SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

SEISMIC ISOLATION ATTEMPT FOR A VERY OLD UNREINFORCED MASONRY BUILDING IN BUCHAREST

Ion VLAD[1], Florin MACINIC[2], Stefan SISU[3] and Mihnea VLAD[4]

ABSTRACT

The "Ghika Tei Palace" is one of the most important and remarkable buildings in Bucharest, Romania. Due to its architectural and cultural history, as well as its age (being one of the oldest buildings in the city), it has been the object of an extensive technical assessment. A thorough examination of the building started with discovering and understanding its past behaviour spread over almost two centuries. The technical assessment of this sensitive building required proceeding with utmost caution. In consequence, a geotechnical research together with non-destructive technique methods were performed, in order to identify the foundation medium below it, its dynamic eigencharacteristics and the actual quality of the used materials (bricks, mortar, wood and, at a reduced scale, concrete, which was used for local strengthening). In addition to establishing accurately the geometry of the building components, the identification of the structural and material characteristics provided knowledge that did not only allow a better technical assessment of this historic building, but also helped determining how to intervene to secure it from future strong earthquakes. The studies and the structural analyses allowed making reliable statements regarding the load paths, stress states and the stability of the building. It was also established that the dead load does not represent a risk for its structural system. As its current technical state does not comply with the present legal requirements in force, a solution of intervention using the base isolation procedure was proposed. This solution was considered, as the condition of historical and architectural monument of the "Ghika Tei Palace" does not allow any intervention at the interior, or at the exterior masonry walls.

INTRODUCTION

Romania is frequently subjected to strong earthquakes. The most important seismic region affecting an important part of its territory (as well as parts of the territories of neighbouring countries) is the Vrancea seismogenic zone (VSZ), which releases in the average, per century, more than 95% of the total seismic energy.

Over the last two centuries, in Romania, seven earthquakes with seismic magnitudes higher than 7.0 have occurred: October 26, 1802 (M_W = 7.9), November 26, 1829 (M_W = 7.3), January 11, 1838 (M_W = 7.5), November 10, 1940 (M_W = 7.7), March 4, 1977 (M_W = 7.4), August 30, 1986 (M_W = 7.1) and May 30/31, 1990 (M_W = 7.0/6.4). So, Romania is situated in a seismically active region and has a history of devastating and deadly earthquakes.

The Bucharest area has experienced all the above mentioned seismic strong motions and the probability for a severe and damaging earthquake to occur is high. The consequences of such a

[1] Professor, Technical University of Civil Engineering, Bucharest, vlad@dial.kappa.ro
[2] Engineer, S.C, ALFA Proiect & Consulting S.R.L. Bucharest, macinicflorin@yahoo.com
[3] Engineer, Technical University of Civil Engineering (RNCEEV), Bucharest, sisu_stefan@yahoo.com
[4] Doctoral student, University of Bucharest, Faculty of Physics, Magurele, mihnel@yahoo.com

disaster will vary greatly, depending upon the circumstances following the quake, and no one can predict with any certainty what condition will exist immediately after the occurrence of a strong motion. Bucharest, the main administrative, economic and cultural center of the country, is one of the capitals with the highest seismic risk in the world. The city is particularly vulnerable to seismic hazard due to: the high density of inhabitants, especially within the residential building with blocks of flats, the old public utility fund, the out-of-date infrastructure, the numerous industrial parks that are undergoing a restructuring process, the inefficient organization of civil protection and a poor education of the population regarding the seismic risk. More than that, due to its economical problems, Romania does not allocate the necessary funds to secure its historical and architectural building stock.

The *Historical Monuments List* prepared by the *National Institute for Historical Monuments* contains more than 25 cultural heritage palaces in Bucharest. Among them, one of the most important historic landmark buildings is the "Ghika Tei Palace", which was completed between 1822-1830 (Fig.1).

Figure 1. Main façade of the "Ghika Tei Palace".

The seismic vulnerability of this building evolves primarily from its age. It was built in a period in which there were no standard design techniques. During its existence it supported all the major earthquakes that have occurred (1838, 1908, 1940, 1977, 1986, 1990, and 2005) and, after each earthquake, local repairs were made without having defined a general concept of strengthening. Initially it was built as summer residence for Grigore Ghika, one of the rulers of the Romanian Principalities. Afterwards it had several destinations, the most recent ones being that of school and starting with 1978 that of an exclusivist restaurant, and rented spaces for special activities. Its various destinations led to several specific transformations, including some structural interventions at a reduced scale.

The building resisted all the seismic events due to its regular in plane configuration with large masonry walls, but especially due to its fundamental eigenfrequency of vibration that was out of the band of dominant frequencies in which the energy of the strong ground motions was maximal. Currently the "Ghika Tei Palace" is in an alarming situation and, for this reason, at the end of 2013 the authors of the paper have performed the technical assessment of this old historic and monumental building. Due to the severe restrictions of the Romanian legislation in what concerns the historical monumental buildings an outside structural intervention is not permitted and an interior intervention is not possible, due to the walls and ceiling paintings and decorations (Fig. 2).

The intent of traditional seismic design and retrofitting of the existing buildings is to allow that specific components of a building behave during a strong earthquake in a nonlinear manner, so that some of the input energy is absorbed (Vlad and Vlad, 2008; 2009). Yielding is only permitted in those structural elements that will not disturb the stability of the structural system of the building. This type of approach to seismic design and retrofitting is only useful for conventional buildings, but it is not appropriate for historic and monumental buildings (Vlad, 2009).

The seismic retrofitting of the existing historic buildings raises many challenges to the structural engineer for improving their structural performance within ruthless architectural and legislative

constrains. The design strategy for the retrofit of a historic building should be done in such a way in order to avoid, as far as possible, the nonlinear behaviour of its structural system which could cause potential damage to the architectural and artistic integrity of façades, interior walls and ceilings.

Due to these facts, the only solution to reduce the seismic demand on the building is its seismic isolation, in order to secure it to the damaging effects of the future severe ground motions. At present the effectiveness and versatility of seismic isolation open the way to new fields of applications that include the seismic protection of masonry structures. Despite the successful application of this technique worldwide, there is still little interest in applying base isolation for the retrofit of historic buildings. This is mainly due to two factors: the first one being the lack of a special code for the strengthening of historic buildings, and the second one being the inherent resistance to innovative solutions of the elder engineering community.

Figure 2. Photos of walls and ceiling decorations within the "Ghika Tei Palace"

This paper summarizes both the technical assessment and the comprehensive approach proposed for implementing the seismic isolation technique for the seismic retrofitting of this historic and monumental building (Parducci, 2000). The possibility of its application depends primarily on the actual configuration of the building, the most important aspect being that it did not undergo structural changes or addition of irregular elements during its lifetime. On the other hand, the authors' attention was focused on the concept of "*seismic isolation*", having in mind the peculiar conditions of the Vrancea seismic motions in Romania (Vlad et al., 2008). When a building is built on an isolation system, it should have a fundamental eigenperiod longer than both its fixed base eigenperiod and the dominant periods of the ground motion. The first mode of the isolated building then involves deformations only in the isolation system, the structure above having the behaviour of a rigid solid. As heavy and massive unreinforced brick masonry walls are the bearing elements of the building in discussion its behaviour under earthquake excitation will be studied in detail. In order to do this, a spatial structural model of analysis using finite elements of shell type was conceived.

The application of a base isolation system requires the construction of a new structural floor at the ground level and a new structure of foundation. The experience gained up to now for the base isolation approach put to evidence high performances when it was applied to protect old masonry buildings, due to their large masonry structural walls stiffness that make base isolation very effective.

This paper was written having in mind the "International Charter for the Conservation and Restoration of Monuments and Sites - The Venice Charter" (1964) addressed to the problem of use of modern techniques in its article 10: "*Where traditional techniques prove inadequate, the consolidation of a monument can be achieved by the use of any modern technique for conservation and construction, the efficacy of which has been shown by scientific data and proved by experience*".

SHORT ARCHITECTURAL DESCRIPTION

The "Ghika Tei Palace" is an edifice built in the well-known Neo-Classical Italian architectural style, with some elements of French Renaissance, according to the Romanian Principalities modernization

trend from the early XIXth century. The overall architecture of the building is part of a symmetrical plan, with the appearance of a perfectly balanced volume designed in two registers: *the lower level*, smaller, with rectangular window openings (flanked by façade brick masonry fill ups with significant dimensions and adorned with horizontal grooves), and *the upper level*, higher, with rectangular window openings finished at the top with arcs centered in their middle and framed by pillars with Corinthian capitels (Fig.3).

Figure 3. Main façade details: monumental entrance (left) and bas-relief detail (right).

The building has an in-plane nearly regular configuration that can be inscribed in a rectangle with the long side equal to 34 m and the short side equal to approximately 20 m.

Its horizontal composition consists of two wings, one in extension of the other, joined through a central hall (developed on the height of the ground floor and of the first floor), with lounges arranged on the left and on the right of a monumental staircase access. In general, the disposal of the rooms is similar both at the ground floor and at the first floor.

The "Ghika Tei Palace" consists of a partial basement (the cellar), two levels (ground floor and the first floor) and an attic floor.

SHORT STRUCTURAL DESCRIPTION

The *structural system* of the building is composed of a vertical component consisting of massive "*unreinforced masonry bearing walls*", and a horizontal component, consisting of "*composite floor structures*" made of metal beams and masonry vaults, wood and reinforced concrete.

The thickness of the masonry walls in the partial basement is equal to 80 cm. The thickness of the exterior masonry walls that constitute the main (West) and rear (East) façades is equal to 80 cm. *At the ground floor*, the interior masonry walls, arranged on the longitudinal direction have a thickness of 50 cm, excepting those close to the South façade of the building, which have a thickness equal to 75 cm. The interior masonry walls disposed on the transversal direction are of 50 cm thickness (on each side of the main entrance) and of 70 cm on the North and South sides of the building. *At the first floor*, the interior masonry walls arranged on the longitudinal direction, as well as on the transversal direction, have a thickness equal to 50 cm.

A special presence in the vertical component of the superstructure of the building consists of eight iron cylindrical columns at the ground floor, as well as at the first floor. There is no vertical continuity between the iron columns of the ground floor and the ones at the first floor, each group having its distinct role. All the iron columns are covered with brick masonry, plastered and finished with stucco-marble at the first floor, and painted with washable paint at the ground floor (Fig.4).

The horizontal component of the structural system consists of a "*composite floor structure*" at the first floor, having different thickness values (110 cm, 60 cm and 30 cm), a ground wood floor structure having a thickness of 90 cm, and an attic wood floor structure of 50 cm. As a conclusion, one can state that the role of the floor structures is far from the normal role they should have had.

The *foundation structure of the building* consists of continuous foundations under brick masonry walls, made also of brick masonry in a wet environment.

Figure 4. View of the iron cylindrical columns (left: ground floor; right: 1st floor).

VIBRATION EIGENCHARACTERISTICS

Despite the fact that the "Ghika Tei Palace" has faced six strong seismic events during the last two centuries, one can state that this old building is in quite a good state of conservation. This statement is inconsistent with the technical legislation in force according to which the building is in an alarming situation. As the building is well maintained, no visible damage can be observed. Thus, the only objective way to get information on the building was to identify the parameters governing its dynamic behavior by performing dynamic testing. In many respects, the practice of vibration testing to such masonry buildings is more *art*, than *science*. The type of testing, its extent and the required quality of the results, follow the defined objectives: to obtain the modal eigenfrequencies, the modal eigenshapes and information on damping. This information helps the designer to calibrate structural models of analysis that are suitable for the given purpose – the technical assessment. To evaluate the dynamic behavior of the building an experimental study was performed by applying the *ambient testing method*.

Instrumental data acquisition was performed with specific Kinemetrics equipment of high performance and high sensitivity. 8 SS-1 Ranger seismometers were used in five configurations (Fig.5). Fig.6 and Fig.7 present time domains and amplitude Fourier spectra representations, both for longitudinal and transversal directions. By processing the experimental data the following eigenperiods of vibration were obtained: $T_{1,L}$=0.39 s; $T_{1,T}$=0.31 s; $T_{1,V}$=0.06...0.08 s; $T_{1,TORS}$=0.23 s.

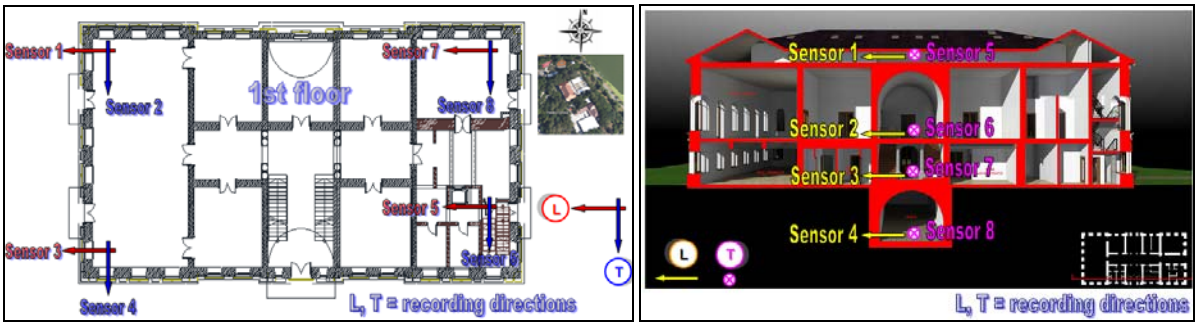


Figure 5. Location of sensors at the 1st floor and on a vertical arrangement.

Following an extensive program of acquisition, processing and interpreting the instrumental data, several conclusions were formulated:

- the recorded signals put to evidence "*a non-synchronous state*" between the motions recorded in various instrumented points; this fact showed that the 1st floor structure doesn't work together with the masonry bearing walls, not even in the condition of small amplitude vibrations;
- the examination of the fundamental eigenvalues derived from records showed that these pertain to a *narrow band of frequencies*, which made it possible to conclude that the entire

building shows a quite homogeneous performance in case of free vibration, along both horizontal directions;

- the existence of a large number of eigenfrequencies besides the fundamental eigenfrequencies (on both directions) proved that the building doesn't possess a *well-defined dynamic identity*;
- it can be stated that the "Ghika Tei Palace" shows *a high degree of vulnerability* to seismic actions with spectral content in the vicinity of the building's eigenfrequencies.

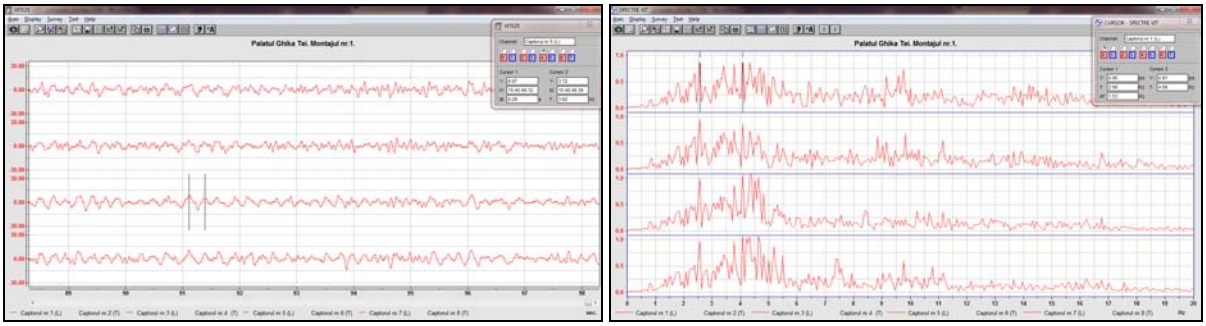


Figure 6. Ambient vibration testing; longitudinal direction; velocities (μm/s).
Time domain and corresponding Fourier amplitude spectra representations.

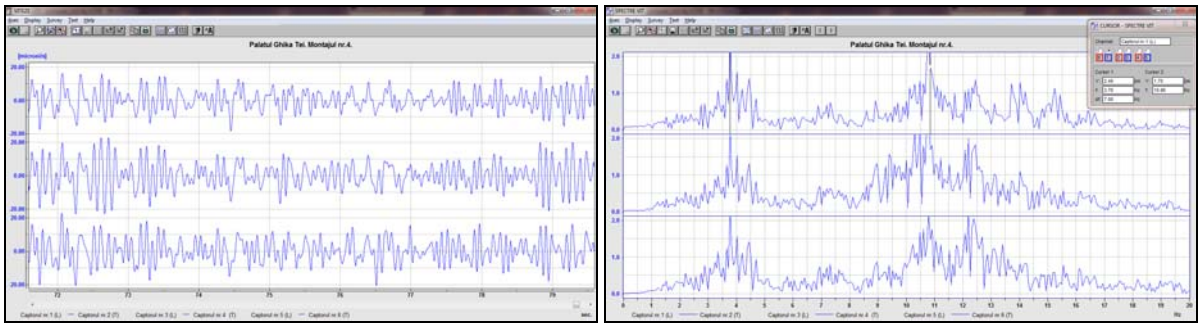


Figure 7. Ambient vibration testing; transversal direction; velocities (μm/s).
Time domain and corresponding Fourier amplitude spectra representations.

DESCRIPTION AND MODELLING OF THE BUILDING

In order to compare the "*fixed*" and "*base isolated*" building, structural models of analysis were conceived and analyzed using the ETABS structural analysis software (Fig.8).

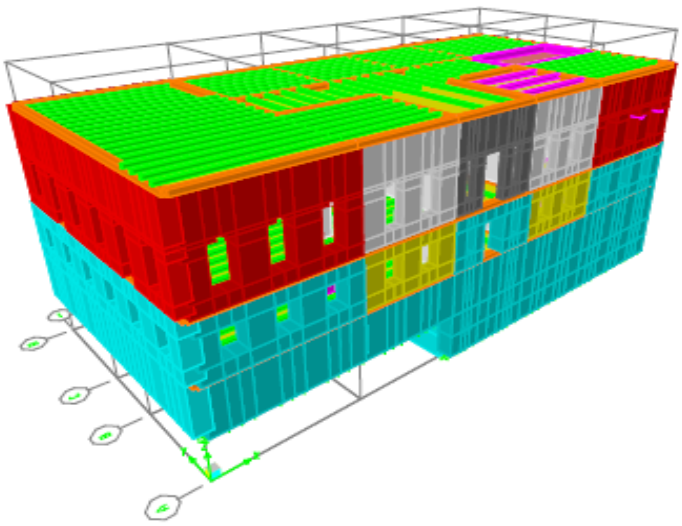


Figure 8. 3D structural model of analysis for the "*fixed base*".

The obtaining of the building eigencharacteristics by instrumental investigations helped to accurately calibrate the structural models of analysis for the "*fixed*" building. The building was modeled as a 3D structural system using for masonry walls, partial reinforced concrete and wood floors "*shell elements*" and for the wood beams "*beam elements*". The superstructure modal damping ratios are assumed to be constant for each mode, as 8% (the value that was instrumentally obtained was 7%).

In Fig.9 and in Fig.10 the "*active sections planes*", at the ground level on both longitudinal and transversal directions, are presented.

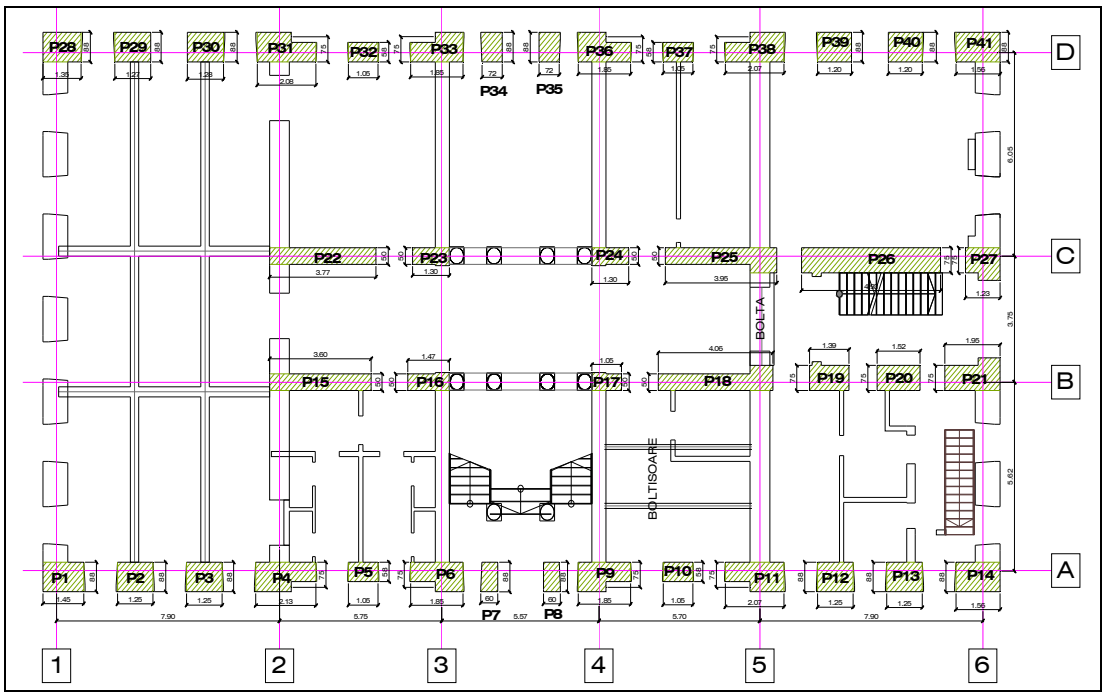


Figure 9. Active section plane on the longitudinal direction (ground floor).

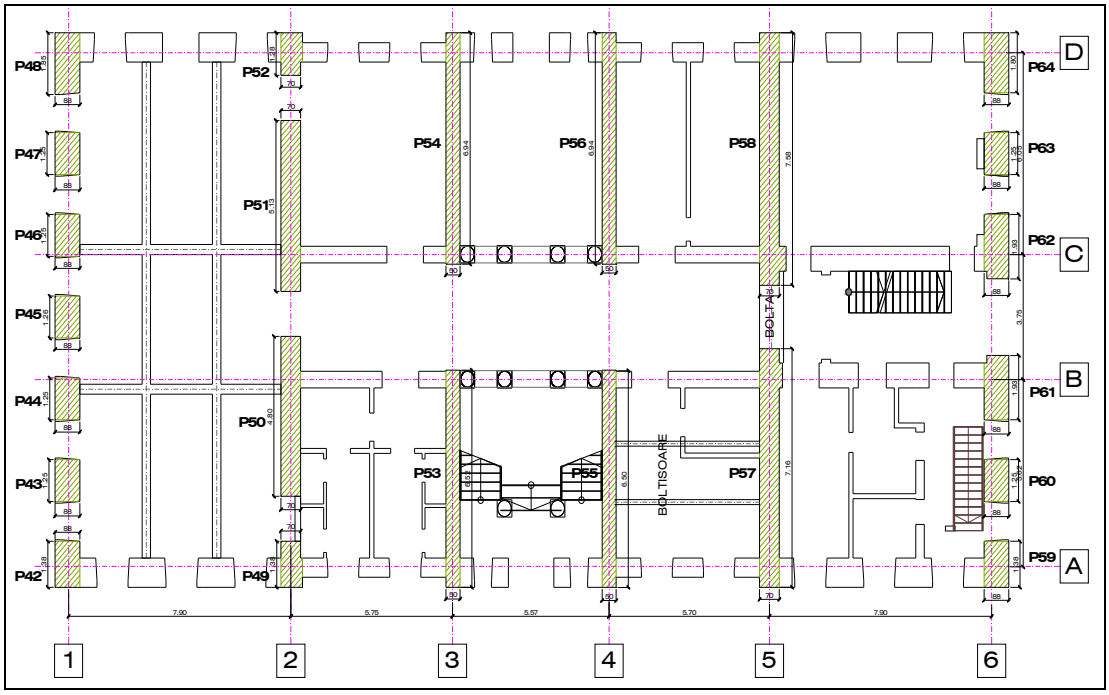


Figure 10. Active section plane on the transversal direction (ground floor).

In Table 1 and in Table 2 are summarized the obtained capacity values for the masonry bearing walls of the building, on both longitudinal and transversal directions.

Table 1. Masonry Bearing Walls Capacity Values (Longitudinal Direction)

LONGITUDINAL DIRECTION																					
												CRACKING		YELDING		ULTIMATE					
Section	Type	N	L	B	A	H	λ	σ_0	f_k	R_2	R_c	$\tau_{0,cap,F}$	$\tau_{0,asoc,F}$	$\tau_{0,cap,C}$	$\tau_{0,asoc,C}$	$\tau_{0,cap,U}$	$\tau_{0,asoc,U}$	$\tau_{0,cap}$	Q_{cap}	Q_{seism}	$R_{element}$
		[t]	[m]	[m]	[m^2]	[m]		[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[t]	[t]	
P1	M	14.77	1.45	0.88	1.28	9.35	6.45	1.16	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.79	6.86	0.26
P2	S	28.06	1.26	0.88	1.11	1.71	1.36	2.53	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	13.04	0.61
P3	S	27.86	1.26	0.88	1.11	1.71	1.36	2.51	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	12.94	0.62
P4	S	37.30	2.13	0.75	1.60	1.71	0.80	2.33	17.12	0.45	14	0.73	0.61	0.41	1.4	0.21	1.45	0.71	11.34	17.33	0.65
P5	S	18.63	1.05	0.58	0.61	1.71	1.63	3.06	17.12	0.45	14	0.83	0.38	0.51	0.8	0.26	0.86	0.64	3.90	8.66	0.45
P6	S	27.65	1.85	0.75	1.39	1.71	0.92	1.99	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	8.74	12.85	0.68
P7	M	8.22	0.59	0.88	0.52	9.35	15.85	1.58	17.12	0.45	14	0.61	0.07	0.27	0.2	0.12	0.21	0.21	1.09	3.82	0.29
P8	M	8.67	0.59	0.88	0.52	9.35	15.85	1.67	17.12	0.45	14	0.61	0.07	0.27	0.2	0.12	0.21	0.21	1.09	4.03	0.27
P9	S	27.91	1.85	0.75	1.39	1.71	0.92	2.01	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	8.74	12.97	0.67
P10	S	16.82	1.05	0.58	0.61	1.71	1.63	2.76	17.12	0.45	14	0.82	0.37	0.5	0.79	0.25	0.85	0.63	3.84	7.81	0.49
P11	S	33.85	2.07	0.75	1.55	1.71	0.83	2.18	17.12	0.45	14	0.73	0.61	0.41	1.4	0.21	1.45	0.71	11.02	15.73	0.70
P12	S	27.46	1.26	0.88	1.11	1.71	1.36	2.48	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	12.76	0.63
P13	S	27.27	1.26	0.88	1.11	1.71	1.36	2.46	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	12.67	0.63
P14	M	17.89	1.56	0.88	1.37	9.35	5.99	1.30	17.12	0.45	14	0.59	0.06	0.23	0.19	0.11	0.2	0.17	2.33	8.31	0.28
P15	M	42.77	3.6	0.5	1.80	9.35	2.60	2.38	17.12	0.45	14	0.75	0.19	0.4	0.46	0.21	0.49	0.28	5.04	19.87	0.25
P16	M	17.73	1.47	0.5	0.74	9.35	6.36	2.41	17.12	0.45	14	0.76	0.12	0.41	0.27	0.21	0.29	0.29	2.13	8.24	0.26
P17	M	10.25	1.05	0.5	0.53	9.35	8.90	1.95	17.12	0.45	14	0.7	0.1	0.34	0.24	0.17	0.25	0.25	1.31	4.76	0.28
P18	M	53.87	4.06	0.5	2.03	9.35	2.30	2.65	17.12	0.45	14	0.8	0.35	0.46	0.74	0.22	0.79	0.59	11.98	25.03	0.48
P19	S	18.69	1.39	0.75	1.04	1.35	0.97	1.79	17.12	0.45	14	0.69	0.48	0.32	1.1	0.16	1.05	0.58	6.05	8.68	0.70
P20	S	18.59	1.52	0.75	1.14	1.35	0.89	1.63	17.12	0.45	14	0.66	0.47	0.31	0.95	0.15	0.9	0.51	5.81	8.64	0.67
P21	M	19.46	1.95	0.75	1.46	9.35	4.79	1.33	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	2.05	9.04	0.23
P22	M	46.32	3.77	0.5	1.89	9.35	2.48	2.46	17.12	0.45	14	0.8	0.35	0.46	0.74	0.22	0.79	0.59	11.12	21.52	0.52
P23	M	15.52	1.3	0.5	0.65	9.35	7.19	2.39	17.12	0.45	14	0.76	0.12	0.41	0.27	0.21	0.29	0.29	1.89	7.21	0.26
P24	M	14.21	1.3	0.5	0.65	9.35	7.19	2.19	17.12	0.45	14	0.73	0.11	0.39	0.26	0.19	0.27	0.27	1.76	6.60	0.27
P25	M	53.33	3.95	0.5	1.98	9.35	2.37	2.70	17.12	0.45	14	0.8	0.35	0.46	0.74	0.22	0.79	0.59	11.65	24.78	0.47
P26	M	82.19	4.93	0.75	3.70	9.35	1.90	2.22	17.12	0.45	14	0.72	0.27	0.4	0.64	0.19	0.67	0.5	18.49	38.19	0.48
P27	M	11.70	1.23	0.75	0.92	9.35	7.60	1.27	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.29	5.44	0.24
P28	M	14.94	1.35	0.88	1.19	9.35	6.93	1.26	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.66	6.94	0.24
P29	M	24.86	1.27	0.88	1.12	9.35	7.36	2.22	17.12	0.45	14	0.73	0.11	0.37	0.25	0.19	0.27	0.27	3.02	11.55	0.26
P30	M	23.66	1.28	0.88	1.13	9.35	7.30	2.10	17.12	0.45	14	0.72	0.11	0.36	0.24	0.18	0.26	0.26	2.93	10.99	0.27
P31	S	37.79	2.08	0.75	1.56	1.71	0.82	2.42	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	11.23	17.56	0.64
P32	S	18.67	1.05	0.58	0.61	1.71	1.63	3.06	17.12	0.45	14	0.83	0.38	0.51	0.8	0.26	0.86	0.64	3.90	8.67	0.45
P33	S	28.88	1.85	0.75	1.39	1.71	0.92	2.08	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	8.74	13.42	0.65
P34	M	8.95	0.72	0.88	0.63	9.35	12.99	1.41	17.12	0.45	14	0.59	0.06	0.23	0.19	0.11	0.2	0.17	1.08	4.16	0.26
P35	M	9.16	0.72	0.88	0.63	9.35	12.99	1.45	17.12	0.45	14	0.59	0.06	0.23	0.19	0.11	0.2	0.17	1.08	4.26	0.25
P36	S	28.79	1.85	0.75	1.39	1.71	0.92	2.07	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	8.74	13.38	0.65
P37	S	16.36	1.05	0.58	0.61	1.71	1.63	2.69	17.12	0.45	14	0.82	0.37	0.5	0.79	0.25	0.85	0.63	3.84	7.60	0.50
P38	S	33.03	2.07	0.75	1.55	1.71	0.83	2.13	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	9.78	15.35	0.64
P39	M	24.27	1.2	0.88	1.06	9.35	7.79	2.30	17.12	0.45	14	0.75	0.11	0.4	0.26	0.2	0.28	0.28	2.96	11.28	0.26
P40	M	25.29	1.2	0.88	1.06	9.35	7.79	2.39	17.12	0.45	14	0.76	0.12	0.41	0.27	0.21	0.29	0.29	3.06	11.75	0.26
P41	M	16.97	1.56	0.88	1.37	9.35	5.99	1.24	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.92	7.88	0.24

Table 2. Masonry Bearing Walls Capacity Values (Transversal Direction)

TRANSVERSAL DIRECTION																					
												CRACKING		YELDING		ULTIMATE					
Section	Type	N	L	B	A	H	λ	σ_0	f_k	R_2	R_c	$\tau_{0,cap,F}$	$\tau_{0,asoc,F}$	$\tau_{0,cap,C}$	$\tau_{0,asoc,C}$	$\tau_{0,cap,U}$	$\tau_{0,asoc,U}$	$\tau_{0,cap}$	Q_{cap}	Q_{seism}	$R_{element}$
		[t]	[m]	[m]	[m^2]	[m]		[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[kg/cm^2]	[t]	[t]	
P42	M	11.02	1.38	0.88	1.21	9.35	6.78	0.91	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.70	5.12	0.33
P43	S	28.48	1.26	0.88	1.11	1.71	1.36	2.57	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	13.23	0.60
P44	S	30.74	1.26	0.88	1.11	1.71	1.36	2.77	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	14.28	0.59
P45	S	32.55	1.26	0.88	1.11	1.71	1.36	2.94	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	15.13	0.56
P46	S	30.70	1.26	0.88	1.11	1.71	1.36	2.77	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	14.26	0.59
P47	S	28.48	1.26	0.88	1.11	1.71	1.36	2.57	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	13.23	0.60
P48	M	18.30	1.85	0.88	1.63	9.35	5.05	1.12	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	11.72	8.50	1.00
P49	M	13.37	1.38	0.7	0.97	9.35	6.78	1.38	17.12	0.45	14	0.59	0.06	0.23	0.19	0.11	0.2	0.17	1.64	6.21	0.26
P50	M	84.80	4.8	0.7	3.36	9.35	1.95	2.52	17.12	0.45	14	0.8	0.35	0.46	0.74	0.22	0.79	0.59	19.82	39.40	0.50
P51	M	89.56	5.13	0.7	3.59	9.35	1.82	2.49	17.12	0.45	14	0.8	0.35	0.46	0.74	0.22	0.79	0.59	21.19	41.61	0.51
P52	M	12.54	1.28	0.7	0.90	9.35	7.30	1.40	17.12	0.45	14	0.59	0.06	0.23	0.19	0.11	0.2	0.17	1.52	5.83	0.26
P53	M	65.69	6.52	0.5	3.26	9.35	1.43	2.01	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	20.54	30.52	0.67
P54	M	70.29	6.94	0.5	3.47	9.35	1.35	2.03	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	21.86	32.66	0.67
P55	M	67.51	6.5	0.5	3.25	9.35	1.44	2.08	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	20.48	31.37	0.65
P56	M	68.62	6.94	0.5	3.47	9.35	1.35	1.98	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	21.86	31.88	0.69
P57	M	98.38	7.16	0.7	5.01	9.35	1.31	1.96	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	31.58	45.71	0.69
P58	M	108.58	7.58	0.7	5.31	9.35	1.23	2.05	17.12	0.45	14	0.7	0.5	0.34	1.21	0.17	1.27	0.63	33.43	50.45	0.66
P59	M	14.03	1.38	0.88	1.21	9.35	6.78	1.15	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	1.70	6.52	0.26
P60	S	31.19	1.26	0.88	1.11	1.71	1.36	2.81	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	14.49	0.58
P61	S	26.42	1.26	0.88	1.11	1.71	1.36	2.38	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	12.28	0.65
P62	S	30.42	1.26	0.88	1.11	1.71	1.36	2.74	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	14.13	0.60
P63	S	29.03	1.26	0.88	1.11	1.71	1.36	2.62	17.12	0.45	14	0.74	0.62	0.42	1.41	0.22	1.46	0.72	7.98	13.49	0.59
P64	S	29.74	1.26	0.88	1.11	1.71	1.36	2.68	17.12	0.45	14	0.8	0.72	0.48	1.55	0.24	0.7	0.76	8.43	13.82	0.61
P65	M	19.91	1.8	0.88	1.58	9.35	5.19	1.26	17.12	0.45	14	0.54	0.05	0.17	0.14	0.09	0.14	0.14	2.22	9.25	0.24

According to the Romanian legislation in force P100-3/2008, the safety evaluation to seismic actions and the inclusion of the building in a *seismic risk class* can be performed using three assessed *indicators*, R_1, R_2 and R_3, associated to the following categories of conditions:

- the degree of fulfillment of the seismic framing conditions (the degree of completion of structural framing conditions, of structural elements conditions and of constructive rules for structures which sustain the seismic action effects); this indicator is noted "R_1";
- the degree of structural damage, which indicates the rate of structural damage produced by either seismic actions or by other causes; this indicator is noted "R_2";
- the degree of structural assurance to seismic actions, which represents the ratio between the seismic capacity of a building and the structural demand for seismic actions, expressed in terms of strength (in the present case for the methodology of level 2), noted "R_3".

The values of the three indicators are associated to a certain *class of seismic risk* and help the technical expert to establish a final conclusion about the "*expected seismic response*", to include the buildings into a specific class of seismic risk and to establish the decision of intervention.

"R_1" which quantifies from the qualitative point of view the building framing, was assessed in respect to the following ten framing criteria: *the structural system quality, the masonry quality, the floor structures, the in-plane configuration, the configuration in elevation, the distances between walls, the elements which can produce lateral pressure, the type of foundation and the foundation medium, the possible interaction with adjacent buildings, and the nonstructural elements*. Thus, a value of the "R_1" indicator equal to 60 was obtained.

The "R_2" indicator defines the degree of seismic damage of the building. Taking into account the present destination of the building which requires permanent maintenance, a value equal to 90 for this indicator was assigned.

The "R_3" indicator was evaluated for each wall (vertical posts or piers) of the structural system of the building. As the building has wooden floor structures (only in a very limited area being made of reinforced concrete), one could not count on the effect of the interior forces redistribution. However, it was considered that these interior forces can be redistributed within the same "*line of resistance*", considering the "*plane frame effect*" created by vertical posts and piers. Therefore, a degree of assurance for each such frame we computed, finally resulting an overall degree of assurance $R_3 = 0.37$ on the longitudinal direction, and $R_3 = 0.38$ on the transversal direction.

Corroborating all the qualitative aspects with the values that were obtained for the three indicators, it was concluded that the building "Ghika Tei Palace" can be assigned to the *seismic risk class* R_sII, that is "*buildings which under the effect of the design earthquake can undergo major unacceptable structural damage (given that the loss of stability is unlikely)*". As the value of the "R_3" indicator resulted smaller than the one given in the seismic code P100-3/2008 ($R_3 < 0.62$), the *strengthening of the building is necessary*.

CHARACTERISTICS OF SEISMIC MOTIONS VERSUS SEISMIC ISOLATION

a) The first characteristic. The first characteristic refers to the fact that in Romania (including Bucharest) the sedimentary geological deposit has significant depths, in comparison with the depths in all the other seismic zones of the world, especially those in the United States and Japan. The seismic waves that are propagated through the lithosphere can generate, in a considered seismic zone, a phenomenon called "*first resonance*", as a result of the identity or the approaching of one of the frequencies of these waves with one of the eigenperiods of the sedimentary geological deposit. This frequency can be called "*dominant frequency*" of the sedimentary geological deposit. The big depths of sedimentary deposits in the seismic zones of Romania have as direct effect the fact that the corner periods of the response spectra can be five times greater than the corner values of earthquakes from the United States, Japan, and from almost the rest of the world (Vlad et al., 2008).

b) The second characteristic. The second important characteristic refers to the number of cycles of a seismic wave in the lithosphere, having the same period. This number of repetitions of identical cycles is also transmitted to the sedimentary geological deposit. The seismic waves that propagate through sedimentary deposits can produce the effect of a "*second resonance*" in a building placed at the surface, as the result of the identity, or of the approach, of one of the seismic waves' frequencies traveling through the sedimentary deposit to one of the eigenfrequencies of the building. Briefly, a component of the seismic motion with "*enough acceleration*" can produce a significant effect on the sedimentary deposit, which, in its turn, can generate a significant effect on buildings at the surface. One can state that the maximum acceleration of the seismic motion in "*free field*" at the Earth surface is the one corresponding to the sedimentary geological deposit, and its size depends on the earthquake seismic magnitude, epicentral distance, number of cycles with the same period etc. Due to the sedimentary deposit features, there are great differences between the spectral characteristics of the different seismic zones existing in Romania (Vlad et al., 2008). One can notice that the accelerations in the case of Vrancea earthquake, computed for Bucharest, have large values in the range of periods up to 2.5 s. Instead, the SA spectrum computed for the El Centro earthquake shows that the acceleration values strongly diminish for periods longer than 1.2 s (Fig.11).

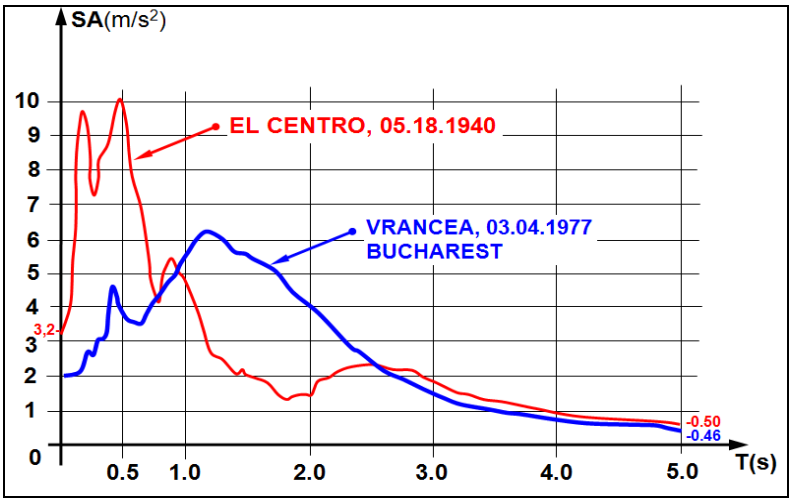


Figure 11. Comparison of the SA spectra for the March 4, 1977, Vrancea and the May 18, 1940, El Centro earthquakes

The goal of base isolation is to *reduce the seismic forces* that are exerted by an earthquake on a building structure. That's why the building which is going to be seismic isolated must be "*placed*" in a zone of the SA spectra of specific locations with convenient periods. At the same time, values of horizontal displacements that the isolators must undergo should be taken into consideration.

So, at the design of a "*seismic isolation*" for *El Centro type earthquakes*, the seismic forces can be reduced by placing the building in the period range of 1.2÷2 s. At the same time, for this period range, the horizontal displacements that the isolators must undergo are small, of about 10...12 cm.

In contrast with the above presented case, the design of a "*seismic isolation*" in Bucharest, for *Vrancea type earthquakes*, the seismic forces can only be reduced by placing the building in the period range over 2.5 s. The straight consequence of being obliged to place the building in the zone of very long periods consists in the fact that the isolators that are to be used must assure horizontal displacements of the order 40...45 cm.

BASE PRINCIPLES OF SEISMIC ISOLATION IN ROMANIA

a) The first principle. In the peculiar conditions of the seismic action in Romania it is required that for the seismic isolation of buildings very long fundamental eigenperiods of vibration should be achieved.

In the case of buildings *without seismic isolation*, the protection measures against earthquakes are based on the fundamental principle of the achievement of a high capacity of ductility. This principle was formulated by Kiyoshi Muto as follows: "*enough strength and high ductility*".

In the case of buildings *with seismic isolation*, a paraphrase of the Muto principle can be formulated as follows: "*very little strength, but very long fundamental period is necessary*". By applying the existing provisions in design codes, the risk of getting a "*seismic action*" that could lead to big strength demands can occur, even in the case of buildings with very long periods of vibration. That is why it is necessary that the response spectra for the design of buildings with seismic isolation to exist. Their use should clarify what minimum strength demands are necessary in the case of the design of buildings with long fundamental eigenperiod of vibration, required by the seismic isolation method.

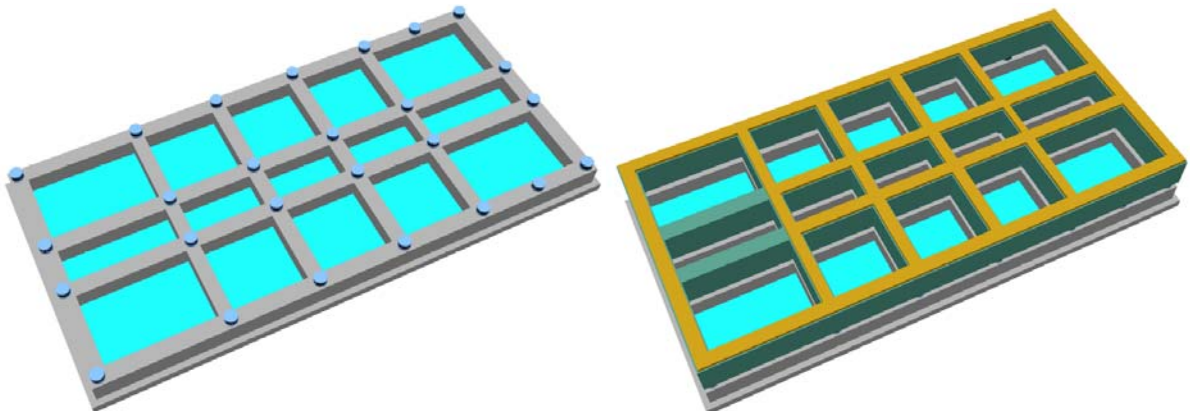
b) The second principle. The second principle refers to the seismic isolation devices. These must allow large displacements of about 40 cm, besides the safety reserves. If by structural analysis it results that the seismic isolation devices must allow horizontal displacements equal to 40 cm, for safety reasons they should allow displacements of even 60 cm. That is why it is necessary to clarify the use of seismic isolation devices in Romania. Towards this aspect, one must know if these devices can assure the same vertical level during the motion of the base of the building superstructure. If these devices have also displacements in a vertical plane (like in the case of the neoprene devices), then, as a result of the loads transmitted by the superstructure, they can have displacements with different values. For an efficient use, these seismic devices must have such configurations in order not to affect the behavior of the superstructure by supplementary and differentiated vertical displacements.

BASE ISOLATION ATTEMPT

Besides traditional technologies, such as tying the masonry walls with steel ties, the strengthening of the walls by injecting cementituous grouts and applying reinforced concrete coating – techniques developed decades ago – new methods based on advanced technologies have been proposed for upgrading the seismic behavior of old masonry buildings. Even if the requirements of preservation of old masonry buildings type limit the application of new methods of strengthening, modern technologies often require minimal interventions by providing at the same time substantial improvement in the seismic behavior. Unfortunately, the base isolation approach cannot always be applied to retrofit historic masonry buildings, even though the idea of separating the *upper structural system* from the *foundation structure* (thus reducing its response to seismic ground motions) is an excellent one. The possibility of its application mainly depends on the actual configuration of the existing historic buildings, especially when their construction has been progressively modified, by adding or removing irregular elements. Moreover, the retrofitting works implied by the application of the base isolation system may be very expensive. Seismic isolation, which is a passive control system, is one of the most promising alternatives to reduce the demands on structural components.

The case of the "Ghika Tei Palace" building is a special one. Due to the decorations and old mural paintings that grant the interior of this historic building a "*cultural heritage*" status, and as a result of the existing architectural details on the exterior walls, this building cannot be retrofitted by traditional technologies, the only possible remaining alternative being its "*seismic isolation*".

The isolation system consists of 26 LRB-S 800-175 produced by FIP Industriale S.p.A., that were placed as illustrated in Fig.12 The building seismic isolation is presented in Fig.13 and Fig.14.



a) b)

Figure 12. a) Layout of the lead rubber bearings. b) Spatial view of the isolated structure of foundation.

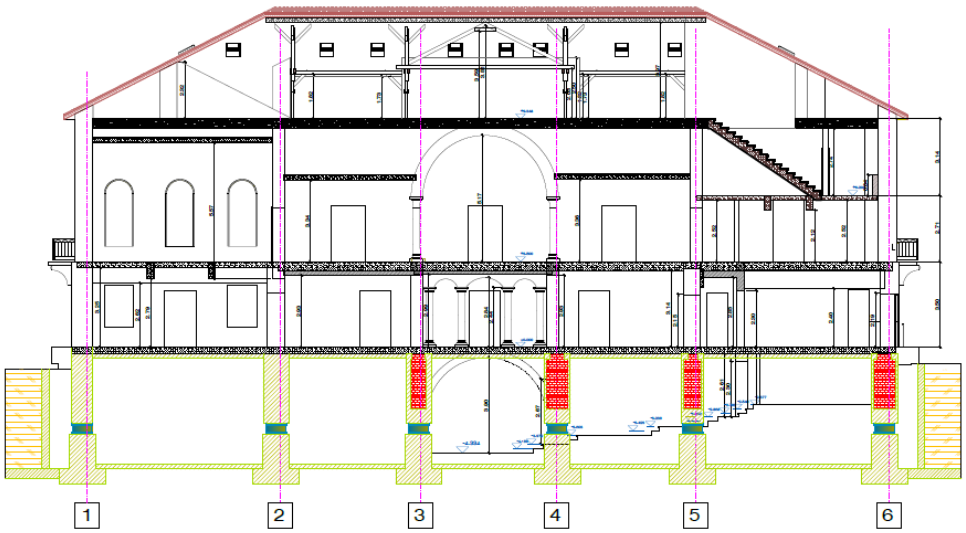


Figure 13. Longitudinal section showing the building isolation system.

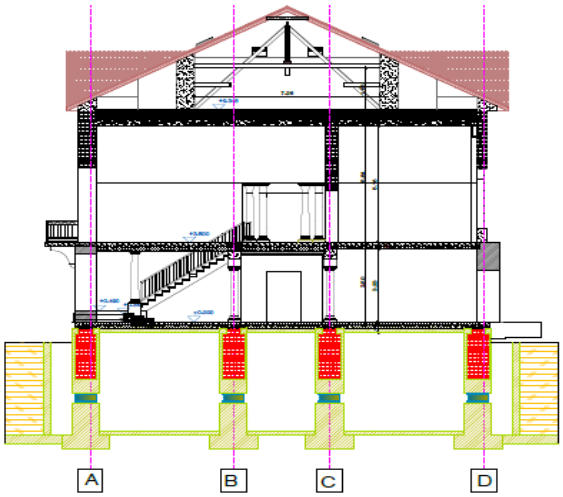


Figure 14. Transversal section showing the building isolation system.

The hysteretic behavior of a LRB was considered linear by means of the effective horizontal stiffness $K_e = 1.71$ kN/mm and an equivalent viscous damping coefficient $\xi_e = 27\%$.

A simplified structural analysis was performed using for each isolator a vertical stiffness K_v = 1571 kN/mm. The stiffness of the isolators has been calibrated to be proportional to the carried load of the superstructure, so that no torsional effect occurs at the isolation level. However, the stiffness of the peripheral isolators is greater than required, in order to uncouple the translational and rotational eigenmodes of vibration, and adjustments have been made because of the use of a single type of isolator. The structural analysis followed the requirements of the new Romanian code P100-1/2013 (a safety factor equal to 1.2), the seismic action being considered through the response spectrum corresponding to the computed damping, on the transversal, longitudinal, at 45°, and N-S directions.

CONCLUSIONS

The technical assessment and an attempt of seismic base isolation of a very old historic and heritage building was presented in the paper. The theoretical study that was performed led to a fundamental eigenperiod of vibration equal to 2.42 s and a lateral displacement equal to 33.3 cm (smaller than the maximum displacement of the used isolators, which is 40 cm). The checking of the values of the maximum vertical load, at load combination including the seismic action, led to inferior values (1600…1800 kN) in comparison with the one permitted by the considered isolator (2810 kN). It is expected that the results of the above studies will offer valuable contributions to improve the design concepts of seismic isolation for masonry buildings and will incorporate them in a usable form in a new technical legislation.

REFERENCES

Parducci A (2000) "Seismic Protection of Italy's Historic Heritage Using the Seismic Isolation Approach", *Proceedings of the 12th World Conference on Earthquake Engineering*, New Zeeland

Vlad I (2009) "The Improving of the Seismic Performance of Existing Old Public Unreinforced Masonry Buildings", *Proceedings of the 2009 ATC&SEI Conference*, San Francisco, California, December 9-11

Vlad I, Sandi H, Sannino U, Martelli A (editors), (2008) Proceedings of the International Symposium "Modern Systems for Mitigation of the Seismic Action", AGIR Publishing House, Bucharest

Vlad I and Sannino U (2008) "The Concept of Base Isolation for the Seismic Protection of Buildings Having in Mind the Peculiarities of Seismic Action in Romania", in *Proceedings of the International Symposium "Modern Systems for Mitigation of the Seismic Action"*, Bucharest, 31 October – 1 November

Vlad I and Vlad M (2008) "Behavior of Dwellings during Strong Earthquakes in Romania", *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing

Vlad I and Vlad M.N (2009) "Technical Assessment and Strengthening of an Architectural and Historical Monument Building in Romania. A Case Study.", in *International Conference on Protection of Historical Buildings (Prohitech),* Rome, 21–24 June