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SEISMIC EVALUATION AND REDUCTION OF THE SEISMIC RISK FOR A HISTORICAL BUILDING IN ITALY

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ABSTRACT:

The Italian historical buildings are mainly composed of masonry structures. These buildings were designed for gravitational loads only and for this reason they are particularly vulnerable to seismic actions, and naturally they do not comply with the current design code provisions. The devastating effects of large number of destructive earthquakes occurred in Italy during recent history have highlighted the need to redefine design strategies and requirements especially in high seismic risk regions. The Italian prescriptions for seismic evaluation and reduction of seismic risk of historical buildings, as well as the 2008 Italian Building Code (NTC2008) have been motivated by the general principals of reducing seismic risk of historical building from the perspective of avoiding structural damage and ensuring human life protection in the case of earthquakes.
This study focuses on the seismic evaluation and retrofitting of a multi-story masonry building called Palizzi Castle located in the Reggio Calabria Province, Italy. The main purpose is to present the seismic evaluation procedure for Level LV1 of seismic vulnerability to determine the safety coefficient $I_{S,SLV}$ and the acceleration factor $f_{a,SLV}$ for the life safety performance level. The study proposes a retrofitting and strengthening strategy for the building based on the prevention of its historical value.

KEYWORDS : Historical buildings, seismic evaluation, retrofitting by reinforced mortar

1. INTRODUCTION

Recent Italians earthquakes, as Umbria and Marche in 1997 [1] and Abruzzo in 2009 [2], have underlined the need of extensive monitoring and safety assessment of historical buildings in Italy. Seismic evaluation of historical buildings is a complex problem due to different aspects involved, such as the quality of the masonry, the structural systems, the economical and cultural implications.

On February 2011 Italian “Guidelines for the evaluation and reduction of seismic risk of buildings of architectural heritage” were published [3]. The publication of these guidelines has highlighted the importance of seismic assessment of historical masonry buildings. The above mentioned guidelines introduced the concept of seismic enhancement, at the same time intended as partial upgrading being able to improve the seismic performance of an historical building, and to respect the preservation requirements. The seismic enhancement is different from the classic full seismic retrofitting required by the technical guidelines for ordinary buildings [4]. The Italian Guidelines suggest an approach in three different phases: knowledge acquisition; seismic safety evaluation; structural intervention design.

This paper presents an evaluation of the seismic vulnerability of the Palizzi Castle located in the Reggio Calabria province (Italy). The three different phases of the seismic evaluation procedure for Level LV1 of seismic vulnerability are described. Several interesting considerations about strengthening measures are reported in order to assess the performance of some common interventions on historical masonry buildings, considering both the global response and local collapse modes.

2. CASE STUDY: PALIZZI CASTLE

2.1. Building description

The Palizzi Castle has medieval origins, and has been built on a rock site close to the Palizzi center (see Figure 1). Originally the Ruffo family had built the Palizzi castle in the XVI century; but along the centuries the castle was subjected to several modifications. Some of them were aimed at renewing the building according to the necessity of the time, such as change in geometry or numbers of rooms, other interventions were aimed at retrofitting the building. In 1866, it has been restored and extended by the Baron Tiberio De Blasio in its actual configuration.

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Figure 1. Several views of the Palizzi Castle in its current form.

The castle, as today, is composed of three different buildings: The main building which is composed of a ground floor, first floor, and under-roof level. The stable that originally was composed of two different floors; but today there are only three out of four external walls remaining. The third component guardhouse, that was also composed of two different levels; but today only its ruins remain. The presented study will be focusing only on the main building where part of the slabs and almost all roof has collapsed (see Figure 1).

The plan layout of the ground floor of the castle is shown in Figure 2, where it is possible to recognize the original medieval external walls that are surrounding the building. The total plan dimension of the main building is approximately 30.00 m by 16.50 m. The main structure is composed of external walls, and an internal wall that goes along the north-south direction. The walls are stone masonry walls with a variable thickness of 90-100 cm at the ground floor and 80 cm at the first floor. The internal wall is dividing the building into two main long rooms. The entrance of the building is not central (see Figure 2) but closer to the old part of the structure that is not an objective of the study presented. In the south façade, see Figure 3, there is a small tower made of simple bricks. The horizontal structure of the first floor is made of masonry vaults covered by a wood slab (see Figure 4), while the upper floor is a wood slab with transversal wood beam. The roof that today is almost completely collapsed was made of wood beams simply supported by the external and central walls.

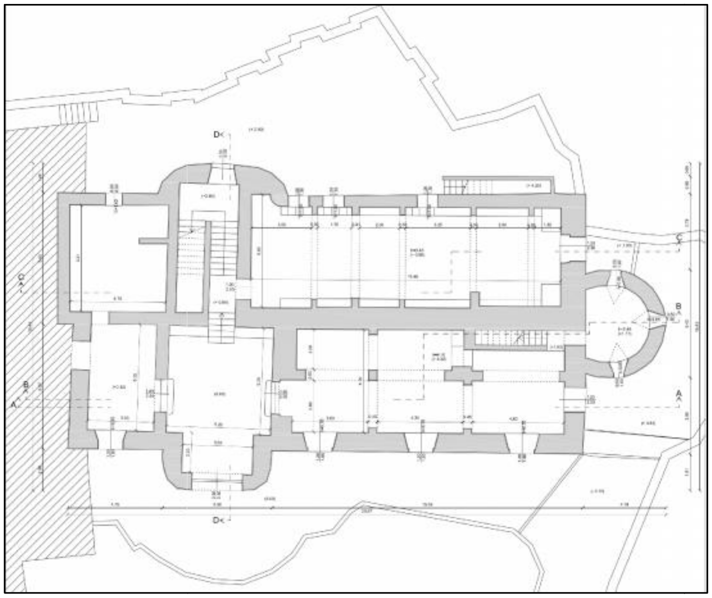

Figure 2. Palizzi Castle: plan layout of the ground floor.

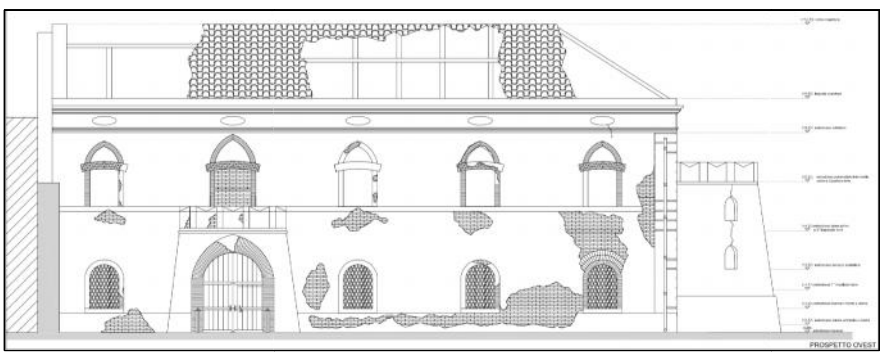

Figure 3. Palizzi Castle: View of the west façade.

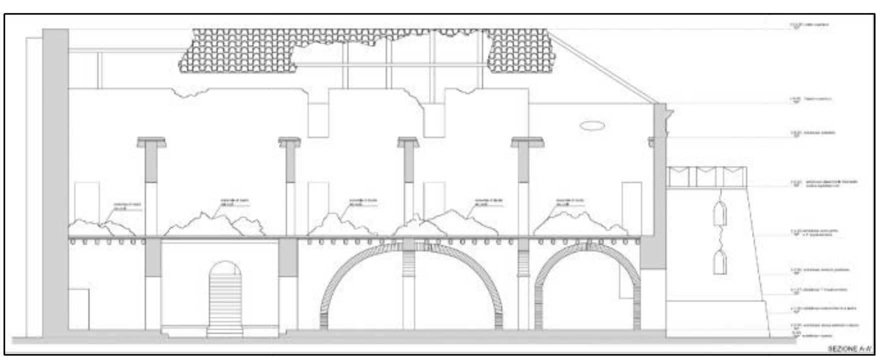

Figure 4. Palizzi Castle: Section A-A.

2.2. Knowledge of the building

The knowledge level of a building implies geometrical mapping, experimental investigation, and historical research. Generally geometrical mapping is easily carried out, while experimental investigations have to focus on the historical value of the building. Historical researches can be very useful to define the evolutionary process of the building, and its construction sequence.

The main aim of this phase is to define a model that allows to give a qualitative interpretation of the structural behavior. With the collected information, it will be possible to evaluate the seismic safety of the building. Once the knowledge collecting phase is completed, it is possible to define the confidence factor F_C which will be the material safety factor to be used for the seismic evaluation. This factor is calculated using the following equation

$$F_c = 1 + \sum_{k=1}^{4} F_{ck} \qquad (1)$$

where the values of the partial confidence factors (F_{ck}) for the case considered are reported in Table 1.

Table 1. Partial confidence factors (Circolare 2009 [5]).

Geometric survey	Complete geometric survey and graphic representation of cracks and deformations	$F_{c1}=0$
Material survey and constructive details	Limited survey of materials and constructive details	$F_{c2}=0.12$
Mechanical properties of material	Mechanical properties obtained by available data	$F_{c3}=0.12$
Geotechnical soil and foundation	Availability of geological and foundation structure data; limited investigations on soil and foundation	$F_{c4}=0.03$

The geometrical survey was conducted with a level of detail that includes also the description of crack and deformation, for this reason FC1 can be assumed equal to 0.

The aim of the material survey (masonry typology, slab typology, vault structure, etc.), and identification of construction details (connections between walls, possible weaknesses, type of slabs, and degree of connection with the walls, thrust reduction elements, material deterioration etc.) are done to characterize all the constructive typologies of the building, and their location with particular attention to the aspects that may trigger local collapse mechanisms. The value of FC2 can be assumed equal to 0.12 since we have a limited knowledge of the materials and constructive details.

Regarding the definition of FC3, it is important to emphasize that often different masonry typologies are used to realize the structure. In these cases, it seems correct to correlate the FC3 factor to the masonry typology which is most relevant for the seismic analysis. For the level of assessment required at this stage, mechanical tests on the materials was not allowed a value of 0.12 was assumed for FC3.

The definition of the FC4 depends on the influence that the foundation system can have on the collapse mechanisms: if the collapse mechanisms are assumed not influenced by the geotechnical parameters, it is possible to use FC4 = 0. Based on the availability of geological and foundation structure data, for FC4 a value of 0.03 can be assumed as a reasonable value.

Considering these values, the confidence factor Fc will be equal to 1.27.
The mechanical properties of the masonry used in the analysis are derived from the current code suggestions (Table C8.A.2.1- Circolare 2009 [5]). In particular, the castle walls are made of brick masonry and mortar. The mechanical properties based on the code suggestions and corrected by the confidence factor calculated are

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reported in Table 2.

Table 2. Mechanical properties of masonry.

Masonry Typology	f_m (N/mm^2)	τ_0 (N/mm^2)	E (N/mm^2)	G (N/mm^2)	W (kN/m^3)
Brick Masonryand mortar	2.52	0.60	1654	394	18

3. SEISMIC SAFETY EVALUATION

For the evaluation of the seismic safety the guidelines introducea new model based on three different levels of investigation: LV1: territorial-scale simplified seismic evaluation;LV2: seismic evaluation to be used in case of local interventions on a building; LV3: comprehensive evaluation of the seismic safety of a building.Since at this stage it was not possible to perform any test on the masonry wall to evaluate the resistance of the materials, a simplified analysis of Level LV1was performed.

LV1 allows evaluating the collapse acceleration of building type structures by using simplified models based on a limited number of geometrical and mechanical parameters or qualitative tools (visual test, construction features, and stratigraphic survey).

The seismic evaluation is based on the Seismic Safety Index (IS,SLV) obtained as the ratio between the limit state return period of the seismic action, TSLV, and the expected return period, TR,SLV, for the analyzed building

$$I_{S,SLV} = \frac{T_{SLV}}{T_{R,SLV}} \quad (2)$$

$$I_{S,SLV} = \frac{T_{SLV}}{T_{R,SLV}}$$

The second index that needs to be evaluated is the Acceleration Factor ($f_{a;SLV}$), obtained as the ratio between the ground acceleration for the specific limit state, a_{SLV}, and the expected acceleration, $a_{g,SLV}$, for the analyzed building

$$f_{a,SLV} = \frac{a_{SLV}}{a_{g,SLV}} \quad (3)$$

$$\cancel{f_{a,SLV} = \frac{a_{SLV}}{a_{g,SLV}}}$$

-All these values are calculated referring to the soil type A.

3.1. Level LV1- Seismic evaluation: a simplified model

If the building has a rigid behavior, and if the corner connections between the walls are in a good state, a simplified model can be used based on the NTC2008 code Based on the detailed visual inspection, no out of plane movement of the external walls was observed, and for this reason it was possible to apply the LV1 method, otherwise it would not be possible.

Referring to the considered limit state, Damage Limit State (SLD) or Life Safety Limit State (SLV), it is necessary to evaluate elastic acceleration response spectrum using the simplified model as

$$S_{e,SLV} = \frac{q\, F_{SLV}}{e^{*}\, M} \quad (4)$$

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where
• F_{SLV} is the building shear resistance
• q is the structural coefficient based on the NTC2008 and 2009 building code
• M is the total seismic mass
• e* is the modal mass participation for the fundamental mode
As an example, considering the X direction for a generic floor, the shear resistance can be evaluated based on the NTC2008 as

$$F_{SLV} = \frac{\mu_{xi}\ \xi_{xi}\ \zeta_{xi}\ A_{xi}\ \tau_{di}}{k_i\ \beta_{xi}} \qquad (5)$$

where
- A_{xi} is the shear resisting area of the walls along X direction for the i^{th} floor
- β_{xi} is the floor irregularity coefficient for the i^{th} floor which can be evaluated as follows

$$\beta_{xi} = 1 + \frac{e_{yi}\ d_{yi}\ A_{xi}}{\sum_k (y_k - y_{ci})^2 A_{xi,k}}; \qquad (6)$$

- μ_{xi} is the homogeneity stiffness coefficient for the i^{th} floor which is equal to

$$\mu_{xi} = 1 - 0.2\sqrt{\frac{N_{mxi}\sum_j A_{xi,j}^2}{A_{xi}^2} - 1} \geq 0.8 \qquad (7)$$

where N_{mxi} is the numbers of walls along the X direction for the i^{th} floor
– ξ_{xi} is a coefficient that depends on the type of collapse mechanism, this coefficient is equal to 1 for shear collapse and equal to 0.8 for combined axial and flexural collapse. For the case in hand, a shear type collapse mechanism is considered
– ζ_{xi} is a coefficient that depends on the total wall resistance of the i^{th} floor along X-direction. It can have a value between 0.8 and 1.0 in function of the walls resistance
– τ_{di} it is the shear resistance of the walls at the i^{th} floor expressed as

$$\tau_{di} = \tau_{0d}\sqrt{1 + \frac{\sigma_{0i}}{1.5\tau_{0d}}} \qquad (8)$$

where τ_{0d} is the calculated shear resistance the confidence factor FC and σ_{0i} is the vertical average resistance along the resisting surface of the walls at the i^{th} floor.

Figure 5 and 6 show the identified masonry walls along the X and Y directions for Level 1 and Level 2, respectively.

Figure 5. Identified masonry elements for Level 1 along X and Y direction.

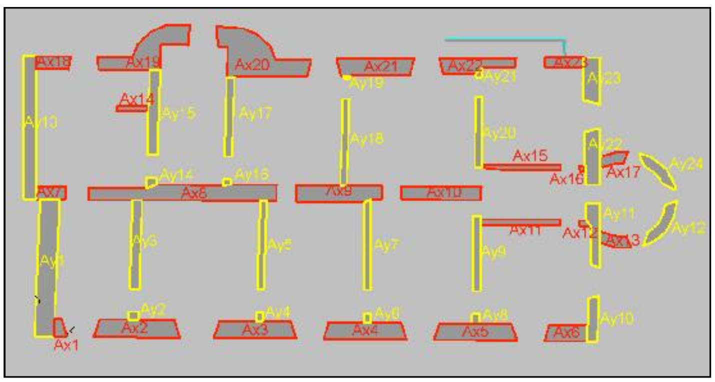

Figure 6. Identified masonry elements for Level 2, along X and Y direction.

Table 3 shows the calculated values of the shear resistance ($F_{SLV,xi}$) and elastic response spectrum ($S_{e,SLV}$) for Level 1 and Level 2 along X and Y directions.

Table 3. Shear resistance and elastic acceleration response spectrum values.

X direction				Y direction			
		Level 1	Level 2			Level 1	Level 2
W_i	(kN)	69803	51395	W_i	(kN)	69803	51395
q	(-)	1.00		q	(-)	1.00	
e*	(-)	0.96		e*	(-)	0.96	
k_i	(-)	1	0.66	k_i	(-)	1	0.66
σ_{0i}	(N/cm^2)	13.13	7.29	σ_{0i}	(N/cm^2)	13.13	7.29
τ_{di}	(N/cm^2)	9.39	8.05	τ_{di}	(N/cm^2)	9.39	8.05
A_{xi}	(m^2)	48.88	42.85	A_{yi}	(m^2)	37.92	27.53
y_{ci}	(m)	6.44	6.99	x_{ci}	(m)	13.08	11.61
y_{gi}	(m)	6.28	6.71	x_{gi}	(m)	13.97	12.13
e_{yi}	(m)	0.16	0.27	e_{xi}	(m)	0.89	0.52
d_{yi}	(m)	7.34	6.01	d_{xi}	(m)	15.85	17.09
β_{xi}	(-)	1.06	1.06	β_{yi}	(-)	1.12	1.08
μ_{xi}	(-)	0.80	0.89	μ_{yi}	(-)	0.87	0.83
ξ_{xi}	(-)	0.80	0.80	ξ_{yi}	(-)	0.80	0.80

ζ_{xi}	(-)	0.80	0.80	ζ_{yi}	(-)	0.80	0.80
$F_{SLV,xi}$	(kN)	2221	1852	$F_{SLV,yi}$	(kN)	1777	1100
$S_{e,SLV}$	(g)	0.187	0.155	$S_{e,SLV}$	(g)	0.149	0.092

In function of the values of $S_{e,SLV}$ it is possible to calculate by interpolating the values on table 4 the return period T_{SLV} and the spectral acceleration a_{SLV} for the damage and life safety limit states (see table 5). In table 6 are reported the calculated values of the Seismic Safety Index ($I_{S,SLV}$) and Acceleration Factor ($f_{a;SLV}$) for the castle before the retrofitting intervention.

~~Table 4. Values of the parameters a_g, F_0, T_c^* for different return periods T_R.~~

T_R [anni]	a_g [g]	F_o [-]	T_C^* [s]
30	0.049	2.349	0.276
50	0.065	2.350	0.296
72	0.078	2.350	0.315
101	0.094	2.354	0.325
140	0.110	2.359	0.335
201	0.132	2.360	0.345
475	0.194	2.399	0.366
975	0.263	2.423	0.386
2475	0.372	2.459	0.429

Table 4. Values of the parameters a_g, F_0, T^*_c for the different return period T_R.

T_R (year)	a_g (g)	F_0 (-)	T^*_c (s)
30	0.049	2.349	0.276
50	0.065	2.350	0.296
72	0.078	2.350	0.315
101	0.094	2.354	0.325
140	0.110	2.359	0.335
201	0.132	2.360	0.345
475	0.194	2.399	0.366
975	0.263	2.423	0.386
2475	0.372	2.459	0.429

Table 4. Seismic safety index and acceleration factors for Damage Limit State (SLD) and Life Safety Limit Sate (SLV).

	X Direction				Y Direction			
			Level 1	Level 2			Level 1	Level 2
SLD	$T_{SLV,x}$	(year)	52	38	$T_{SLV,y}$	(year)	35	<30
	$I_{SLV,x}$	(-)	0.70	0.50	$I_{SLV,y}$	(-)	0.46	<0.40
	$a_{SLV,x}$	(m/s^2)	0.65	0.54	$a_{SLV,y}$	(m/s^2)	0.52	<0.48
	$f_{a,SLV,x}$	(-)	0.83	0.69	$f_{a,SLV,y}$	(-)	0.66	<0.61
SLV	$T_{SLV,x}$	(year)	52	38	$T_{SLV,y}$	(year)	35	<30
	$I_{SLV,x}$	(-)	0.07	0.05	$I_{SLV,y}$	(-)	0.05	<0.04
	$a_{SLV,x}$	(m/s^2)	0.65	0.54	$a_{SLV,y}$	(m/s^2)	0.52	<0.48
	$f_{a,SLV,x}$	(-)	0.29	0.24	$f_{a,SLV,y}$	(-)	0.23	<0.21

4. PROPOSED INTERVENTION

In order to increase the shear resistance of the walls, reinforced mortar will be applied on each side of the walls. The type of mortar is R FIBER (structural M15) and steel net GFRP FIBRE NET with the followings characteristics

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FB MESH 66 x 66 T192 66 x 66 mm weight 1000 gr/mq
The mechanical characteristics of the mortar are listed below
Class: M15
$f_{c,int}$ = 16,21 MPa Average compression resistance
$f_{t,int}$ = 1,00 MPa Average tension resistance
E_m = 9900 MPa Average elastic modulus

Due to application of the reinforced mortar, the mechanical properties of the masonry will increase. New values, corrected by the confidence factor, are reported in Table 5.

Table 5. Mechanical properties of masonry after the proposed retrofitting works

Masonry Typology	f_m (N/mm^2)	τ_0 (N/mm^2)	E (N/mm^2)	G (N/mm^2)	W (kN/m^3)
Masonry of brick and mortar	4.91	1.16	3225	768	18

Other than reinforcing the walls, it will be necessary to reconstruct the slabs at the first and second floors as well as the roof with the same typology and material as in the original building.

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In order to assess the effectiveness of the proposed intervention, a new safety index and acceleration factor must be calculated incorporating the effects of to be constructed slabs and roof, and using the new values of the improved resistance of the masonry.

The index has been calculated for each floor and direction as described in the previous paragraph. The obtained results are reported in Tables 6 and 7.

Table 6. Shear resistance and elastic acceleration response spectrum values after the proposed retrofitting work.

X Direction				Y Direction			
		Level 1	Level 2			Level 1	Level 2
W_i	(kN)	69803	86440	W_i	(kN)	69803	86440
e*	(-)	0.96		e*	(-)	0.96	
q	(-)	2.50		q	(-)	2.50	
k_i	(-)	1	0.66	k_i	(-)	1	0.66
σ_{0i}	(N/cm^2)	18.11	12.26	σ_{0i}	(N/cm^2)	18.11	12.26
τ_{di}	(N/cm^2)	16.64	15.21	τ_{di}	(N/cm^2)	16.64	15.21
A_{xi}	(m^2)	48.88	42.85	A_{yi}	(m^2)	37.92	27.53
y_{ci}	(m)	6.44	6.99	x_{ci}	(m)	13.08	11.61
y_{gi}	(m)	6.28	6.71	x_{gi}	(m)	13.97	12.13
e_{yi}	(m)	0.16	0.27	e_{xi}	(m)	0.89	0.52
d_{yi}	(m)	7.34	6.01	d_{xi}	(m)	15.85	17.09
β_{xi}	(-)	1.06	1.06	β_{yi}	(-)	1.12	1.08
μ_{xi}	(-)	0.74	0.89	μ_{yi}	(-)	0.87	0.83
ξ_{xi}	(-)	1.00	1.00	ξ_{yi}	(-)	1.00	1.00
ζ_{xi}	(-)	1.00	1.00	ζ_{yi}	(-)	1.00	1.00
$F_{SLV,xi}$	(kN)	6151	5466	$F_{SLV,yi}$	(kN)	4922	3247
$S_{e,SLV}$	(g)	0.939	0.834	$S_{e,SLV}$	(g)	0.751	0.496

From the obtained results, it is possible to see that the weakest direction is the Y direction where the resisting area of the walls is smaller.

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Table 7. Seismic safety Index and Acceleration factor for Damage Limit State (SLD) and Life Safety Limit State (SLV) after the proposed retrofitting work.

	X Direction				X Direction			
			Level 1	Level 2			Level 1	Level 2
SLD	$T_{SLV,x}$	(year)	1768	1296	$T_{SLV,y}$	(year)	946	382
	$I_{SLV,x}$	(-)	23.57	17.29	$I_{SLV,y}$	(-)	12.62	5.09
	$a_{SLV,x}$	(m/s^2)	3.14	2.81	$a_{SLV,y}$	(m/s^2)	2.54	1.70
	$f_{a,SLV,x}$	(-)	4.00	3.57	$f_{a,SLV,y}$	(-)	3.23	2.16
SLV	$T_{SLV,x}$	(year)	1768	1296	$T_{SLV,y}$	(year)	946	382
	$I_{SLV,x}$	(-)	2.48	1.82	$I_{SLV,y}$	(-)	1.33	0.54
	$a_{SLV,x}$	(m/s^2)	3.14	2.81	$a_{SLV,y}$	(m/s^2)	2.54	1.70
	$f_{a,SLV,x}$	(-)	1.39	1.24	$f_{a,SLV,y}$	(-)	1.12	0.75

5. CONCLUSIONS

The obtained results after the retrofitting show that the weakest direction is the Y direction especially for the second floor. The first floor level has a higher strength due to the thickness of the walls (80-100 cm) at that level.

In particular, for the existing state Seismic Safety Index ($I_{SLV,y}$) value is smaller 0.40 for both damage and life safety limit states. With the proposed intervention it was possible to obtain values for Seismic Safety Index as

$$\mathbf{I_{SLV,y} = 5.09 \text{ for the SLD}}$$
$$\mathbf{I_{SLV,y} = 0.54 \text{ for the SLV}}$$

In terms of Acceleration Factor, $f_{a,SLV,}$ for the existing configuration a value of $f_{aSLV,y} < 0.61$ for the SLD and $f_{aSLV,y} < 0.21$ for the SLV are obtained. With the proposed intervention, it was possible to obtain values for for the acceleration factor as

$$\mathbf{f_{aSLV,y} = 2.16 \text{ for the SLD}}$$
$$\mathbf{f_{aSLV,y} = 0.75 \text{ for the SLV}}$$

From these values corresponding to the retrofitted case, it can be seen that the seismic resistance of the castle is increased according to the philosophy of the new building code.

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