope curves for structural behavior.

c) Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 15 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

To obtain full evidence in the most important response parameters of Building No. 15 in longitudinal x-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 15 in longitudinal x-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.15.8., Fig. 3.15.9., and Fig. 3.15.10., respectively.
Fig. 3.15.8. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Longitudinal Direction-x

Fig. 3.15.9. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of El-Centro Earthquake in Longitudinal Direction-x

Fig. 3.15.10. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of Prishtina-Synthetic Earthquake in Longitudinal Direction-x
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 15 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 15 under different earthquake intensity levels in longitudinal direction-x are presented in Table 3.15.1. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.15.1. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 15 Under Different Earthquake Intensity Levels in Longitudinal Direction-x

<table>
<thead>
<tr>
<th>EQI – Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (%) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.105  0.210  0.461  0.989  1.483  1.993  2.318  2.712  3.139  3.318  3.700</td>
</tr>
<tr>
<td>3</td>
<td>0.275  0.556  0.997  1.695  2.381  3.042  3.849  6.338  9.459 10.066 13.045</td>
</tr>
<tr>
<td>2</td>
<td>0.405  0.822  1.713  2.873  4.100  5.293  6.480  9.103 12.384 13.251 16.242</td>
</tr>
<tr>
<td>1</td>
<td>0.444  0.900  1.943  3.299  4.574  5.979  7.532 12.076 19.154 23.103 27.740</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – El-Centro</th>
<th>Computed Inter-Story Drift ISD (%) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.109  0.217  0.423  0.719  1.285  1.697  2.000  2.213  2.607  2.547  2.532</td>
</tr>
<tr>
<td>3</td>
<td>0.269  0.538  0.961  1.408  2.160  2.755  3.202  3.734  5.335  7.604  8.508</td>
</tr>
<tr>
<td>2</td>
<td>0.390  0.779  1.529  2.405  3.804  4.961  5.837  6.299  7.825 10.634 11.668</td>
</tr>
<tr>
<td>1</td>
<td>0.426  0.852  1.689  2.692  4.254  5.604  6.870  9.876 12.704 12.801 15.828</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Story Drift ISD (%) in Transversal Direction-x</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.075  0.150  0.303  0.652  1.120  1.566  1.993  2.453  2.779  3.090  3.127</td>
</tr>
<tr>
<td>3</td>
<td>0.190  0.384  0.764  1.190  1.852  2.495  3.106  3.849  5.314  7.526  9.837</td>
</tr>
<tr>
<td>2</td>
<td>0.284  0.568  1.148  1.997  3.218  4.529  5.689  6.958  8.257 10.218 12.399</td>
</tr>
<tr>
<td>1</td>
<td>0.311  0.625  1.269  2.236  3.674  5.187  6.541  8.082  9.773 12.471 15.653</td>
</tr>
</tbody>
</table>

e) The predicted Seismic Vulnerability Functions of Building No. 15, Under The Effect of Three Selected Earthquake in Longitudinal Direction-x.

The predicted direct analytical vulnerability functions of the integral Building No. 15 in x-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.15.11, Fig. 3.15.12, and Fig. 3.15.13.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements. In this particular case adopted is uniform cost distribution of structural and non-structural elements throughout the height of the building.
Fig. 3.15.11. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 15 in Direction-x Under Ulqin – Albatros earthquake

Fig. 3.15.12. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 15 in Direction-x Under El-Centro earthquake

Fig. 3.15.13. The Predicted Cumulative Seismic Vulnerability Functions (with participation SE and NE) of masonry building No. 15 in Direction-x Under Prishtina Synthetic – artificial Earthquake
3.15.3. Seismic Vulnerability Analysis of Building No. 15 Transversal Direction-y

a) Formulation of Non-Linear Mathematical Model of Building No. 15 in Transversal Direction-y and Structural Dynamic Characteristics

The derived such systematic and detailed data is further implemented for formulation of realistic non-linear mathematical model of Building No. 15 in transversal direction, Fig. 3.15.14.

![Diagram of Building No. 15](image)

**Fig. 3.15.14 Building No. 15: Part of Individual Wall Segments 1-1 & 11-11, Considered in Formulation of Non-Linear Multi-Component (MC) Mathematical Model for Direction-y**

Mathematical model used for vulnerability analysis of Building No. 15 in direction-y is based on the previous description of structural system as well as on characteristics of structural and non-structural elements.

![Mathematical Model Diagram](image)

**Fig. 3.15.15. Building No. 15: Non-Linear MC Model for Direction-y**

**Fig. 3.15.16. Building No. 15: Mode Shape-1, Direction-y; $T_{1y}=0.291$ sec**

**Fig. 3.15.17. Building No. 15: Mode Shape-2, Direction-y; $T_{2y}=0.101$ sec**

In Figure 3.15.15, shown is the formulated mathematical model of the building consisting of four concentrated masses interconnected with two principal elements for each storey representing non-linear stiffness properties and hysteretic non-linear behavior characteristics of structural and non-structural elements, respectively. In Figure 3.15.16, and Figure 3.15.17,
presented are in graphical form the calculated fundamental vibration mode shape-1 and mode shape-2 with corresponding vibration periods, respectively.

b) **Computed Basic Non-Linear Force-Displacement Envelope Curves For Structural and Non-Structural Elements of Building No. 15 for Transversal Direction-y**

To assure comparative evidence in resulting specific data the computed envelope curves are presented in graphical form in Fig. 3.15.18.

![Envelop curve for structural behavior](image)

**Fig. 3.15.18, Envelope curves for structural behavior.**

c) **Computed Maximum (Pick-Response) Relative Storey Displacements of Building No. 15 Under Different Earthquake Intensity Levels in Transversal Direction-y**

To obtain full evidence in the most important response parameters of Building No. 15 in transversal y-direction, the computed maximum or “Pick-Response” relative storey displacements under different earthquake intensity levels are presented in graphical form. Actually, from the performed in total 33 complete non-linear seismic response analyses of Building No. 15 in transverse y-direction, considering the selected three earthquake records: (1) EQR-1, Ulcinj-Albatros, component N-S, (2) EQR-2, El-Centro, component N-S and (3) EQR-3, Pristina Synthetic earthquake record, the computed relative storey displacements are presented in Fig. 3.15.19., Fig. 3.15.20., and Fig. 3.15.21., respectively.
Fig 3.15.19. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of Ulcinj-Albatros Earthquake in Transversal Direction-\(y\)

Fig 3.15.20. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of El-Centro Earthquake in Transversal Direction-\(y\)

Fig 3.15.21. Computed Pick Relative Storey Displacements of Building No. 15 Under Different Intensity Levels of Pristina-Synthetic Earthquake in Transversal Direction-\(y\)
d) Computed Maximum (Pick-Response) Inter-Storey Drift (ISD) of Building No. 15 Under Different Earthquake Intensity Levels in Transversal Direction-y

The computed maximum or “Pick-Response” Inter-Storey Drift (ISD) of Building No. 15 under different earthquake intensity levels in transversal direction-y are presented in Table 3.15.2. In the same table (in separate sub-tables) presented are the computed results for the case of all three considered input earthquake records: (1) Ulcinj-Albatros, component N-S, (2) El-Centro, component N-S and (3) Pristina Synthetic earthquake record (EQR).

Tab. 3.15.2. Computed Maximum (“Pick-Response”) Inter-Storey Drift (ISD) of Building No. 15 Under Different Earthquake Intensity Levels in Transversal Direction-y

<table>
<thead>
<tr>
<th>EQI - Ulcinj – Albatros N-S</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.142 0.281 0.562 0.861 1.210 1.569 1.704 2.457 2.880 4.693 6.052</td>
</tr>
<tr>
<td>3</td>
<td>0.263 0.532 1.109 1.713 2.314 3.353 4.677 7.659 12.003 13.151 16.233</td>
</tr>
<tr>
<td>2</td>
<td>0.350 0.707 1.662 2.792 3.846 5.088 6.435 10.583 16.695 18.311 22.535</td>
</tr>
<tr>
<td>1</td>
<td>0.387 0.779 1.804 3.166 4.411 5.650 7.085 11.332 17.438 19.103 23.438</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – El-Centro</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.146 0.292 0.558 0.798 1.079 1.393 1.670 1.685 1.734 1.753 1.910</td>
</tr>
<tr>
<td>3</td>
<td>0.278 0.553 1.051 1.529 2.063 2.668 3.589 4.598 5.949 8.653 8.347</td>
</tr>
<tr>
<td>2</td>
<td>0.369 0.737 1.438 2.202 3.163 4.263 5.746 7.432 8.350 11.121 11.610</td>
</tr>
<tr>
<td>1</td>
<td>0.405 0.813 1.553 2.420 3.541 4.837 6.492 8.375 9.033 11.982 12.486</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>EQI – Prishtina Synthetic</th>
<th>Computed Inter-Story Drift ISD (‰) in Transversal Direction-y</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.025g 0.05g 0.10g 0.15g 0.20g 0.25g 0.30g 0.35g 0.40g 0.45g 0.50g</td>
</tr>
<tr>
<td>4</td>
<td>0.101 0.202 0.393 0.670 0.929 1.199 1.468 1.663 1.899 2.382 2.610</td>
</tr>
<tr>
<td>3</td>
<td>0.190 0.384 0.779 1.263 1.773 2.293 2.837 3.707 5.565 7.308 9.100</td>
</tr>
<tr>
<td>2</td>
<td>0.260 0.514 1.060 1.888 2.843 3.804 4.792 5.840 8.136 10.861 12.734</td>
</tr>
<tr>
<td>1</td>
<td>0.287 0.568 1.157 2.042 3.230 4.384 5.589 6.737 8.955 11.610 13.474</td>
</tr>
</tbody>
</table>

e) The Predicted Seismic Vulnerability Functions of Building No. 15, Under the Effect of Three Selected Earthquakes in Transversal Direction-y

The predicted direct analytical vulnerability functions of the integral Building N0 7 in y-direction, expressing the total losses in percent of the total building cost for increasing the PGA levels, as final results from this analysis are obtained throughout completion of several subsequent steps, and presented in corresponding figures (Fig. 3.15.22., Fig. 3.15.23. and Fig. 3.15.24.). In this case, based on the gathered statistical information on participation of structural and non-structural elements on the overall cost of the masonry buildings, adopted is the cost ratio of 65% for structural elements and 35% for non-structural elements. Through the adapted ratio, defined are loss functions for structural and non-structural elements.
Fig. 3.15.22. The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 15. in Direction-y Under Ulcinj-Albatros earthquake

Fig. 3.15.23 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 15. in Direction-y Under El-Centro earthquake

Fig. 3.15.24 The Predicted Cumulative Seismic Vulnerability Function (with participation of SE & NE) of Masonry Building No. 15. in Direction-y Under Prishtina Synthetic earthquake
### Damage Propagation Through SE & NE of Masonry Building No. 15 for El-Centro Earthquake in Longitudinal Direction-x

**EQI Values:**
- EQI1 = 0.025G
- EQI2 = 0.05G
- EQI3 = 0.10G
- EQI4 = 0.15G
- EQI5 = 0.20G
- EQI6 = 0.25G
- EQI7 = 0.30G
- EQI8 = 0.35G
- EQI9 = 0.40G
- EQI10 = 0.45G
- EQI11 = 0.50G

### Damage Propagation Through SE & NE of Masonry Building No. 15 for Prishtina Synthetic Earthquake in Transversal Direction-y

**EQI Values:**
- EQI1 = 0.025G
- EQI2 = 0.05G
- EQI3 = 0.10G
- EQI4 = 0.15G
- EQI5 = 0.20G
- EQI6 = 0.25G
- EQI7 = 0.30G
- EQI8 = 0.35G
- EQI9 = 0.40G
- EQI10 = 0.45G
- EQI11 = 0.50G
3.15.4. General Remarks on Predicted Seismic Vulnerability of Masonry Building No. 15. Under The Effect of Three Considered Earthquakes in Directions - x & y

Based on obtained results from the performed seismic vulnerability study and presented seismic vulnerability functions and damage propagation for Building No. 15, the following general conclusions can be derived:

(1) Building collapse happens on PGA = 0.30g in referent direction y. By comparison of capacity diagrams along both orthogonal directions it can be seen that the stiffness is approximately same along both orthogonal directions (Fig. 3.15.7 and Fig. 3.15.18) therefore also along the longitudinal direction x collapse takes place on SE and NE for PGA = 0.35g.

(2) Along the transversal direction y collapse takes place under the Ulcinj-Albatros earthquake. Building collapses simultaneously on the second level. Thickness of the structural walls at the first level is 50cm, and at the higher levels the thickness is 38cm. This is the reason for collapse on the second level.

(3) Under the impact of Ulcinj-Albatros earthquake, displacements at the collapse peak for the longitudinal direction x are 2.493cm, and along the direction y are 2.345cm for PGA = 0.30g. Displacement values along two directions are very similar what means that building collapses simultaneously along both directions, even though it occurs for very small displacement values.

(4) Total loss is 8.83% from the total building cost in the collapse moment in case of Ulcinj Albatros earthquake acting in referent longitudinal direction-x, meanwhile structural elements take part in this loss with 4.76% and non-structural elements with 4.07%. The building with its stiffness, at the initial earthquake impact, resists the forces (for low PGA values) where small displacement.
Chapter 4

GENERAL CHARACTERISTICS OF EXPECTED SEISMIC VULNERABILITY OF MASONRY BUILDINGS CLASSIFIED IN FOUR REPRESENTATIVE BUILDING CLASSES.

Seismic Vulnerability analyses of 15 selected buildings were presented in chapter 3. From the calculated results for each building in particular, comparative analysis of results is presented for different groups of buildings [M.T. 2000]. The representative results are presented for classified buildings in four classes based on: (1) number of stories, (2) use – function, (3) construction and (4) base shape.

These classes are categorized in a way to define conditions for more unified representation of basic seismic safety characteristics of existing masonry building in Pristina city.

4.1. Classification 1: Seismic Vulnerability of analyzed Masonry Buildings, classified in building classes by Number of Stories.

Classification of considered buildings according to number of stories statistical is presented in table 4.1.

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>Number of Analyzed buildings</th>
<th>Total number of Selected Buildings</th>
<th>Percent of analyzed Buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>2</td>
<td>16</td>
<td>29.10</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>14</td>
<td>25.45</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>22</td>
<td>40.00</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>4</td>
<td>7.30</td>
</tr>
<tr>
<td>Total</td>
<td>15</td>
<td>55</td>
<td>100</td>
</tr>
</tbody>
</table>

a. Five storey Buildings

Displacements producing collapse of the buildings No. 6 and No. 7 under considered seismic impacts, are comparatively shown in table 4.2.
Table 4.2. Maximum computed relative Displacement for PGA value producing collapsed

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Max Displacement (cm)</th>
<th>PGA</th>
<th>Earthquakes</th>
<th>Collapse direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal x</td>
<td></td>
<td>Transversal y</td>
<td></td>
</tr>
<tr>
<td>No. 6</td>
<td>3.902</td>
<td>0.15 g</td>
<td>Ulcin Albatros, El-Centro, Pristina Synthetics</td>
<td>X &amp; Y</td>
</tr>
<tr>
<td>No. 11</td>
<td>2.539</td>
<td>0.10 g</td>
<td>Ulcin Albatros</td>
<td>X</td>
</tr>
</tbody>
</table>

From table 4.2, it is evident that maximum displacement value is calculated for building No. 6, even though height of building No. 11 is larger.

Table 4.3. Comparison the Damage Propagation for two analyzed with 5 stories (buildings No. 6 and No. 11)

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Collapse direction</th>
<th>Value of PGA Collapse</th>
<th>Earthquakes</th>
<th>Damaged Storey</th>
<th>Total Loss of SE + NE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 6</td>
<td>X</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>4</td>
<td>5.53</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td>2 and 4</td>
<td>8.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>4</td>
<td>6.93</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td>2</td>
<td>3.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 11</td>
<td>X</td>
<td>0.10 g</td>
<td>Ulcin Albatros</td>
<td>4</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>2 and 3</td>
<td>9.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td>2 and 3</td>
<td>21.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>2 and 3</td>
<td>9.48</td>
</tr>
</tbody>
</table>

Comparing damage propagation and collapse for two buildings (Figure 3.6.25 and Figure 3.11.25, Part II, Chapter 3), and presented results in table 4.3, it is evident that building No. 11 collapses earlier (for PGA=0.10g) than building No. 6, for which collapse PGA = 0.15 g.

b. Four storey Buildings

Collapse Displacements for three other four-storey buildings, under three earthquakes are cooperatively presented in Table 4.4.
Table 4.4. Maximum computed relative Displacement for PGA producing collapse

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Max Displacement (cm)</th>
<th>PGA</th>
<th>Earthquakes</th>
<th>Collapse direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal x</td>
<td>Transversal y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 3</td>
<td>4.149</td>
<td>2.123</td>
<td>0.20 g</td>
<td>Pristina Synthetics</td>
</tr>
<tr>
<td>No. 7</td>
<td>2.073</td>
<td>2.237</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 15</td>
<td>2.493</td>
<td>2.345</td>
<td>0.30 g</td>
<td>Ulcin Albatros</td>
</tr>
</tbody>
</table>

Maximum building displacements correspond to PGA producing collapse for the corresponding direction. Table 4.4 shows that values of displacements the building collapse state are not high, having in mind height of buildings.

Table 4.5. Comparison the Damage Propagation for building No.3, No. 7 and No. 15

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Collapse direction</th>
<th>Value of PGA Collapse</th>
<th>Earthquakes</th>
<th>Damaged Storey</th>
<th>Total Loss of SE + NE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3</td>
<td>X</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>2 and 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>2 and 3</td>
<td>6.70</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.20 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>2 (S.E.)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 7</td>
<td>X</td>
<td>0.20 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>2</td>
<td>6.93</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 15</td>
<td>X</td>
<td>0.35 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>3 and 4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>3 (S.E.)</td>
<td>11.58</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.30 g</td>
<td>Ulcin Albatros</td>
<td>El Centro</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Among three four-storey buildings, for equal values of PGA=0.15g, absorbed is collapse of buildings No.3 and 7, but building No.15 resists higher earthquake intensity.
c. Three storey Buildings

Refer actual storey capacity diagrams we can group three storey buildings to the ones with higher capacity (buildings No. 14, 4, 12, 9, 8, 2) and building No.10, which has a lower story capacity respective direction.

Table 4.6. Maximum computed relative Displacement for PGA producing collapsed

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Max Displacement (cm)</th>
<th>PGA</th>
<th>Earthquakes</th>
<th>Collapse direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal x</td>
<td>Transversal y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 2</td>
<td>2.161</td>
<td>2.082</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 4</td>
<td>1.239</td>
<td>3.474</td>
<td>0.20 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 8</td>
<td>1.945</td>
<td>1.099</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 9</td>
<td>1.201</td>
<td>2.064</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 10</td>
<td>1.732</td>
<td>1.044</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 12</td>
<td>3.385</td>
<td>0.825</td>
<td>0.30 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td>No. 14</td>
<td>2.228</td>
<td>0.715</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
</tr>
</tbody>
</table>

Table 4.7. Comparison the Damage Propagation for building No.2, 4, 8, 9, 10, 12 and No. 15

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Collapse direction</th>
<th>Value of PGA Collapse</th>
<th>Earthquakes</th>
<th>Damaged Storey</th>
<th>Total Loss of SE + NE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 2</td>
<td>X</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>6.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td>2</td>
<td>6.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td>2</td>
<td>5.94</td>
</tr>
<tr>
<td>No. 2</td>
<td>Y</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>8.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 4</td>
<td>X</td>
<td>0.45</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>17.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 4</td>
<td>Y</td>
<td>0.20 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>4.41</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
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<td></td>
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<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 8</td>
<td>X</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>6.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
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<td></td>
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<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 8</td>
<td>Y</td>
<td>0.35 g</td>
<td>Ulcin Albatros</td>
<td>1 and 3</td>
<td>7.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>X</td>
<td>0.40 g</td>
<td>Ulcin Albatros</td>
<td>1 and 2</td>
<td>15.53</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
<td>--------</td>
<td>----------------</td>
<td>-----------</td>
<td>-------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y</td>
<td>0.25</td>
<td></td>
<td>Ulcin Albatros</td>
<td>1</td>
<td>5.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>X</td>
<td>0.25 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>12.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y</td>
<td>0.35 g</td>
<td></td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>3.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>X</td>
<td>0.30 g</td>
<td>Ulcin Albatros</td>
<td>2 and 3</td>
<td>17.63</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y</td>
<td>0.50 g</td>
<td></td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>5.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>X</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>2</td>
<td>5.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y</td>
<td>0.30 g</td>
<td></td>
<td>Ulcin Albatros</td>
<td>1</td>
<td>6.59</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Even though collapse takes place the computed values of relative displacements are considerably low.

Table 4.7 shows that buildings are exposed to collapse under relatively low PGA values, with the exception of building No.14, which is shows lower safety than others. Collapse of this building takes place as a result of the existing large mass compared to other buildings in this category.

Table also shows that collapse takes place in the first or in the second storey, and always under the impact of considered Ulcin Albatros earthquake record.
d. Two storey Buildings

Table 4.8. Maximum computed relative Displacement for PGA value producing collapsed

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Max Displacement (cm)</th>
<th>PGA</th>
<th>Earthquakes</th>
<th>Collapse direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal x</td>
<td>Transversal y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 1</td>
<td>0.500</td>
<td>1.793</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El-Centro</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
</tr>
<tr>
<td>No. 5</td>
<td>2.278</td>
<td>0.237</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El-Centro</td>
<td>X</td>
</tr>
<tr>
<td>No. 13</td>
<td>0.814</td>
<td>1.103</td>
<td>0.45 g</td>
<td>El-Centro</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Y</td>
<td></td>
</tr>
</tbody>
</table>

The computed relative displacements producing building collapse are presented in table 4.8. For this class of two-storey buildings it is also evident that relative displacements in the collapse stage are relatively low.

Table 4.9. Comparison Damage Propagation for building No.1, No. 5 and No. 13

<table>
<thead>
<tr>
<th>Number of Buildings</th>
<th>Collapse direction</th>
<th>Value of PGA Collapse</th>
<th>Earthquakes</th>
<th>Damaged Storey</th>
<th>Total Loss of SE + NE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1</td>
<td>X</td>
<td>0.30</td>
<td>Ulcin Albatros</td>
<td>1</td>
<td>8.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>1</td>
<td>8.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td>3.88</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td>3.72</td>
</tr>
<tr>
<td>No. 5</td>
<td>X</td>
<td>0.15 g</td>
<td>Ulcin Albatros</td>
<td>1 and 2</td>
<td>5.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td>4.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.40g</td>
<td>Ulcin Albatros</td>
<td>1</td>
<td>8.48</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pristina Synthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 13</td>
<td>X</td>
<td></td>
<td>Ulcin Albatros</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>El Centro</td>
<td>1</td>
<td>7.72</td>
</tr>
</tbody>
</table>

Total collapse of buildings in this class takes place for low PGA values, with the exception of building No.13 which has smaller base dimensions and in the analysis is treated as a small building.
Finally it can be concluded from the above conducted theoretical analysis and presented results that all buildings analyzed collapse under relatively low intensities of earthquake impacts. This observation is direct confirmation of the expected level of intolerable vulnerability of this type of buildings which are constructed in the past basically as non-seismic buildings.

Table 4.10. Damage propagation and collapse PGA for buildings classified by the number of storey.

<table>
<thead>
<tr>
<th>PGA</th>
<th>0.10 g</th>
<th>0.15 g</th>
<th>0.20 g</th>
<th>0.25 g</th>
<th>0.30 g</th>
<th>0.45 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of storey Building</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Build</td>
<td>No. 11</td>
<td>No. 6</td>
<td>No. 3</td>
<td>No. 7</td>
<td>No. 14</td>
<td>No. 4</td>
</tr>
<tr>
<td>no</td>
<td>14</td>
<td>1</td>
<td>6+1=7</td>
<td>6</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>1+6+2+5=14</td>
<td>1</td>
<td>7</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Percent</td>
<td>25.45 %</td>
<td>1.80 %</td>
<td>12.73 %</td>
<td>10.91 %</td>
<td>3.60 %</td>
<td>25.45 %</td>
</tr>
</tbody>
</table>

Table 4.10 and figure 4.5, figure 4.6 and figure 4.7, leads us to conclude that buildings with larger number of storeys collapse under small PGA values, and lower buildings are more resistant to dynamic impacts. This however can not be accepted as a general rule in evaluation of collapse based on the number of storeys, since the building response depends on many other factors that can be leading to developed different damage level.

Figure 4.5. Distribution of damage for the classes based on number of the storyes under Ulcin Albatros earthquake, PGA = 0.10g, longitudinal x-direction and transversal y-direction
Figure 4.6. Distribution of damage for the classes based on number of storyes under Ulcin Albatros earthquake, PGA = 0.15 g, longitudinal x-direction and transversal y-direction

Figure 4.7. Distribution of damage for the classes based on number of storyes under Ulcin Albatros earthquake, PGA = 0.25 g, longitudinal x-direction and transversal y-direction

4.2. Classification 2: Seismic Vulnerability of analyzed Masonry Buildings, classified in building classes according to Usability.

Buildings selected for analysis in this class, as shown in table 1.1 part I, have the following functions: Education Buildings, Residential Buildings and Private Houses.

<table>
<thead>
<tr>
<th></th>
<th>Residential Buildings</th>
<th>Educational Buildings</th>
<th>Private House</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Buildings</td>
<td>49</td>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>
The category of existing residential buildings is considered to be highly important for dynamic analysis under earthquake impacts for Pristina city having in mind their large number and the interest to determine their level of stability, therefore the number of buildings selected in this category is large compared to Educational Buildings and Private Houses.

The building categories interesting for comparison can be defined based on their applied different structural systems.

Risk level of Residential Buildings is considerably high taking in consideration that these buildings have suffered occasional modifications, where partial or overall stability of the building was not considered. On the other hand, occupancy of these buildings is considerable. For example, from statistical data, in “Block 1” permanently are present about 1320 occupants. Similar situation is also present in other city areas.

Pristina, being a capital has around 500,000 inhabitants. This figure shows the existence of large number of Educational Buildings present today in this city. However, there is a small number of masonry educational buildings. Therefore are our study, we have not included more buildings in this category.

Contrary there is a large number of private houses built with masonry walls in Pristina. These buildings were constructed mainly before the 1960-s, and they are daily being replaced with new buildings. Therefore, their number is permanently reduced and in this study, considered small number of Private Houses. The other reason for this is the fact of observed higher stability of smaller buildings towards earthquake impacts.

Based on the calculated results for 15 analyzed buildings, it can be concluded that Residential Buildings are very vulnerable, collapse can be expected under low PGA values and losses are. There are many factors that have impact in the collapse of buildings in this category under the earthquake impacts, including number of storeys, footprint shape, quality of material, mass of mezzanine structures etc.

Regarding the Educational Buildings, it appears that buildings No.1 and No. 4 resist earthquake impacts better than residential buildings.
Private Houses No.10, 12 and 13, that mainly have small footprints and smaller number of storeys, appear to be more resistant to expected earthquake impacts.

### 4.3. Classification 3: Seismic Vulnerability of Analyzed Masonry Buildings, classified in building classes according to Quality of Construction.

In many cases of analyzed buildings, load bearing walls in the first storey are thicker than in the upper storeys. In these cases (tables 4.3, 4.5, 4.7 and 4.9) collapse takes place on the second storey, Contrary in the cases of buildings with constant thickness of walls, initial collapse takes place on the first storey. Exception to this pattern is in the overbuilt buildings with additional load in the last storey, where as a result collapse takes place simultaneously on the second and third storey (for example see table 4.5, building No.3).

There is no doubt that quality and type of construction have a primary importance for actual building capacity and resistance to external factors, including dynamic earthquake impacts.

For the buildings with the structure that has a different calculated stiffness and load bearing capacity in two directions, collapse always takes place along the direction that has a smaller stiffness and strength capacity.

In conducted analysis of all 15 selected buildings, non-structural elements were also considered in calculation of overall stiffness and strength of the building. However, for many buildings of this type, participation of non-structural elements in the overall building stiffness and strength is very low and in many cases this contribution was close to zero.

Based on evaluation of integral analysis results for buildings, it can be concluded that buildings with masonry walls in both orthogonal directions, bricked with clay bricks and plaster, have limited capacity for resistance are controlled absorption of dynamic earthquake impacts. These structures behave as highly stiff in the beginning of the earthquake, having low displacements, but with increase of force the critical stage the buildings is rapidly reached and collapse is commonly observed immediately after such stage.
4.4. Classification 4: Seismic Vulnerability of Analyzed Masonry Buildings, classified in building classes according to General Floor shape in base.

Calculated results of the dynamic response of all considered buildings show that the floor plan of the building – organization of masonry walls has impact in the building capacity to resist horizontal earthquake forces.

Comparing buildings with symmetric floor plans to the ones with no symmetry, it is evident that the response of buildings with symmetric floor plan to dynamic earthquake impacts is more favorable than of buildings with no symmetry.

Table 4.11 shows analyzed buildings, the base shape, PGA values and the building collapse direction. It can be seen that ratio of base dimensions Lx and Ly is not always valid for classification of level of damage. Building response basically depends on the internal organization of load bearing walls in both principal directions.

In case of Private Houses, favorable base shape of the building has resulted with improved safety (higher PGA values) as can be seen in the table.

If we compare building No.15 and building No.1. Dimensions ratio Lx/Ly for Building No.15 indicate base close to the square shape, and the base of Building No.1 has considerably larger length than width. It is observed that Building No.15 with a square base shows higher resistance to earthquake loads than Building No.1 with a rectangular base shape.
Table 4.11. Classes of Building according to the floor shape with defined collapse PGA and Collapse Direction

<table>
<thead>
<tr>
<th>Address Buildings</th>
<th>Number of Buildings</th>
<th>Usability</th>
<th>Storey</th>
<th>Dimensions of Base</th>
<th>Form Plane</th>
<th>PGA</th>
<th>Collapse Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Lx (m)</td>
<td>Ly (m)</td>
<td>Ratio Lx/Ly</td>
<td></td>
</tr>
<tr>
<td>B No. 13</td>
<td>2</td>
<td>Private House</td>
<td>2</td>
<td>9.20</td>
<td>10.40</td>
<td>0.88</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 12</td>
<td>1</td>
<td>Private House</td>
<td>3</td>
<td>18.00</td>
<td>13.20</td>
<td>1.36</td>
<td>Non Rectangular</td>
</tr>
<tr>
<td>B No. 10</td>
<td>1</td>
<td>Private House</td>
<td>3</td>
<td>12.00</td>
<td>8.50</td>
<td>1.41</td>
<td>Non Rectangular</td>
</tr>
<tr>
<td>B No. 15</td>
<td>7</td>
<td>Residential Buildings</td>
<td>4</td>
<td>15.50</td>
<td>10.40</td>
<td>1.49</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 5</td>
<td>1</td>
<td>Residential Buildings</td>
<td>2</td>
<td>27.80</td>
<td>17.00</td>
<td>1.64</td>
<td>Non Rectangular</td>
</tr>
<tr>
<td>B No. 3</td>
<td>6</td>
<td>Residential Buildings</td>
<td>4</td>
<td>18.53</td>
<td>11.74</td>
<td>1.78</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 11</td>
<td>14</td>
<td>Residential Buildings</td>
<td>5</td>
<td>20.68</td>
<td>11.49</td>
<td>1.80</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 4</td>
<td>1</td>
<td>Secondary School</td>
<td>3</td>
<td>26.60</td>
<td>14.60</td>
<td>1.82</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 8</td>
<td>2</td>
<td>Residential Buildings</td>
<td>3</td>
<td>18.50</td>
<td>10.00</td>
<td>1.85</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 7</td>
<td>1</td>
<td>Residential Buildings</td>
<td>4</td>
<td>21.10</td>
<td>9.50</td>
<td>2.22</td>
<td>Non Rectangular</td>
</tr>
<tr>
<td>B No. 6</td>
<td>1</td>
<td>Residential Buildings</td>
<td>5</td>
<td>25.11</td>
<td>10.95</td>
<td>2.29</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 9</td>
<td>6</td>
<td>Residential Buildings</td>
<td>3</td>
<td>22.96</td>
<td>10.00</td>
<td>2.30</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 2</td>
<td>5</td>
<td>Residential Buildings</td>
<td>3</td>
<td>2.9</td>
<td>9.20</td>
<td>2.72</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 14</td>
<td>6</td>
<td>Residential Buildings</td>
<td>3</td>
<td>42.50</td>
<td>14.00</td>
<td>3.04</td>
<td>Rectangular</td>
</tr>
<tr>
<td>B No. 1</td>
<td>1</td>
<td>Education Buildings</td>
<td>2</td>
<td>47.4</td>
<td>11.95</td>
<td>3.97</td>
<td>Rectangular</td>
</tr>
</tbody>
</table>
Chapter 5

IMPLEMENTATION OF THE PRESENT SEISMIC VULNERABILITY STUDY FOR DEVELOPMENT OF POSSIBLE SEISMIC DAMAGE SCENARIOS AND PLANNING OF SEISMIC RISK REDUCTION MEASURES.

In Chapter 3 presented are results and essential diagrams for all the selected buildings in order to show their structural behavior under the impact of different earthquakes. Deformations and damage of SE and NE for the selected buildings under the increasing intensity of three earthquakes are analyzed for 11 with changed intensity levels in small steps different PGA values, in order to capture effects of structural behavior under the earthquake impacts in short intervals.

Considering the computed results for all analyzed buildings, in this chapter will be analyzed characteristic of structural behavior, regardless of other characteristics, under considered equal impacts intensities. These comparative analysis is performed for PGA = 0.025g, PGA = 0.10g, PGA = 0.15g, and PGA = 0.25g.

5.1. Seismic damage Scenario – 1: Expected Seismic Damage Levels of analyzed buildings under Earthquake Intensity Characterized with PGA = 0.025g.

In order to have a clear picture of the building behavior under the impact of three earthquakes, from the previously computed results, damage propagation is shown below for PGA = 0.025g. Figure 5.5 shows damage propagation for PGA = 0.025g, for each building, in two orthogonal directions and for three earthquakes (EQ=1 Ulcin Albatros, EQ=2 El-Centro and EQ=3 Pristina Synthetic). In the case of initial stage characterized with small earthquake intensity all buildings in general behave very well and have sufficient capacity to resist generated seismic forces (small forces).

In this stage (small earthquake intensity) many of SE and NE are without damages (around 50%, are non damaged elements), and the rest of the buildings have SE and NE with initial small cracks (cracked elements, yellow color). Buildings No.3, No.8 and No.12 are exceptions, as their NE in this stage show light damages (Light Damaged Elements, green color).
If we analyze Building No.11, we can see small progress of damages along longitudinal direction x. Cracks appear only in the second level. Along transversal direction y, all SE and NE have initial cracks, what shows irrational distribution of loads in the buildings for two orthogonal directions.

Also for this PGA value, it can be observed that SE and NE of buildings with more stories receive initial cracks, as opposed to the buildings with small number of stories which in the case of this small earthquake intensity level.

Figure 5.1 shows damage propagation in SE and NE for all buildings, under the impact of Ulcin Albatros earthquake for both orthogonal directions x and y, (PGA = 0.025g). It is clear that for this small acceleration level most of building elements suffer low damage levels, characterized with appearance of only small initial cracks.

![Figure 5.1. Distribution of damage (in SE and NE) corresponding to PGA = 0.025g, in case of Ulcin Albatros Earthquake](image)

5.2. Seismic damage Scenario – 2: Expected Seismic Damage Levels of analyzed buildings under Earthquake Intensity Characterized with PGA = 0.10g.

Under the impact of three different earthquakes, and for the acceleration of 0.10g for all analyzed buildings, in figure 5.6 presented is damage propagation for each building and for both orthogonal directions x and y. SE and NE of each building relived change in damage propagation showing increased damage in represent to initial analyzed stage for PGA acceleration of 0.025g.

Slower damage propagation is visible in building No.13. In this building, for the acceleration value of 0.10g, SE and NE mostly suffer initial cracks. The reason of such behavior of this
building with a lower damage propagation is in the number of stories (Building No.13 has only 2 stories).

Buildings where participation of NE in overall capacity is low, for the same PGA level, always NE have a higher damage propagation than SE. In opposite case of buildings where NE have larger participation, damage propagation SE and NE is equal for the same PGA level.

In buildings that have different stiffness through stories (usually walls of the first level are thicker), damage propagation SE and NE, for the same acceleration level, usually are largest in the second level, where collapse usually takes place.

It is important to point out that for this acceleration level observed is collapse (DG 5, red color, Collapsed Elements) of SE and NE that in a multi-storey building. In building No.11 that has 5 levels, total collapse of SE and NE takes place on the 4th level along longitudinal direction x under the impact of Ulcin Albatros earthquake, even though SE and NE of other levels have lower level of damage (initial cracks – DG 2, yellow color).

While analyzing transversal direction y for the same earthquake, it can be observed that SE and NE have large damages. Even though SE and NE along transversal direction y for this acceleration value do not collapse, we consider that beyond this point the building suffers a total collapse.

Also for Building No.6 that has five levels, we can observe large damage propagation in SE and NE. For all three cases of earthquake impacts, SE and NE suffer damage propagation in the value of DG 3 and DG 4 (DG 3, green color, Light Damage of Elements, and DG 4, blue color, Heavy Damaged Elements). Following the same pattern we can analyze damage propagations of SE and NE in buildings with four, three and two levels. However for this acceleration value, none of the remaining buildings suffer total collapse. In conclusion, for this acceleration value, first collapsed buildings, or buildings with large damages of SE and NE are multi-storey buildings.
From Figure 5.2 we can observe new stage characterized by higher damage propagation, where many SE and NE of the buildings received damage level DG2 and DG3, meaning that they suffer small cracks and small damage levels.

**Figure 5.2. Distribution of damage (in SE and NE) corresponding to PGA = 0.10g, under Ulcin Albatros Earthquake**

**5.3. Seismic damage Scenario – 3: Expected Seismic Damage Levels of analyzed buildings under Earthquake Intensity characterized with PGA = 0.15g.**

Figure 5.7 shows damage propagations of each analyzed building for both orthogonal directions x and y under the impact of Ulcin Albatros, El Centro and Pristina Synthetic earthquake with PGA = 0.15g.

Damage propagation in SE and NE for this acceleration value is considerable. In a large number of analyzed buildings there is a total collapse of SE and NE. As can be seen figure 5.7, a total collapse of SE and NE appears in Buildings No.1, No.3, No.5, No.6, No.7, No.11 and No.14. Expressed in percentage, for this PGA value 46.67% of buildings suffer total collapse.

Damage propagation in SE and NE of buildings that resist this acceleration level under the impact of three earthquakes, can be seen in figure 5.7. It can be observed that buildings with more levels (mainly three storey buildings), suffer DG 3 and DG 4.

Usually collapse of SE and NE occur in the first level of the buildings with constant wall thickness in all levels (for example in Building No.1 collapse takes place in the first level),
and in other buildings that have thicker walls on the first level, collapse takes place on the second level.

Collapse of Building No.1 occurs as a result of base dimensions and stiffness along two orthogonal directions, and building No.5 collapses along the longitudinal direction x, where along the direction y, SE and NE have suffer DG 1 and DG 2. This occurs because of large stiffness difference along two orthogonal directions.

Damage propagation in SE and NE of Building No.13 has the level of DG 1, DG 2 and DG 3, showing that SE and NE have a capacity to absorb higher impacts. This occurs because the building is small, possess a small base and is low – has only two storeys. In figure 5.3. shown is damage propagation in SE and NE of buildings for this PGA level. In this case, a large number of SE and NE have reached the collapse and in other SE and NE elements observed are different DG damage levels.

![Figure 5.3. Distribution of damage (in SE and NE) corresponding to PGA = 0.15g, under Ulcin Albatros Earthquake](image)

5.4. Seismic damage Scenario – 4: Expected Seismic Damage Levels of analyzed buildings under Earthquake Intensity Characterized with PGA = 0.25g.

Figure 5.8 presents damage propagation for all analyzed buildings under the impact of three earthquakes along both orthogonal directions x and y, for PGA = 0.25g.
For this acceleration value, among the analyzed buildings, there is a total collapse in all buildings except Buildings No. 12, No.13 and No.15. Presented in percentage, 80% of buildings suffer total collapse under this acceleration value.

Buildings that have considerable stiffness and load bearing differences along orthogonal directions x and y, show that while collapse takes place along one direction. Elements of the other direction can absorb larger impacts, even though at the peak of collapse of some SE and NE in DG 5, we consider that the building suffers a total collapse [Tar. 97]. This phenomena can be used for creating suggestion to designers when design buildings with similar behaviour properties in both principal direction.

SE and NE for this acceleration level and for the same building storey collapse simultaneously. Usually in higher storeys elements have lower damage propagation.

We can see in figure 5.4 that a large percentage of SE and NE have reached the total collapse. From these diagrams that show the damage propagation in SE and NE, it is visible that there is an uniformity of damage propagation, even though there are some SE and NE that jump from DG 2 to DG 5 – total collapse.

![Figure 5.4. Distribution of damage (in SE and NE) corresponding to PGA = 0.25g, under Ulcin Albatros Earthquake](image)

5.5. Synthesis of Obtained Seismic Vulnerability results and planning of short-term and long-term Seismic Risk Reduction measures.

After analysis of damage propagation and respectively total collapse of all the analyzed buildings, it can be concluded that under the impact of earthquakes with increasing intensities in masonry buildings with clay bricks or stones are not proven resistant [Ta.Ao.G3 03].
These structures have sufficient capacity in the case of small earthquake intensities, where there are small displacements at the top of the buildings [Gu.Ka.Ya 05]. However with the increase of earthquake intensity, displacements are rapidly enlarged resulting with rapid local collapse of SE and NE in some critical storey in many cases, SE jump from low damage propagation (DG 1 or DG 2) to total collapse for the next increased acceleration level. This unfavorable behavior of masonry building should be avoided in the future with application of appropriate structural or detailing measures.
Figure 5.5. Damage propagation for All Buildings under PGA = 0.025g
Figure 5.6. Damage propagation for All Buildings under PGA = 0.10g
Figure 5.7. Damage propagation for All Buildings under PGA = 0.15g
Figure 5.8. Damage propagation for All Buildings under PGA = 0.25g
CONCLUSIONS AND RECOMMENDATIONS

Although natural phenomena can not be prevented, their effects can significantly be reduced with improvement of construction standards, more sophisticated land use policy and better vulnerability source identification of the main elements at risk or mitigation of consequences and their reconstruction.

Buildings that are exposed to earthquake impacts are challenged, and often we witness considerable human and property losses.

One common way to express the damageability of a structure is to utilize a so-called loss function (also referred to as a damageability function, vulnerability function, or damage function). In order to relate physical damage to buildings to other socio-economic issues, damage is expressed in terms of economic loss: the greater the damage the greater the loss. One common measure of damage is the cost to repair the structure divided by the replacement cost of the structure [Tom 03].

In order to reduce damages from earthquakes, we perform studies, experiments and various analysis, all giving results that can be used to increase stability of buildings, respectively higher the designed capacity of buildings, but also for sanation – strengthening of existing buildings.

The main objective of this PhD thesis was to develop method and to evaluate (calculate) the risk level and respectively vulnerability material loss of a number of buildings in Pristina city under the earthquake impact. For the present study purposes 55 Buildings have been selected and used for detailed analysis. In fact they present a small percentage of buildings in Pristina city but the study is providing highly important results for the future constructions. For all selected buildings based on used specific selection criteria (number of storeys, Usability, Quality of Construction, Floor Shape), calculated stiffness, response displacements, maximal and minimal forces on mezzanine levels, mode shapes, damage propagations as well as vulnerability functions and stage of total collapse.
In order to determine the damage propagation and losses of all existing masonry buildings, it is required to define the Analytical Vulnerability Function for each analyzed separate building. From the calculated results of analytical vulnerability functions for complete set of 15 analyzed buildings among 55 selected buildings, it is possible to derive the following conclusions and recommendations:

6.1. CONCLUSIONS

- Structural elements of masonry buildings that are exposed to earthquake impacts with different intensities, behave as stiff elements with very low ductility. Initially, there is a solid response with small horizontal displacements, but with the increase of earthquake intensity elements suffer cracks and suffer rapid collapse.
- Calculated losses of analyzed buildings show that, at the collapse stage peak level of observed loss is low. This leads to the conclusion that buildings have small losses under the low PGA values, but increase of earthquake intensity results with a rapid collapse of SE and NE, therefore the building has no more capacity to absorb additional forces and comes to a total collapse. This phenomena appears for the fact that these structures are not ductile.
- Multi-storey masonry buildings with load bearing walls, are vulnerable to earthquakes, compared to buildings with fewer levels. This appears also for the reason that these structures are massive and consist of composite materials that have poor tension capabilities.
- Building floor plans that have irregular shapes or have no symmetry, in cases where center of mass is far from center of rigidity, are not resistant to earthquake impacts, or are resistant for low PGA values only.
- Masonry buildings with base dimensions ratio far from 1 (buildings where one orthogonal dimension is considerably larger than the other), collapse much sooner than buildings with a base closer to a square.
- Function of the building defines sizes of internal areas. In educational buildings, internal areas are larger, what leads us to understand that also walls have limited stiffness, giving a disproportional stiffness along orthogonal directions x and y. As a result, we can say that along one of the orthogonal
directions (direction of the smaller stiffness and bearing capacity) there is larger damage propagation, compared to the other direction with smaller dimensions.

- In Residential Buildings load bearing walls are more dense in the base, and often stiffness is larger along the orthogonal direction with the smaller dimension, but cases of approximately equal dimensions of sides along orthogonal directions x and y (square base) result with approximately equal stiffness for each direction. As a result, building collapses along the orthogonal direction with a smaller stiffness and strenth.

- Type of construction of masonry buildings also has considerable impact on their capacity to absorb earthquake impacts. Wall planes with more openings are not resistant to earthquake impacts, compared to wall planes with no openings.

- Wall planes with openings (necessary for the purpose of the building), the wall section between openings bares a large vertical load, and as such presents a weak point in the load bearing wall.

- Variation of the wall thickness along storeys, as observed in numerous cases in the analyzed buildings (wall thickness on the first level is larger than in the upper levels) indicates that collapse does not take place on the first level, but on the upper levels (usually on the second level).

- Even though Pristina Synthetic earthquake used in the analysis is characterized with a low intensity and short impact duration, analyzed buildings still show no significant resistance. Therefore, these buildings have small capacities even for earthquakes with small time duration.

### 6.2. RECOMMENDATIONS

Based on the earlier conclusions, we can give the following recommendations for masonry buildings:

- Local authorities, leadership of Pristina Minicipality, but also authorities of other Kosovan cities, are recommended to issue regulations regarding existing masonry buildings, defining the risk factors regarding their usage, and to take steps towards their reconstruction.
• These regulations should also cover cases of overbuilt and change of destination (modifications on load bearing walls).
• In regards to the newly constructed masonry buildings, importance should be given to the form of construction of the load bearing masonry walls.
• In Pristina, there is a number of masonry buildings with a special historic importance. These buildings should be treated in the same manner as the buildings subject to this study.

a. Existing Buildings

• For newly constructed masonry buildings, it is highly recommended that along the wall planes exist horizontal and vertical reinforced concrete bond elements [Ri.Zi 97]. By strengthening the wall plane with horizontal and vertical reinforced concrete bond elements (beams and columns), we get compact structural elements with increased capacity to absorb earthquake impacts.
• In order to increase the capacity of these buildings, respectively their structural elements, we recommend reconstruction – strengthening of existing walls. Strengthening ways can be as follows:
  - Wrapping of existing walls with reinforced concrete layers that rest on reconstructed strip foundations on the level of existing foundations and are anchored on the mezzanine levels. Also, a cap flashing is concreted on top of the wall to join the concrete layers [S.F.P.A 91].
  - Reinforcement of building angles with steel profiles, and placement of steel ropes along the perimeter of the building in order to absorb tension forces.
  - Reinforcement of wall planes with carbon fibers diagonally (this case can be implemented only if mezzanine structures are reinforced concrete slabs).
• Many of existing masonry buildings in Pristina city have been subject to renovations (reconstructions, new openings, increasing existing openings, removal of wall sections on the lower floors etc.) that are quite often, considerably reducing buildings capacity to absorb earthquake impacts [Lo. 07].
• Overbuilding of masonry buildings is a frequent phenomena in town, what increases the mass on the top level of the building, what reduces overall capacity to absorb horizontal forces [Mi.Pe.Ri 96].

b. New constructed Buildings

• Base shape of the buildings should be symmetric, in order to have the center of mass and center of rigidity as close as possible to each other (this will avoid torsion impacts on structural elements).
• During the design stage of masonry buildings, it has to be kept in mind that stiffness along both orthogonal directions x and y should be approximately equal.
• It is in favor of the overall stability to have thicker walls on the first levels of the building, therefore it is always recommended that in cases where collapse is expected to take place on the first level, to implement thicker walls (this can be applied to unutilized basements).
• It is recommended that during the design stage of newly constructed masonry buildings, load bearing walls are confined with reinforced concrete elements that will absorb tension forces in the structure.
• Mezzanine structures of newly constructed masonry buildings should be constructed as reinforced concrete solid slabs, capable of absorbing horizontal loads. Ribbed reinforced concrete slabs are not recommended, especially for Educational buildings.
• Horizontally, maximal span between confinement elements should not exceed 5,0m, and vertically the span should not be larger than 3,0m between bond beams.
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