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Improving the Structural Performance of Heritage Buildings. A Comprehensive Romanian Experience.

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IMPROVING THE STRUCTURAL PERFORMANCE OF HERITAGE BUILDINGS. A COMPREHENSIVE ROMANIAN EXPERIENCE.

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ABSTRACT

The paper is devoted to historical masonry buildings' protection against the destructive influence of earthquakes. Experimental and analytical investigations were performed to verify an original methodology that was developed for improving the structural performance of such a building. The seismic retrofitting of the cultural heritage requires compliance with the severe restrictions of the Romanian legislation related to the preservation of the original artistic and structural features. The intervention on the building started with the understanding of the original idea that was in the mind of the first designer. To accomplish this desideratum, two studies have been performed: a historical study and a geotechnical one. These studies have been followed by a technical assessment and a proposal of intervention. As the building has been able to carry severe loads during its lifetime, the possibility of preserving the original idea of its configuration was taken into account. In the paper, the main stages during the technical assessment and the strengthening project are presented. Within the technical assessment, a theory of damage and failure of unreinforced masonry walls was applied. The strengthening solution has been chosen so that the character of historical and architectural monument should not be affected.

INTRODUCTION

In the second part of the XIXth century, as well as the beginning of the XXth century, in the villages and towns of Romania, small height masonry buildings with one or two levels, realized in traditional system were predominant. One can state that, as in most countries of the world, buildings with structural systems – masonry walls type, were the most frequent ones used in the past. For many centuries masonry buildings have been "*designed*" by using some practical rules derived from well defined ratios among the dimensions of the main structural elements, based on experience acquired over the years. Even after the evolution of most specific theoretical background in the field of the theory of structures (the end of the XIX century and the beginning of the XX century), the design of masonry buildings has been mostly done through the same old established procedures. The beginning of the XX century marked the introduction of new materials in the structural systems of masonry buildings, such as reinforced concrete and steel. In the above mentioned period, hundreds of thousands of rural residential and hundreds of public buildings with solid bricks structural walls have been built in Romania. From the second class of existing masonry type buildings, the most representative are those for: public services, military, education, health and culture (theatres, cinema, and museums).

In the period between 1880 and 1920 a series of low-rise (as a rule, not more than three-stories) heritage buildings were built in Romania. At such buildings with wall thickness between 28÷70 cm (1 brick ÷ 2½ bricks), having storey heights of 4.5…6 m, the wider spans were covered by brick vaultlets supported by steel beams. This type of floor was widely used over basements towards the end of the XIXth century. Rolled steel was increasingly used for lintels, balconies and bow-windows. The introduction of reinforced concrete during the first part of the XXth century has gradually replaced floors and lintels made of wood or steel by reinforced concrete members.

The building that is the subject of this paper was "designed" by an architect in 1911, and the project consisted only of a few architectural drawings. The founding stone of the building was set in 1911 and the building was built in several stages (because of the First World War), being completed between 1919 and 1921. Despite the fact that it has faced all the strong seismic events that have occurred during the last century in Romania (1940, 1977, 1986 and 1990), one can state that this building is in quite a good state of conservation. During its life the building has had two destinations: public building, housing the offices of the Town Hall, and spaces used as restaurants and cafeterias (1921-1976), and museum building, where the "National History and Archeology Museum" is located at present.

Many centuries of history are generously represented in this museum, ranged as a culture establishment of national importance due to the rich patrimonial collections from the Paleolithic Age until the present day.

As a matter of fact, this heritage together with the building should not be lost, and even if the theoretical background was, at that time missing, it is not allowed to modify old buildings just following the results obtained by modern calculation techniques. As the "*National History and Archeology Museum*" is a major attraction for tourism in the region, the Constanța County Council, which is the owner of the building, has decided its rehabilitation.

The present paper synthesizes the information contained in the following three papers, works to which I was the main author:

- "*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, 2009
- "*The Improving of the Seismic Performance of Existing Old Public Unreinforced Masonry Buildings*", Proceedings of the 2009 ATC & SEI Conference, 2009
- "*Foundation Structure Design for an Old Historical Building*", Proceedings of the 7th International Conference on Case Histories in Geotechnical Engineering, paper 2.29, Chicago, 2013

SOME INFORMATION ON STRONG EARTHQUAKES IN ROMANIA

During the last 70 years, Romania was struck by two destructive intermediate-depth earthquakes which occurred in the Vrancea region on November 10, 1940 ($M_{G\text{-}R}$ =7.4) and March 4, 1977 ($M_{G\text{-}R}$ =7.2).

These two were followed by other three strong ground motions, with the same focus, on August 30, 1986 ($M_{G\text{-}R}$ =7.0), May 30, 1990 ($M_{G\text{-}R}$ =6.7) and May 31, 1990 ($M_{G\text{-}R}$ =6.1).

March 4, 1977 Vrancea earthquake. The first strong motion recorded in Romania was the triaxial accelerogram obtained on a 1967 SMAC-B type strong motion accelerograph during the March 4, 1977 Vrancea event, in the soil condition of Bucharest. The peak ground acceleration values in the N-S, E-W and V directions were 0.20g (*PGA*=194.9 cm/s^2), 0.16g and 0.10g, respectively. A glance at the record shows that the long period components were present, aspect that surprised the engineering community of Romania, although engineers were acquainted with the first code proposal written by engineers Emilian Ţiţaru and Alexandru Cişmigiu at the 2WCEE (Japan). So, one can consider as birth date of the instrumental earthquake engineering in Romania the date of March 4, 1977. It is interesting to note that the shape of the spectral accelerations was very different of that generally assumed in the code in force. It must be mentioned that the elastic spectra shape had been imported from the Soviet code SN-8-57, characterized by a maximum dynamic amplification factor β_0 = 3.0 and a corner period of response spectra T_C = 0.3s, which, at its turn, corresponded to the 1940 El Centro earthquake spectra. In order to compare the acceleration response spectrum of the March 4, 1977 earthquake N-S component with the acceleration response spectrum of the N-S component of the 1940 El Centro earthquake, the latter was normalized to the same peak magnitude and plotted on the same diagram. The shapes of the spectral accelerations of these two earthquakes are very much different from each other. *The highest values of periods occurred in the range of 1.0...1.6 s for the N-S component and of 0.7...1.2 s for the E-W component.* Taking into account the above-mentioned values of the observed periods, it was to be expected that the damage should occur especially for the flexible buildings, having fundamental eigenperiods of vibration of about 1.0 s or more.

August 30/31, 1986 Vrancea earthquake. On 30 to 31 August 1986, Romania was shaken by another earthquake originating in the Vrancea seismogenic zone. This earthquake affected with high intensities extensive areas. The maximum acceleration recorded during this seismic event was close to 0.3g (in fact, the highest *PGA* value was recorded in Focşani, a town located in the nearest vicinity of the instrumental epicenter). The *PGA* values in Bucharest ranged between 0.06 g and 0.16 g (for the N-S component) and between 0.04g and 0.11g (for the E-W component), with periods ranging between 0.7 and 1.1 seconds. There were considerable differences in the spectral contents of the motion at different sites. The magnitude of this earthquake was M_W =7.3 (M_S ≅6.8, $M_{G\text{-}R}$ =6.9, m_b =6.5 to 6.6). The 1986 INCERC record at the same location as in 1977 had *PGA* values of 0.10g (E-W component) and 0.09g (N-S component), with periods of about 1.1 s. This supports the idea that intermediate depth earthquakes tend to produce motions characterized by longer periods when their focal distances increase.

May 30 and 31, 1990 Vrancea earthquakes. During the May 1990 earthquakes at least 29 seismic instruments were triggered in various towns, especially in the East and South of the Carpathians, and 9 seismic instruments recorded the motion in different locations in Bucharest. Firstly, it must be mentioned that 5 stations recorded *PGA* values larger than 0.20g in a wide area (maximum value in Câmpina equal to 0.26g). A variety of *PGA* values between 0.07g and 0.14g were reported during the main shock in Bucharest. Several new lessons seemed to emerge with the first information obtained from the 1990 accelerograms in Bucharest. Many records of the main shock on the E-W direction were stronger than on the N-S components (opposite to the previous two seismic events). The second important remark was that the observed periods were, this time, much shorter.

SHORT SEISMIC CHARACTERIZATION OF THE AREA WHERE THE BUILDING IS LOCATED

The paper is devoted to the building that houses the "*National History and Archeology Museum*", located in Constanța, a seaside city in Romania.

According to the present Romanian seismic code P100-1/2006, a code similar to the EUROCODE 8, the seismic characteristics of the Constanța zone are:

- the design peak ground acceleration value for earthquakes having a reference return period of 100 years is $a_g = 0.16g$ (Fig. 1);

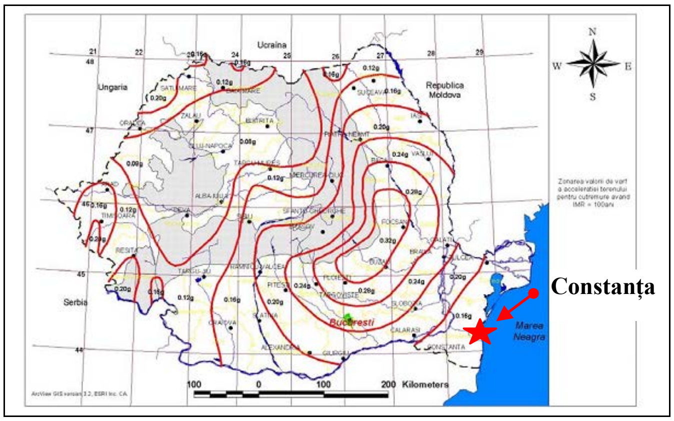


Fig. 1. Design peak ground acceleration values for a reference return period of 100 years (P100-1/2006).

- the corner period of the absolute acceleration response spectrum, for structural systems with behavior in the elastic range is $T_C = 0.7$ s, while $T_B = 0.07$ s and $T_D = 3$ s (Fig. 2);
- the dynamic amplification factor for response spectra periods ranged between T = 0.07 s and T = 0.7 s is $\beta_0 = 2.75$ (Fig. 3);

Fig. 2. Romania territory zoning in terms of the control period T_C of the response spectrum (P100-1/2006).

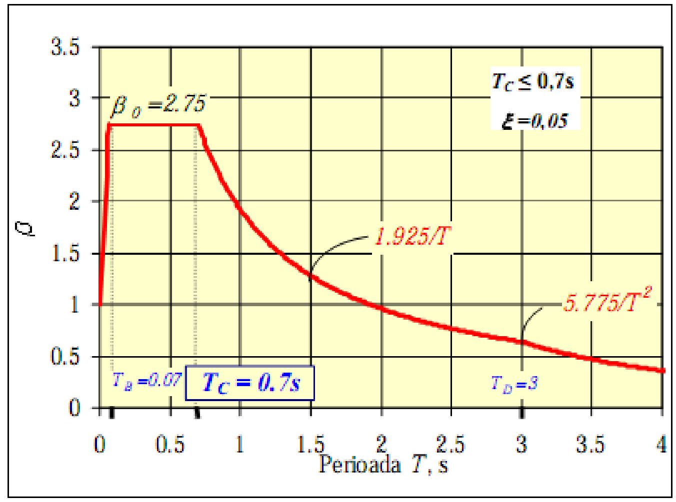


Fig. 3. Normalized elastic acceleration response spectra for horizontal components of seismic motions: $T_C = 0.7$ s (P100-1/2006).

It is considered necessary to present some aspects related to earthquakes that have affected the municipality of Constanța and remained in the media collection since 1916. For Constanța, a particular importance is given by the so-called "*Pontic earthquakes*." These earthquakes have their focus along a line very close to the Black Sea coast in the region of Constanța - Mangalia - Kavarna - Balchik.

On March 31, 1901 a catastrophic earthquake occurred, its macroseismic epicenter being located in the region "Shabla - Kavarna" in Bulgaria. The seismic magnitude was assessed as being equal to $M_{G\text{-}R} = 7.2$ and the earthquake affected an area of 500,000 km^2, of which 250,000 km^2 land. The severely affected region (42,000 km^2) included the city of Constanța where buildings were damaged, the *"culminations"* of the seismic motion generally having NW-SE direction. Comparing these *"culminations"* to those highlighted by the earthquake of November 10, 1940 one can find some obvious overlaps. The Romanian geologist Ion Atanasiu stated in his works that the Pontic seismic motions, after being triggered, stop after about 3-4 years, and that their macroseismic intensity can reach even degree X. It was also noted that this earthquake has generated a *"tsunami"* that hit the south of the current city of Mangalia, a city located in the immediate vicinity of Constanța.

On June 14, 1913 in Bulgaria, at Veliko Tarnovo, a strong earthquake occurred and had in the epicentral zone a macroseismic intensity I_0 = IX and a seismic magnitude $M_{G\text{-}R} = 6.8$. This seismic motion was felt throughout Dobrudja area, especially on the *"culmination Cernavoda-Constanța"* (I_0 = VII), that was well individualized also during the devastating earthquake of November 10, 1940.

Analyzing the distribution of strong ($M_{G\text{-}R} \geq 4.0$) and weak earthquakes ($M_{G\text{-}R} < 4.0$), it was reached to the identification, within Romania territory, of 10 seismic regions, among which one is Dobrudja. It is not known which seismic events *"inspired"* a reporter writing in 1916 about the danger of earthquakes, but during 1902-1916 period in Romania have occurred 13 earthquakes, with magnitudes ranging between 5.0 and 6.5. The first 12 had the seismic source located in the Vrancea seismic zone, and the last one, the most powerful dating 1916, was located in the seismogenic region Campulung.

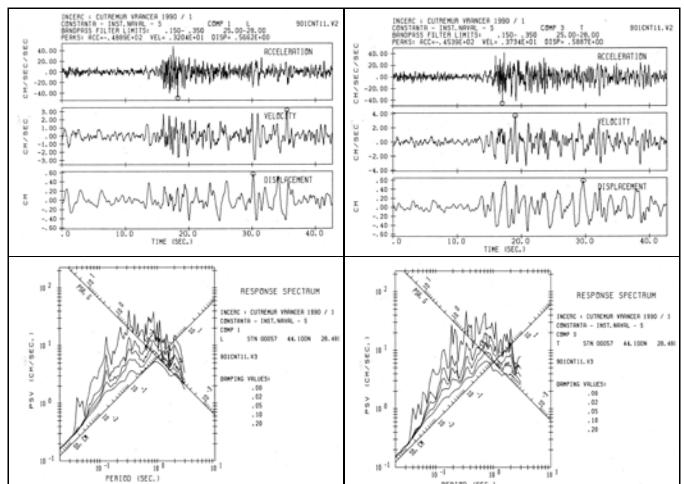
On November 13, 1981, in the Dobrudja region occurred an earthquake with a macroseismic intensity I_0 = VI÷VII and seismic magnitude $M_{G\text{-}R} = 5.2$.

In any seismic characterization of the Dobrudja area should be kept in mind that there may take place normal earthquakes (H_F < 60 km), but not forgetting that the Vrancea region is at the intersection of three tectonic units: the Carpathian-Alpine unit, the Podolia platform and the Dobrudjea unit (Radu and Polonic, 1982).

By scoring the epicenters of the Vrancea earthquakes on a tectonic map of the region, two "*seismic lines*" that define the limits of the movable Dobrudjan block immersed under the Carpathian Mountains are revealed. The mobility of the Dobrudja block is considered to be one of the main causes of the high seismicity of the Vrancea region (Radu, 1973-1974). Considered together, the other earthquakes occurring in Northern Dobrudja are associated with local sources of low energy (M ≤ 5), initiated within the Earth's crust and whose isoseismal lines are elongated towards NW and only sometimes to NE.

There is an accelerographic record in the Constanța city of the earthquake which occurred on May 30, 1990 (INCERC, "Naval Institute" station, Fig. 4). The examination of the accelerographic record and of the response spectra allows making the following observations:

1. The ground motion was more severe (peak ground horizontal accelerations of order 0.5 m/s^2) than it would have been expected for an earthquake of magnitude 6.7, as the one dated May 30, 1990. This coincides with the instrumental observations from Cernavoda - City Hall station (a town pertaining to the same Dobrudja area), where the peak ground accelerations were of the order of 1.0 m/s^2, far surpassing those recorded during an earthquake (August 30/31, 1986) of higher magnitude (7.0), that presented a more common directivity, NE-SW, a characteristic feature for Vrancea earthquakes. A statistical study of the attenuation phenomenon on all available records showed in fact a less common directivity for the 30 and 31 May 1990 earthquakes, which is reflected in an abnormal sequence of different intensities produced in Cernavoda during the three mentioned earthquakes.
2. The examination of the response spectra on the two horizontal directions highlights main spectral peaks at periods of the order of 0.15 ... 0.4 s. The relatively high spectral ordinates for periods in the range 0.5 … 1.0 s should not be neglected, as they can lead to major spectral peaks for an earthquake of high seismic magnitude (over 7.0). Given the shape of spectra for the horizontal motion, a design spectrum for the given location should have a plateau of maximum values at least until the period of 1 s.
3. Seismic hazard studies performed using advanced computational techniques have shown that, for various return periods, the seismic intensities for Constanța city are 1.0...1.5 degrees lower than for Bucharest. For instance, the return periods for Constanța are 30 years for the seismic intensity degree 6, of the order of 100 years for the seismic intensity degree 7 and of the order of 500 years for the seismic intensity degree 8. It is understood the fact that, depending on the site local conditions, clarifications or corrections of these values will be considered.
4. It is expected that a building like the one in which the "Museum of National History and Archaeology" operates, withstands without problems to a seismic intensity degree 6, to be damaged to a seismic intensity degree 7, and to be put in danger of collapse to a seismic intensity degree 8. The comparison of seismic hazard data allowed a certain estimation of the seismic risk associated to this building.



Station	Code Earthq	Code Station	Code Axis	pga	pgv	pgd	epa	Max Saa/2.5	epv	epd	Tc	Td	Is	Duration
Constanta	901	CNT1	N148W	0.4889	0.0320	0.0057	0.4633	0.7240	0.0398	0.0088	0.54	1.39	5.60	42.675
Constanta	901	CNT1	N58W	0.4539	0.0373	0.0059	0.4168	0.6681	0.0456	0.0101	0.69	1.39	5.61	42.675

Fig. 4. May 30, 1990 earthquake recordings. Horizontal accelerations and corresponding response spectra (INCERC, 1990).

ELASTIC BASE SHEAR FORCE

According to the Romanian P100-1/2006 code, the base shear force for the behavior of the building in the elastic range ($Q_{B,\text{elastic code}}$), will be established in the followings:

- the value of the acceleration spectrum for the response of a structural system in the elastic range to horizontal components of the seismic acceleration, in the interval of the above mentioned periods, is:

$$S_e(T) = a_g \times \beta_0 = 0.16 \times 2.75 = 0.44g \qquad (1)$$

- it should be mentioned that for buildings with structural systems of unreinforced masonry walls, the value of the response spectrum $S_e(T)$ is computed with the formula:

$$S_e(T)_{\xi=8\%} = S_e(T)_{\xi=5\%} \cdot \eta\,;\ \eta = \sqrt{10/(5+\xi)} \geq 0.55 \qquad (2)$$

where η is a correction factor that considers the influence of viscous damping; for this building, the spectrum $S_e(T)$ value has been decreased by multiplying it with a correction factor $\eta = 0.88$, that corresponds to a critical damping factor of 8%, which is characteristic to masonry walls buildings:

$$S_e(T)_{\xi=8\%} = 0.88 S_e(T)_{\xi=5\%} = 0.3872g \qquad (3)$$

- according to P100-1/2006 (as indicated in paragraph 4.4.5 and Table 4.2), the building is a cultural institution (museum), whose resistance to seismic actions is important considering the consequences associated with critical damage or collapse; it pertains to the class of importance II for which an importance factor $\gamma_I = 1.2$ is assigned; the value of the elastic response spectrum thus results:

$$S_e(T) = 1.2 S_e(T)_{\xi=0.8\%} = 1.2 \cdot 0.3872g = 0.46g \qquad (4)$$

The coefficient of the base shear force for the structural system response of the building in its elastic range, for the period interval between 0.07 s and 0.7 s (where the fundamental period of vibration of the building is found, $T_{n,1}=0.4\ s$) is:

$$c_{B,CODE,elastic} = \frac{Q_{B,CODE,elastic}}{G} = \frac{mS_e(T)}{mg} = 0.46 \qquad (5)$$

Finally, the elastic base shear force has resulted:

$$Q_{B,CODE,elastic} = c_{B,CODE,elastic} \cdot G = 58000\ kN \qquad (6)$$

where G = 126000 kN (weight of the superstructure).

THE TECHNICAL ASSESSMENT OF THE BUILDING

Content of the technical assessment

According to the Romanian legislation in force, the technical assessment had 16 chapters, as follows: (1) Reason and goal of the technical assessment; (2) Methods of investigation; (3) Comments regarding the condition of historical monument of the building and on its location in a historical protection area; (4) General data on the building; (5) Structural description of the building; (6) Geotechnical information on the foundation medium; (7) Description of the in-time modifications of the building; (8) Detailed qualitative assessment; (9) Ambient vibration instrumental investigations; (10) Materials non-destructive testing; (11) Advanced methods of investigation in order to assess the structural vulnerabilities of the building to seismic action; (12) Correlation of the obtained results and conclusions; (13) Establishing the seismic risk class of the building; (14) Proposal of intervention and remedial measures; (15) Substantiating the decision for the necessity of structural intervention; (16) Final conclusions and cost estimate for the proposed works.

The technical assessment contained also: (1) A historical study; (2) A geotechnical study; (3) Mapping of the existing cracks and damage; (4) Architectural and structural plans and details.

Urbanistic and heritage value of the building

The building in which the "*National History and Archeology Museum*" holds its activity comprises an urban and architectural dominant, not only for what the inhabitants of Constanța city know as the "*Ovidius Square*", but for the whole zone, which is in fact the historic center of the city.

For the entire old construction assembly in Constanța, this building represents the most important exponent of the Neo-Romanian architecture realized at the beginning of XX century. It is a massive monumental construction, which dominates the entire square, with expressive façades, conceived by architect Victor Stefănescu, one of the students of the architecture school founded by the great Romanian architect Ion Mincu (Fig. 5).

Fig. 5. General view of the main façade of the building.

Taking into account its architectural value, the building was declared "*architectural monument*" in 1979 and at present pertains to in the "List of Historical Monuments 2004", as a "*building of national interest*".

From the architectural point of view, the construction of the building was made under the influence of the reevaluation current of the Romanian traditional architecture that appeared at the end of the XIX century and the beginning of the XX century, known as “*Neo-Romanian*” style.

The essential characteristic of this building is given by the proportion and the unity of its volume, by the way of solving the façades, by the ratio between the “*compact*” zones and the “*void*” ones, but especially by the elements of adornment specific to the epoch when it was built. The dominant architectural element consists of the main façade, marked by a slight withdrawal of the entrance area in regard with the façade’s plane, but also by a vertical detachment of the central volume, ended with an octagonal tower with a clock, of open “*turret*” type (Fig. 6). The cupola of the tower is sustained by eight arches supported by eight reinforced concrete pillars.

The main façade is dominated by three window openings placed above the entrance, extended on the height of two storeys (first and second floors), framed by architectural forms of arch type and supported on brick masonry pillars.

Elements of Romanian traditional architecture are visible on all façades in the framing areas of all openings (especially of those for the windows) and at the marking of the cornice. On the lateral façades the traditional architecture is present in the area of the terraces, where seven arched vaults sustained by brick masonry pillars can be seen (Fig. 7).

Fig. 6. Detail of the reinforced concrete turret.

The *architectural* and *structural descriptions* of the building were made in the paper entitled “*Technical assessment and strengthening of an architectural and historical monument building in Romania. A case study.*” (Vlad and Vlad, 2009).

Fig. 7. View of the lateral façade.

<u>Architectural description</u>

The shape in plane of the building can be inscribed in a rectangle having its sides equal to 35 m and 45 m, respectively (Fig. 8). Its configuration consists of four wings, which realize an in-plane tubular shape, generating a central perimeter (16 m × 16 m) of “interior courtyard” type.

The building has a general basement (B) of about 5.0 m height, a ground floor of about 6.0 m height (P), a partial mezzanine occupancy of about 30% of the ground floor space (M), two more storeys of 5.0 m height (I), respectively 4.0 m (II), and an attic of 3.0 m height (A). The attic-storey can be found only on three of the four sides of the building (N, E, W).

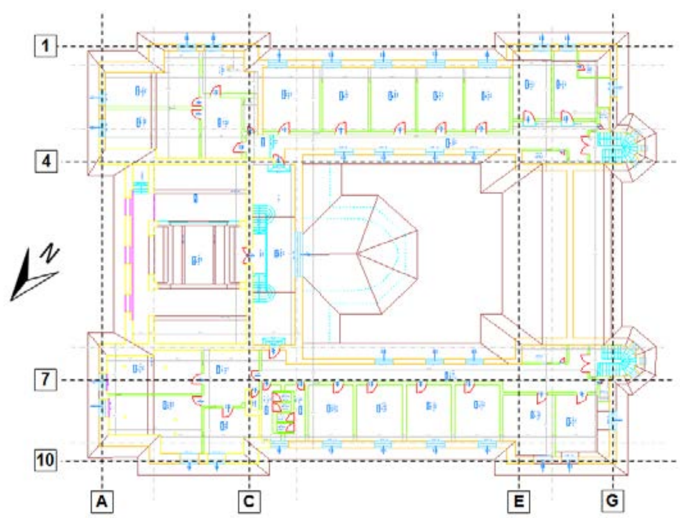


Fig. 8. Ground floor layout.

From the north wing, by two symmetrical stairs, one can reach a hall placed at the level + 18.85 m where from the access into the tower is possible.

The roof of the building is of general wood framework type, with a cover of ceramic tiles.

The principal access into the building is located on the north side, a large door by which one can enter into the central hall; from there the access to the following two storeys is assured by a monumental stair. On the south wing of the building there are two secondary staircases for the access to the three storeys of the superstructure.

Having in mind the provisions existing in the actual generation of codes, the entire building didn't have a favorable behavior to seismic actions.

From an architectural point of view, the main deficiencies were the followings: the building didn't have a regular, compact and symmetric shape in-plane and the existing dissymmetries in the volume, masse and stiffness distributions, as well as the big and different storey heights make it vulnerable to seismic actions.

<u>Structural description</u>

The overall structural system of this building consists of: the superstructure, the substructure, the structure of foundation and the foundation medium. For an existing building, such as that of the "National History and Archeology Museum", the above constituent parts were identified. In the following, some relevant information is given.

The superstructure comprises the storeys situated above the ground-floor (the floor above the semi-basement): the ground-floor, the partial mezzanine, the first floor, the second floor, the attic and the tower.

The vertical component of the structural system of the superstructure consisting of structural masonry walls had the following thicknesses: exterior walls at the ground and at the first storeys - 70 cm (2 ½ bricks), and at the second and at the attic storeys - 56 cm (2 bricks); interior walls at the ground and at the first storeys - 56 cm (2 bricks) and at the second and the attic storey - 42 cm (1½ bricks).

For each main direction the structural masonry walls were disposed along four axes (Fig. 8), as follows: on the longitudinal direction (axes "1", "4", "7" and "10"); on the transversal direction (axes "A", "C", "E" and "G").

The main structural deficiencies of the vertical component of the structural system were the followings:

- *irregularities in the disposing of door and window openings, together with the variability of the dimensions of these openings, both in the horizontal sections and in the vertical planes;*
- *the fact that the structural wall horizontal section areas differed on the two main directions of the building (as an example, at the ground level $A_{masonry,long} \cong 100m^2$ and $A_{masonry,transv} \cong 65m^2$);*
- *there were also irregularities of the structural walls horizontal sections, at each storey, on the vertical direction.*

The horizontal component of the structural system of the superstructure consists of four floor structures with steel girders and reinforced-concrete plates. The floor of the mezzanine is a reinforced-concrete one, with a very small area. The floor above the first level is incomplete (an area of about 150 m^2 situated between axes "F"-"G" and "4"-"7", in the "Adrian Radulescu Hall" zone is missing, Fig. 9). Above the attic, no floor is present.

Fig. 9. "Adrian Radulescu Hall" at second floor.

The main structural deficiencies of the horizontal components of the structural system were the followings:

- *the limited floor area of the mezzanine storey (which was a later structural modification) represents a local zone of irregularity which affects the structural walls stiffness and contributes to an eccentric distribution of masses;*
- *the lack of a floor area at the first storey created by the existence of the "Adrian Radulescu Hall" is an irregularity that can lead to important damage in this part of the building during an earthquake;*
- *the lack of a reinforced–concrete floor at the attic storey.*

The substructure of the building is 6 to 8 m high and consists of stone masonry walls, constituting the general basement (Fig. 10).

Fig. 10. View of the basement of MINA building.

The structure of the foundation consists of continuous stone cyclopean concrete walls type of approximately 10m height, beneath all the substructure walls. This information was taken from the National Archive documents of Constanța and from the press at that time, and was confirmed in 2008 by performing a geotechnical study (Fig. 11).

Fig. 11. MINA building: structure of foundation.

AMBIENT VIBRATION MEASUREMENTS

Within the seismic assessment it was considered necessary to identify the parameters governing the dynamic behavior of the building by performing vibration testing. In many respects, the practice of vibration testing to such masonry building is more of an art than a science. The type of test, the extent of the test and the required quality of the results all follow from the defined objectives:

- to obtain mode frequencies of the building;
- to obtain mode shapes and damping information for the building;
- to calibrate (correlate) a finite element structural model of analysis with measured results from the actual building, in order to assess the effects of a range of in-time modifications, and to obtain finally a theoretical model as a better representation of the dynamic characteristics of the real building; in other words, the requirement is to obtain a structural model of analysis of the building that is suitable for the given purpose – its technical assessment.

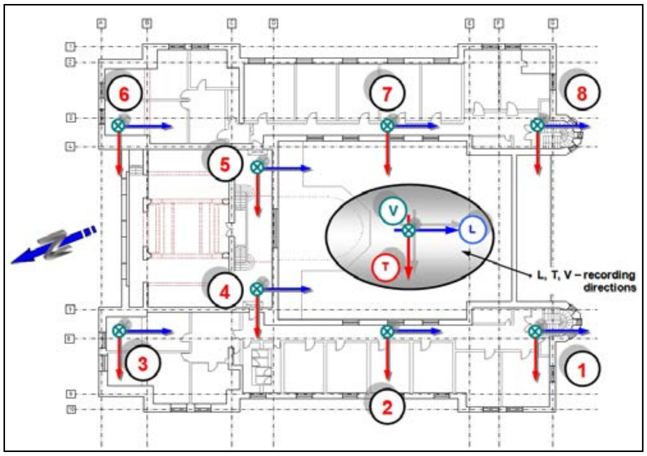
An experimental study was developed to evaluate the dynamic behavior of the building by applying the ambient vibration testing method. This method is relatively simple and requires equipment easy to be transported; such a test was performed with the building in use.

The response of the structure in time domain was recorded with highly sensitive sensors, compatible with the data acquisition system. The equipment consisted of SS-1 Ranger seismometers, a 16-channel fully portable acquisition unit which controls the outputs from the seismometers, connecting cables and a laptop. Eight short period velocity - type transducers were used to record the motions caused by ambient vibrations. The following typical types of analysis have been carried out:

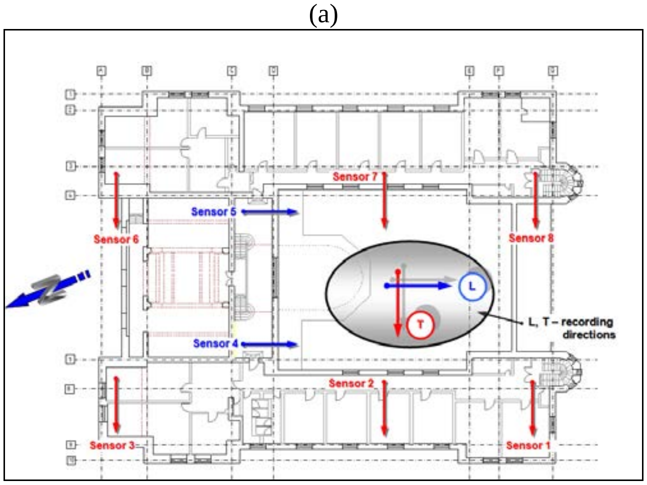
- numerical integration in time domain, obtaining in this manner from the basic signal (velocities) the vibration displacements;
- numerical derivation in time domain, obtaining in this manner from the basic signal (velocities) the vibration accelerations;
- Fast Fourier Transform (FFT) of the real signal, both for velocities and displacements (Fourier Amplitude Spectra);
- auto-correlation functions (cross-correlation of an input signal with itself), by means of which it was possible to detect an inherent periodicity in the signal itself and to determine the damping ratio;
- computation of maximum displacement values in different points of interest;
- simple mathematical combinations (sums or differences) between some primary records to indicate, when appropriate, average movements or rotations in different planes of oscillation;
- Fourier Amplitude Spectra for the above mentioned combinations.

The time domain representations (velocities and displacements) were performed in view of getting an overall image of the spatial motion of the building subjected to environmental vibrations. The Fourier Amplitude Spectra and the auto-correlation functions emphasized the frequency content of the recorded motions, as well as the frequencies of the dominant compounds.

The number of measuring points was established at the attic level, in several configurations, as shown in Fig. 12a, b.



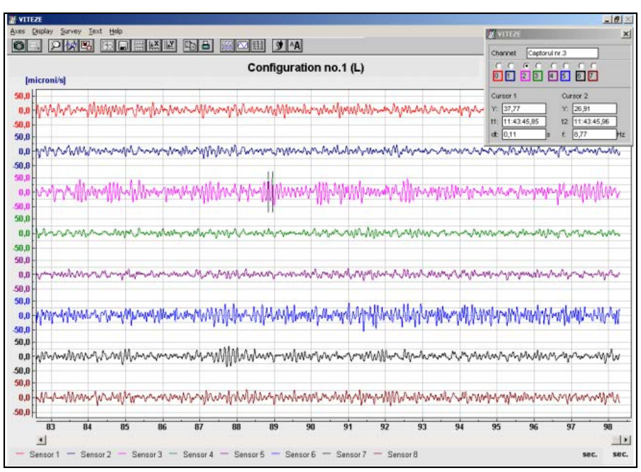
(a)



(b)

Fig. 12. Location of sensors at attic level.

Fig. 13÷16 present time domains, amplitude Fourier spectra, and auto-correlation functions representations, both for longitudinal and transversal directions.



(a)

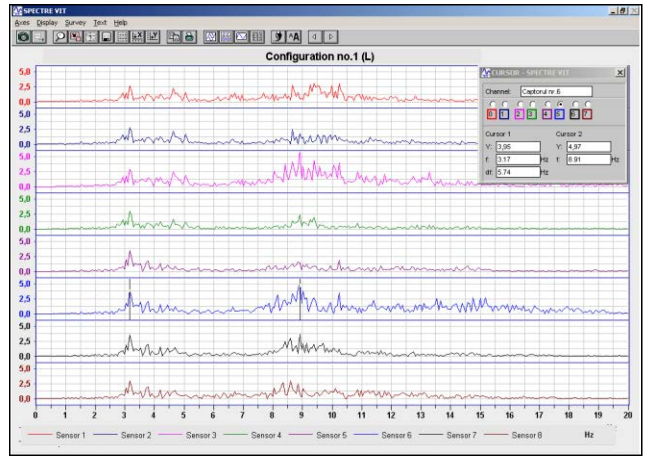


Fig. 13. Ambient vibration testing; longitudinal direction; velocities (μm/s). Time domain (a) and Fourier amplitude spectra representations (b).

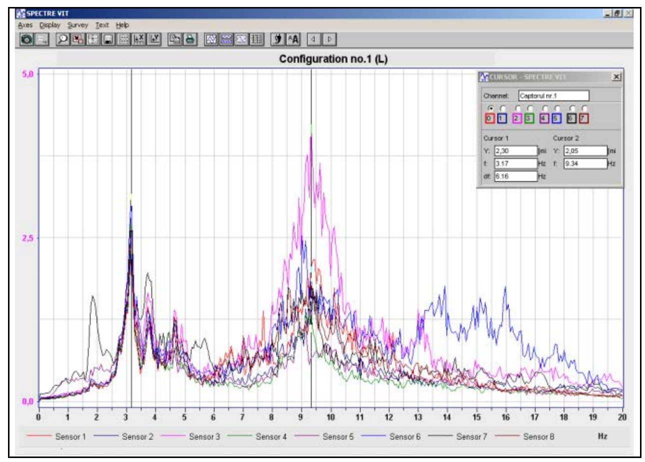


Fig. 14. Ambient vibration testing; longitudinal direction; velocities (μm/s). Average Fourier amplitude spectra representations.

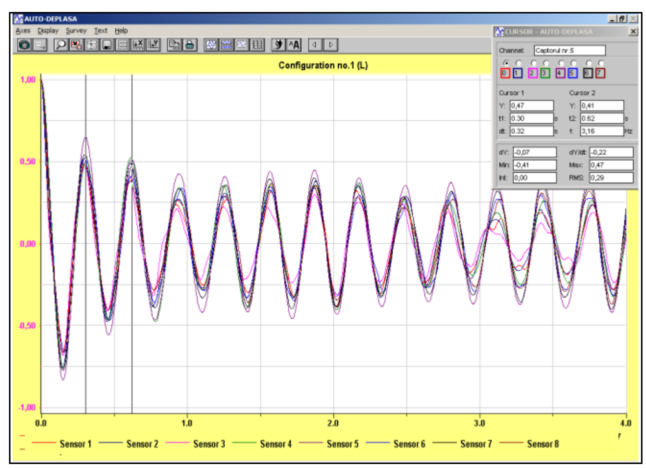
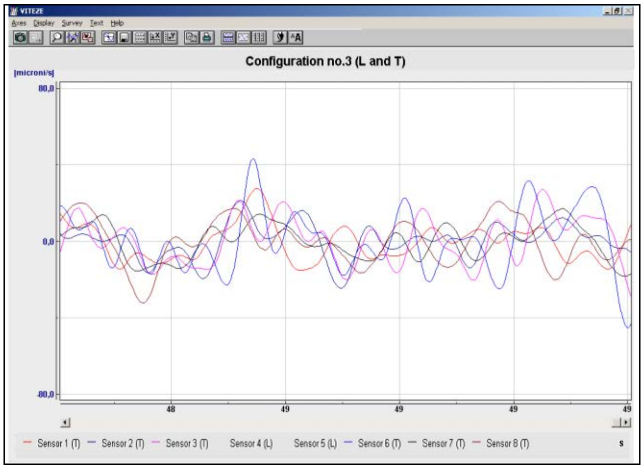
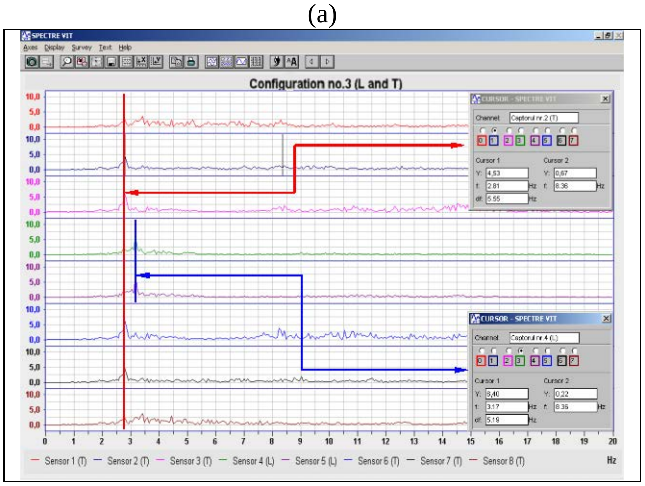


Fig. 15. Ambient vibration testing; longitudinal direction; auto-correlation functions representation.



(a)



(b)

Fig. 16. Ambient vibration testing; longitudinal and transversal directions; velocities ($\mu m/s$). Time domains (a) and Fourier amplitude spectra (b) representations.

By processing the experimental data for the performed recordings, the fundamental eigenfrequencies/eigenperiods of vibrations of the MINA building have been established. These *measured values* are shown in Table 1.

Table 1. MINA building – eigenvalues

Recording direction	Eigenvalues	
	Eigenfrequencies (Hz)	Eigenperiods (s)
Longitudinal	3.17	0.32
Transversal	2.81	0.35
Vertical	8…20	0.05…0.13
Torsion	3.80	0.26

<u>Some remarks after performing ambient vibration tests</u>

1. The recorded signals for identifying the dynamic characteristics of vibration of the building have revealed a *non-synchronism* between the motion recorded in various points, fact that proved that the floor structure of the attic didn't provide an acceptable cooperation of the structural load-bearing masonry walls with the floor structure at this level (even for low dynamic loading conditions); this was confirmed by the observed general micro-cracking state and by the existing cracks, both in the structural masonry walls and floor.
2. The examination of the fundamental eigenvalues derived from records showed that these pertain to a relatively 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 (there are no noteworthy differences between the values of the fundamental natural frequencies along the two principal directions).
3. The results also revealed a higher degree of flexibility on the transversal direction of the building (corresponding to E-W direction).
4. Although the shape in plane of the building is quasi-symmetrical, the existing dissymmetries in the volume, masse and stiffness distributions, as well as the big and different storey heights, led to significant rotational motions and modal coupling (the phenomenon of torsion was present, $T_{TORSION} = 0.26$s).
5. On the basis of auto-correlation functions of the recorded signals, it turned out that the values of the fraction of critical damping obtained on the basis of specific processing pertain to the interval 3…4%, which is quite low compared with those obtained for similar masonry buildings (at least 6%).

In conclusion, based on the records and on the results of the signal processing, it can be stated that MINA building had a high vulnerability degree to strong seismic actions and, therefore, it required extensive interventions for strengthening and making it safe.

THE RESULT OF THE TECHNICAL ASSESSMENT

The technical assessment revealed that almost all first level masonry structural walls presented a *brittle mode of failure* and, more than that, the first level was of "*weak and soft story*" type. After performing the entire process of the technical assessment, it was concluded that the structural system of the existing building does not resist (in the elastic range of behavior) to the shear force established according to the seismic code in force. Adding the main deficiencies of the vertical and horizontal components of the structural system, the building was classified in the *first seismic risk class "$R_S I$"*, according to the Romanian technical legislation (building with a high level risk of collapse in case of occurrence of an earthquake corresponding to the code seismic intensity of the Constanţa city, a_g=0.16g). All the aspects mentioned in the paragraph "*Content of the technical assessment*" are presented in detail in the papers (Vlad and Vlad, 2009) and (Vlad, 2009).

The owner of this historical and architectural monument decided to go ahead to the next step, which was the strengthening of the MINA building.

ASPECTS OF STRUCTURAL STRENGTHENING OF MASONRY BUILDINGS TO SEISMIC ACTION

The assessment of the structural behavior of old masonry buildings under seismic loadings cannot be as accurate and reliable as for new ones, due to the inherent difficulties in conceiving structural models of analysis. In the following, some aspects regarding buildings with masonry structural walls will be presented.

a) The vertical structural elements of the superstructure are: *isolated solid structural walls* (without window openings) and *structural walls with one or more lines with openings*; to the second type the following elements can be distinguished: *piers* – vertical structural elements and *lintels* – horizontal elements with beam or arch effects and, sometimes, with combined effects (beam + arch).
b) Frequently, to masonry buildings, the *failure modes* of the vertical structural elements (identified by structural analysis or caused by earthquakes) are of *brittle* type (cracks and breaks in inclined sections to isolated solid structural walls, masonry areas under windows, joints and lintels); this type of damage is caused by the principal tension stresses developed as effect of the shear forces, or as effect of the insufficient ductility capacity to bending.
c) As a result of the inappropriate constructive framing and proportioning deficiencies, sometimes, the floor structures are severely damaged by earthquake and may lead to "*structural disintegration*", characterized by effects of "*partial collapse*", or "*total collapse*", of the building.
d) The damage mentioned at items (b) and (c) was observed and studied for many buildings that have undergone the Skopje (1963), Banja-Luka (1969), Vrancea (1977), Thessaloniki (1978), El Asnam (1980) and s.o. earthquakes. The above mentioned seismic events were used by the Romanian engineer Emilian Titaru to elaborate a theory regarding the "*failure in inclined sections caused by shear force*".
e) In the past, masonry buildings have been repaired or strengthened by using some conventional seismic upgrading methods, which often have been proved both ineffective and incompatible with the original structure. The strengthening solution that has to be adopted is sometimes expensive, and sometimes is not acceptable for authorities. The strengthening practice for masonry buildings in Romania can be summarized as follows:
    - *option 1*: application of reinforced concrete jacketing using shotcrete technology for all damaged vertical masonry walls, or for those with potential brittle failure tendency (in severe cases on both sides of the structural elements); by this practice the following improvements for the simple masonry structural elements can be achieved: the increase of the bending strength and stiffness capacities; the elimination of the brittle mode of failure through fissures – cracks in inclined sections caused by shear force; achieving the necessary capacity of ductility for combined bending and axial stresses developed after the structural element yielding;
    - *option 2:* introduction of a subsystem of cast-in-place reinforced concrete structural walls; sometimes, as a result of its strength and stiffness characteristics, this subsystem may become predominant in relation to the existing simple masonry subsystem, so that, practically, the latest does not require the strengthening of its elements;
    - *option 3:* combining the two previous options, as follows: introduction of a subsystem of cast-in-place reinforced concrete structural walls with prevailing effects of the strength and stiffness capacities and, if necessary, the strengthening of some existing masonry walls.
f) In the process of the strengthening solution design, according to one of the three above options, the following aspects should be kept in mind:
    - establishing and imposing in a conscious manner by structural analysis, framing and proportioning, the principal components of the strengthened structural system: the *superstructure* and the *physical basis of the structural system* (consisting in the substructure and the structure of foundation);
    - establishing and imposing by structural analysis, framing and proportioning, the *yielding mechanism*, namely of the *energy dissipation mechanism* of the superstructure, which is constituted of the sections' ensemble (areas) where post-elastic deformations develop (idealized in structural model of analysis as plastic hinges);
    - structural analysis, framing and proportioning of the structure of foundation for the strengthened structural system of the building; the designed solution must ensure a proper interaction between the existing structure of foundation and the new foundation for the structural elements which were strengthened; it also must ensure a proper interaction with the new foundation of the new structural elements of the strengthening solution;
    - other main important aspects are related to the balance of the overturning moments due to seismic loads; in this respect, it is necessary to conceive a "*transfer"* structure of foundation which needs connections between the existing and the new foundations, thus resulting a network of foundation structural elements.

g) Based on acquired experience, the execution details for a foundation structure must be performed, having in mind the following well-known principle: *"the new structural elements of the strengthening solution have to have foundations with the same inferior levels as the existing masonry wall foundations"*.

h) Sometimes, it is necessary and advantageous that the elements of the new structure of foundation develop on the entire height of the basement.

Some of these principles were applied for the strengthening solution of the MINA building, and those corresponding to the structure of foundation are presented in detail in the companion paper 2.29 at this conference.

THE STRENGTHENING SOLUTION

In the design of the rehabilitation, the concept of "*spectral position*" was used. By "*spectral position*" it is understood the pair of values represented by the fundamental eigenperiod ($T_{n,1}$) and the base shear force coefficient ($c_{B,y}$), corresponding to the maximum strength capacity offered by the structural system, considering the associated mechanism of yielding. The "*spectral principle*" of the strengthening solution can be thus expressed: for improving the safety of the building to strong future seismic actions, its present "*unfavorable*" spectral position should be changed to a "*favorable*" one. According to inelastic response spectra for Romania earthquakes, this means the shortening of the fundamental period of vibration and the increasing of the strength capacity of the building.

For the building that is the subject of this paper the spectral positions correspond to the following characteristics: on the longitudinal direction ($T_{n,1}$=0.4 s; $c_{B,y}$=0.20) and on the transversal direction ($T_{n,1}$=0.4 s; $c_{B,y}$=0.13). The $c_{B,y}$ values correspond to the brittle mode of failure of the existing building.

One can notice that the pairs of values "$T_{n,1}$" and "$c_{B,y}$" placed the structural system of the building in unfavorable spectral positions of the inelastic response spectra. In Fig. 17 (Ţiţaru and Crăifăleanu, 2009), for a period of vibration $T_{n,1}$=0.4s and for the two values of $c_{B,y}$ (0.20 and 0.13), large values of the displacement can be observed. These unfavorable "spectral positions", on both directions, led to exaggerated values for the required ductility factors.

The "spectral principle" of the strengthening solution can be thus expressed: for improving the safety of the building to strong future seismic actions, its "unfavorable" spectral position must be changed to a "favorable" spectral position.

According to the aspects presented in the previous paragraph and understanding the behavior of the structural system of the building, the designer presented a strengthening solution. This proposed solution had to be modified in order to be approved by the "*National Committee for Historical Monuments*".

The design strengthening solution consists of the introduction of a subsystem of coupled reinforced concrete walls disposed along the perimeter of the existing building interior courtyard (Fig. 18).

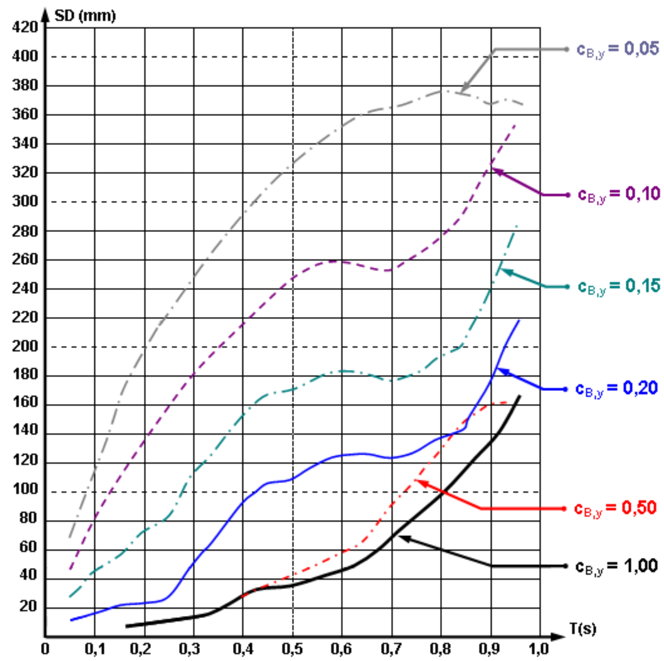


Fig. 17. Inelastic displacement response spectrum.

The strengthening subsystem of reinforced concrete walls, by the interaction with the masonry structural walls of the existing superstructure, will assure the following structural concepts:

- by its stiffness it will increase the overall structural stiffness of the building, thus obtaining a shortening of the fundamental period of vibration;
- by its strength capacity it will increase the value of the indicator of the strength capacity of the overall superstructure "$c_{B,y}$";
- by the interaction between the two subsystems of structural walls new structural elements of reinforced concrete and masonry will result; the new composed structural elements will have enough strength, stiffness and ductility, so that damage during a future strong earthquake be avoided;
- by its stiffness capacity, the strengthening subsystem of reinforced concrete walls will take over an important part of the induced seismic forces in the overall strengthened superstructure; as a result the internal forces generated by seismic actions in the masonry structural system of the superstructure will be significantly reduced (the risk of damage and of brittle mode of failure being thus eliminated).

The structural analysis was performed in compliance with the present existing national regulations, by use of the finite element method, applying the ETABS computer software.

As a result of the adopted strengthening solution for the existing building, imposed by its statute of "*architectural and historical building monument*", two 2D structural models of analysis have been formulated, one for the transversal direction and one for the longitudinal direction. The two structural models of analysis were conceived having in mind on one hand the existing masonry building, and on the other hand, the two reinforced concrete structural walls separately considered for each direction.

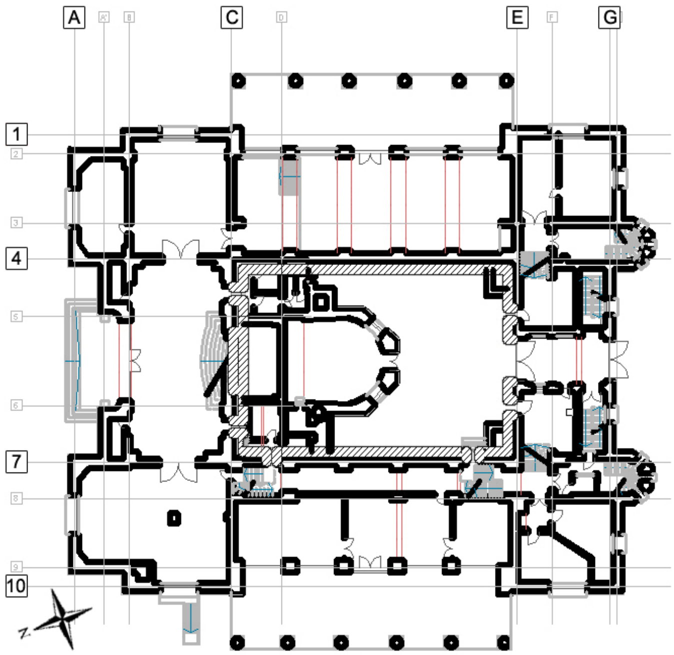


Fig. 18. The adopted strengthening solution.

For the *existing building*, an equivalent model of analysis "*stick*" type was adopted (Fig. 19,a), that is a fixed reinforced concrete "*column*" whose height was equal with the existing building height, namely 19 m (only the height of the superstructure was considered). It must be mentioned the fact that for these structural models of analysis only the height of the superstructure was considered (from the finished floor of the first level up to the superior part of the existing building), as the substructure was of thicker solid bricks and stone walls. More than that, the substructure had a reduced number of window openings, which conferred a much more resistance capacity in comparison with that of the first level.

Having in mind the assumptions used in structural analysis, the masonry wall was considered as an equivalent reinforced concrete wall with reduced thickness, which was obtained considering the ratio between the compressive strength of masonry and of concrete. In Fig. 19,b and Fig. 19,c, an evaluation of the actual load "*p*" is realized. Thus, only the weight of the masonry walls of the existing building was computed, as follows:

$$A_{existing\ floor} = 35\text{ m x } 35\text{ m} - 17\text{ m x } 17\text{ m} = 1270\text{ m}^2$$

$$A_{total} = 3.6\text{ floors x } 1270\text{ m}^2 = 4580\text{ m}^2$$

$$G_{total,\ masonry} = 4580\text{ m}^2\text{ x } 20\text{ kN/m}^2 = 91600\text{ kN}$$

Consequently, a distributed load "*p*" per meter resulted:

$$p = 91600\text{ kN/19 m} \approx 4800\text{ kN/m} = 4800\text{ daN/cm}$$

For the equivalent reinforced concrete "*stick*" a elasticity modulus E = 300,000 daN/cm^2 was considered.

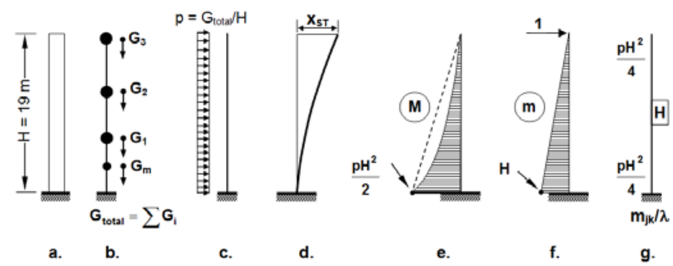


Fig. 19. Equivalent "stick". Conventional static deflection.

The equivalence of the existing building with a vertical reinforced concrete column was achieved on the basis of the equality between the fundamental eigenperiod of the "*stick*" and the fundamental eigenperiod of vibration of the existing building, that was obtained by instrumental investigations performed with Kinemetrics equipment. The value of the conventional static deflection "x_{ST}" was computed using the well-known Maxwell-Mohr formula:

$$x_{ST} = \int m \cdot M \cdot \frac{dx}{EI} \tag{7}$$

By applying the "*transformed lengths*" rule of integration (Fig. 19,e, f, g), it resulted:

$$6EI_{equiv.}x_{ST} = 6EI\int mM\frac{dx}{EI} = 2\cdot H\cdot H\cdot\frac{pH^2}{2} - H\cdot\frac{pH^2}{4}\cdot H =$$
$$= pH^4 - \frac{pH^4}{4} = \frac{3pH^4}{4} \tag{8}$$

From the above relation, the expression of the conventional static deflection, x_{ST}, can be obtained:

$$x_{ST} = \frac{pH^4}{8EI_{equiv.}} \tag{9}$$

The fundamental eigenperiod of vibration for the dynamic system with many dynamic degrees of freedom is given by:

$$T_{n,1} = 0{,}18\sqrt{x_{ST}} \tag{10}$$

By equalizing the expression of the fundamental eigenperiod of vibration (10) with the value of the fundamental eigenperiod of vibration instrumentally obtained:

$$T_{n,1} = 0{,}18\sqrt{x_{ST}} = 0.4\ s, \tag{11}$$

the value of the static deflection at the superior part of the existing building will result:

$$x_{ST} = 4.94\ cm \cong 5\ cm \tag{12}$$

Then, for the equivalent reinforced concrete stick, the value of the deflection at its superior part will result from the formula:

$$x_{ST} = \frac{pH^4}{8EI_{equiv.}} = 5\ cm \tag{13}$$

Finally, the inertia moment of the reinforced concrete equivalent stick resulted:

$$I_{equiv.} = \frac{pH^4}{8E \cdot x_{ST}} = \frac{4800 daN/cm \cdot (1900 cm)^4}{8 \cdot 300{,}000 daN/cm^2 \cdot 5cm} = 0.521 \cdot 10^{10}\ cm^4$$

The two structural model of analysis "2D" are presented in Fig. 19. The links between the two structural walls and the stick that substitutes the existing building consist in horizontal pendulums of infinite stiffness. The reinforced concrete structural walls of strengthening with window openings behave as coupled structural walls consisting of piers and structural coupling beams.

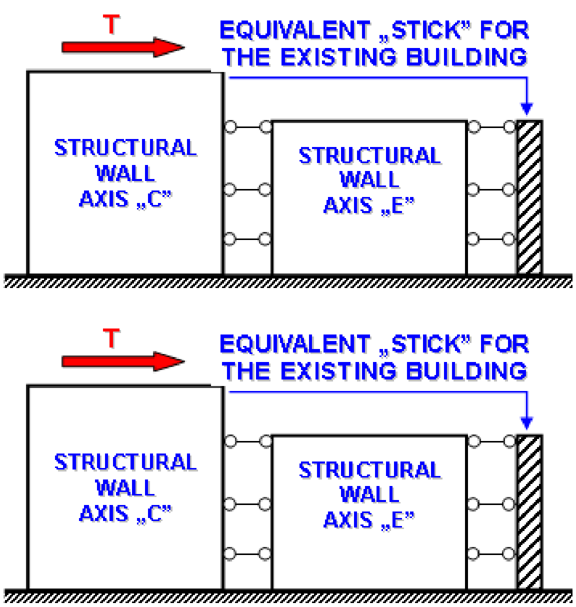


Fig. 19. "2D" structural models of analysis.

OBTAINED RESULTS

The introduction of the strengthening subsystem of coupled reinforced concrete walls will have the following two main effects:
(a) the shortening of the eigenperiod of vibration of the strengthened building in comparison with its value before strengthening, as follows:

- on the longitudinal direction (direction parallel to axes "4" and "7"), $T_{n,1}$=0.26s ($T_{n,1,measured}$ = 0.4 s);
- on the transversal direction (direction parallel to axes "C" and "E"), $T_{n,1}$=0.30s ($T_{n,1,measured}$=0.4 s).

(b) the decrease of the values of the base shear forces in the initial superstructure, as follows:

- on the direction parallel to axes "4" and "7", the base shear force will be reduced to 35.5%, compared to its value before strengthening;
- on the direction parallel to axes "C" and "E", the base shear force will be reduced to 54%, compared to its value before strengthening.

It was arrived to a value of the indicator of the strength capacity of the overall superstructure $c_{B,y}$= 0.25, and thus to acceptable values of displacements.

CONCLUSIONS

This paper presents a comprehensive summary of the recent practical engineering activity of the author.

1. The technical assessment and the strengthening of an old monumental unreinforced masonry building are domains where decisions are taken based on risk analysis, in order to reach a compromise between the historical value, the cost of the investigations, and the cost of interventions. Namely, in the first part, the *technical assessment* of the building and the *strengthening solution* that was architecturally and technically acceptable for authorities and economically feasible for the owner, are presented. In the second part of the paper, based on the structural concepts of the strengthening solution, a subsystem of coupled reinforced concrete walls, disposed along the perimeter of the existing building's interior courtyard, was designed.
2. The solid brick masonry walls showed tendencies of brittle failure in inclined sections due to the principal tension forces caused by shear force effect.
3. It was found out that the building has the tendency of localizing damage at the first level, with the development of a "*soft and weak first level*" effect (situation which corresponds to a possible general progressive collapse).
4. All the aspects related to the substructure (basements) and to the structure of foundation are presented in detail in the accompanying paper 2.29, entitled "*Foundation Structure Design for an Old Historical Building*".
5. The strengthening subsystem of coupled reinforced concrete walls will have the results that have been already presented in the section "*Obtained Results*". At present, the strengthening solution is already implemented, and the works for retrofitting and modernization of the museum will continue, as the owner will allocate the necessary funds.
6. Based on the experimental and analytical investigations carried out so far, one can conclude that the problem of

seismic resistance of old masonry buildings can be handled by means of adequate technical methods.

7. The strengthening of a monumental old unreinforced masonry building is engineering in its purest form. The relationships and responsibilities of the engineer with regard to other participants in the strengthening and rehabilitation process are unique.
8. In the followings, some photos taken during the strengthening works are presented.

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