ABSTRACT

Experience has shown that the spatial finite element structural models of analysis often fail to predict accurately the buildings fundamental eigencharacteristics, as they are “idealizations” conceived to assess the response of actual structures. The question that is being asked today by designers is the following: how will the tall buildings in Bucharest behave under the peculiar nature of the seismic motions occurred at intermediate-depth (Vrancea) and under the propagation-distance effects? Some aspects referring to the special local conditions of Bucharest (a sedimentary deposit of about 1500 m depth) will be presented. This subject is justified because it is known that the strong earthquakes that have been recorded have highlighted an accentuated “variability” of the seismic motions from one location to another, during the same earthquake, as well as from a seismic event to another in the same location. The paper also presents the results obtained for two tall buildings in Bucharest based on ambient vibration measurements, and the comparison with those obtained by the linear-elastic structural analysis and empirical formulas. The knowledge of the actual dynamic parameters of each tall building gave the possibility to perform a new calibration of the structural models of analysis in order to predict their behavior under future strong Vrancea earthquakes. Two other aspects of interest will be also mentioned: the geological and geotechnical conditions of Bucharest city area and the author’s vision on the propagation of the seismic waves for intermediate earthquakes. Some comments and summary results will also be presented.

Keywords: Vrancea; tall building; seismic wave propagation; intermediate earthquake; ambient vibration.

1. TALL BUILDINGS IN BUCHAREST

The seismic hazard for much of the territory of Romania is determined by the Vrancea earthquake nest, which generates from time to time intermediate-depth earthquakes and whose physical mechanism is still under debate. One of the largest urban agglomerations exposed to strong seismic motions occurring in the Vrancea area is the city of Bucharest. According to the present requirements of CTBUH, the buildings that have more than 14 levels and heights of 50 to 300 m could be considered “tall buildings”, those who exceed 300 m could be classified as “supertall buildings”, and those who go beyond 600 m “megatall buildings”, as shown in Figure 1. Bucharest, the capital of Romania, is the location of 60 completed tall buildings having more than 60 m, the tallest one reaching 137 m. This paper is devoted to two tall buildings – one of 120 m and the other of 75 m (Figure 1). Over decades, seismologists and structural engineers have gained extensive knowledge about why some buildings collapse (or are severely damaged), while others remain standing (even with minor damage) during a strong earthquake. A part of the answer for the buildings which exist in Bucharest lies within the lack of a design code until 1963 and in an inappropriate code until 1978. The first multi-story reinforced concrete residential buildings were built in the interwar period, when the most powerful influence over the structural design engineers was held by the German technical legislation. The period between 1920–1940 was characterized by an important economic and urban development of large towns in Romania, among them Bucharest having the prominent place. All the multi-story RC buildings were conceived, analyzed and designed by private design entities, which used “quick” design methods. A general characteristic of that period is that the design of buildings was carried out almost exclusively for gravity loads Vlad and Vlad (2008).
Figure 1. (a) "One Central Park" (117 m), Australia. (b) "432 Park Avenue" (426 m), New York. (c) "Shanghai Tower" (632 m), China (Internet). Overall image of (d) "Cathedral Plaza" building and of (e) “Globalworth Tower”, Bucharest.

The November 10, 1940 earthquake (Ms=7.4) severely damaged many multi-story residential buildings in Bucharest and led to the collapse of the 14 level “Carlton” building. This accident prompted the problem of buildings safety to seismic actions in Romania. In the mentioned period, buildings of unequal quality resulted, this situation being due to the capability and responsibility of both designers and entrepreneurs.

During the March 4, 1977 earthquake, 28 such multi-story buildings, located in the central area of Bucharest, have totally or partially collapsed, 20 of them being blocks of flats provided with RC structural members. Only two new tall apartment buildings partially collapsed.

Until 1990, the three tallest buildings in Bucharest were: “The House of Free Press” (104 m) built in 1956, the “Intercontinental Hotel” (87 m) built in 1970, and the “TVR Tower” (74 m), built in 1968. After 1990, over 50 buildings with different heights were built in Bucharest, the tallest being at present the “Sky Tower” (137 m). The three tallest buildings built before 1990 behaved well enough during the March 4, 1977 earthquake. Among the 50 tall buildings, the office-space segment has undergone an accelerated development, more than 15 such tall buildings being erected. Those who designed them had to face the following issue: the lack of experience concerning the design of tall buildings with large open spaces and several levels for car parking. The design of tall building substructures (the group of basements) and foundation structures in soft foundation environments also comprised a new challenge. Finally, the structural methods of analysis during the transition to the European standards have not been applied in a unitary and acceptable manner. Last but not the least, the technical disputes between the Romanian designers and the foreign investors’ consultant teams (which sometimes imposed solutions not approved by the initial designers) should be highlighted. All these mentioned aspects led to obtaining unequal high-rise buildings from the point of view of their safety to seismic actions. In order to have the picture complete, the vast majority of the owners is not interested in knowing if the building they now exploit match, from the point of view of their dynamic characteristics, with the ones that were originally designed.

The period 1992-2018 was influenced by some important external factors: the radical political changes after 1989 and the prospects of the EUROCODES implementation in engineering activities. The gained political freedom and the free market conditions had some negative side effects, meaning that the control of the design and construction activities decreased, leading to some cases of serious mistakes. As it was already mentioned, the absolute novelty of the 1990-2018 period consists of a trend to design and construct tall buildings. No one knows how these buildings will behave at the incidence of a future strong earthquake. The first thing we need to talk about is that of the local seismic conditions. The design of a “physical structural base” of a tall building (substructure + structure of foundation) for the soft foundation environment of Bucharest was a new challenge. The authorities approved the construction of such buildings in Bucharest, but had no concern in terms of drawing up a study on the foundation conditions for buildings with several underground levels for cars parking.
The biggest problem that Romania had to face since January 1, 2007 (date when Romania has joint the EU) was represented by a severe labor migration on one hand, and by the departure of eminent young students to study abroad who never returned, on the other hand. For having a complete picture, it is important to mention that the vast majority of specialists with important contributions in removing the March 4, 1977 effects are no longer in activity.

2. GEOLOGICAL AND GEOTECHNICAL CONDITIONS OF BUCHAREST CITY AREA

The city of Bucharest is situated in the Romanian Plane with microrelief resulting from erosion and sedimentary processes at about 160 km distance from the Vrancea seismic zone. It was built along the rivers Colentina and Dambovita which drain south-eastwards across the low relief. Geologically, Bucharest is situated above a sedimentary basin which has been formed in the central part of the Moesic Platform by accumulation of deep water deposits of clay, sand and gravel. Based on geotechnical information from over 2000 borings, it was concluded that beneath Bucharest there are seven alternating deposits of non-cohesive sedimentary deposits with different peculiarities and large thickness intervals. These shallow complexes were classified as follows (Ciugudean et al. 2006):

- **Layer 1**: recent surface sediments consisting of organic soil and clay-rich complexes with a local thickness reaching 15 m.
- **Layer 2**: upper sandy-clayey complex constituted partly of loess deposits, often moisture sensitive, and including sand layers; this unit ranges in thickness from 16 m in the North of the city to less than 1 m on the river sides.
- **Layer 3**: Colentina gravel complex consisting of poorly sorted, cross-stratified sand and gravel with large variations in the grain size, with interbedded clayey layers; within this deposit the level of groundwater is variable, ranging from 1.5 m to 14.0 m; the deposit thickness can locally reach 20 m.
  - **Layer 4**: intermediate clay complex which has in its composition alternating layers of brown and gray clays, with intercalation of hydrological fine confined sandy layers; the thickness of this deposit can reach a maximum of 23 m in the North of the city, but towards South it becomes very thin and disappears; this unit is believed to be of lacustrine origin.
- **Layer 5**: Mostistea sand-complex – a partially confined water-bearing layer constituted of gray, fine-grained sands with lenticular intercalations of clay; its thickness varies from 10 to 15 m and is present as a continuous layer beneath Bucharest.
- **Layer 6**: lacustrine-complex which consisted of clay and silty clay, with lenticular sandy layers, located mostly at the top of the deposit; its thickness varies from 10-60 m and the gray color and the carbonate content are indicative of a lacustrine origin.
- **Layer 7**: Fratesti sands complex is the deposit is the deepest founding layer with a thickness of 100 to 180 m and has three levels (A, B and C); it has in its composition sands and gravel, from which drinking water is usually pumped out.

An image of the seven distinct sedimentary layers is presented in Figure 2, where one can notice the dipping of the sedimentary deposit toward North.

![Figure 2. Lithological cross-section for the Bucharest area (Mandrescu et al. 2004)](image-url)
The above presentation offers a picture for the lithological succession from surface to a depth of about 250 m, but experimental tests using geophysical methods during the last earthquakes (1986, 1990) confirmed the existence of a geological deposit of sedimentary nature with depths up to 1500 m. To understand how the geological formation behave during very particular earthquakes generated by the Vrancea source was, and still is, a great challenge. In the paper (Sandi et al. 2004) the influence of Vrancea source mechanism versus that of local conditions upon the spectral content of the ground motion was presented. The rich accelerographic information obtained during the strong Vrancea earthquakes (1977, 1986, 1990) put to evidence locations with tendency to strong variability of the same from one event to another. The dynamic model adopted for the sedimentary geological deposit (consisting of eight layers) was defined as follows:

- layers in plane-parallel stratifications;
- homogeneous layers with jumps of properties at interfaces between the adjacent ones;
- isotropic material with physically linear behavior: damping characteristics independent of oscillation frequency;
- direction of wave propagation: vertical (normal to layer interfaces) and consideration of plane S waves (unique oscillation direction: horizontal);
- numbering of layers: downwards;
- imposing conditions of continuity of displacements and stresses at interfaces of adjacent layers.

Sinusoidal oscillations were basically considered, in order to determine (complex, scalar) transfer functions, defined as ratios of complex amplitudes at free surface to complex amplitudes of motion at lower boundary of a geological package. Besides that, spatial transfer functions, which are functions not only of the circular frequency, but also of the abscissa measured along the propagation direction, were also determined in order to get a picture of the deformation of the geological package, as a whole.

Among the obtained results the following aspect is of primary interest for the present paper. The main spectral peak of $T \approx 1.5$ s observed on March 4, 1977 seismic event at Bucharest (INCERC station) became on August 30, 1986 a secondary peak at the same place, as well as at other recording stations located inside or around Bucharest, and totally disappeared on May 30, 1990 at all stations referred to (Sandi et al. 2004). It can be concluded that the main factor having influenced the spectral content of seismic motions inside and around Bucharest was represented by the source mechanism. This aspect is very important for tall buildings in Bucharest (where thick relatively soft geological deposit exists) as long dominant periods, in the range of 1.5 s, should be expected in case of strong earthquakes. The performed parametric analysis for geological columns, like that of Bucharest, highlighted some relevant spectral peaks for periods longer than 1.6 s (dominant periods in case of highest magnitude earthquake). The result raises the question of the opportunity of completing tall buildings in Bucharest and, if it is done, under what technical conditions.

3. PROPAGATION OF THE SEISMIC WAVES

The stored potential energy which is released during an earthquake in the form of kinetic energy is radiated outwards from the area associated to the focus. The complexity of the observed records arises from the fact that it is possible that the intermediate Vrancea source has multiple subsources, and the energy radiation at a receiver station is the sum of the energy radiations from each of the subsources. In the process of transmitting the seismic vibrations from the seismic source (focus) to structures in a given location, three principal subsystems should be considered: the subsystem represented by the lithosphere, the subsystem represented by the geological sedimentary deposit, the subsystem represented by the existing structures at the surface of the Earth in a given location. Starting from the intermediate earthquake focus zone, the cycles of deformation are propagated through lithosphere, with the velocity of primary waves “P” and with the velocity of secondary waves “S”. These seismic “deformation waves” engage in vibration motions increasingly higher volumes of rock of the lithosphere. The immediate result of this physical phenomenon consists of an attenuation tendency of the seismic waves’ intensity (the reduction of peak values of acceleration and velocity together with the displacement amplitudes). In the following some aspects referring to the frequencies of the waves will be highlighted. The lithosphere vibrations act as an exciter for the above existing sedimentary deposit. In specific analysis the sedimentary deposit (the ground) can be considered a
one-dimensional shear vibrating model, as shear vibration is predominant inside the ground during an earthquake. Thus, the ground can be converted into an appropriate number of layers of discrete masses connected by springs. The adopted structural model of analysis for the sedimentary deposit with n DOF has “n” vibration eigenmodes. The seismic waves that are propagated through the lithosphere can generate, in a considered location, a phenomenon known as “first partial resonance”, as a result of the identity or the approaching of the frequency value of one of the dominant seismic motion components with the value of the fundamental eigenfrequency of the sedimentary geological deposit. That's why it can be concluded that the transmission of the seismic vibration from the lithosphere to the sedimentary geological deposit corresponds to an effect of a first partial resonance.

The dominant component of the seismic motion which has generated in the sedimentary geological deposit the first partial resonance produces, in the transmission process of seismic vibrations from the sedimentary deposit to the existing structures above it, a “second partial resonance”. This new second partial resonance is present as a result of the vibration of the sedimentary geological deposit and the structures above it having the fundamental eigenfrequency of vibration equal to the fundamental eigenfrequency of the sedimentary geological deposit (or, one can say, to the frequency of the dominant component of the seismic motion in the lithosphere next to a given location).

4. SHORT ON STRUCTURAL ANALYSIS

As known, the structural performance of an existing tall building under wind and seismic loads depends on stiffness and mass distribution. The degree of structural performance can be estimated by studying the behavior of different structural models of analysis using specialized software based on finite elements.

The “structural models of analysis” used in the design process of tall buildings to loads generated by the dynamic actions are “idealizations” more or less effective, but conceived to get the real response of these buildings to actions like strong earthquakes, severe dynamic actions generated by wind gusts, blasts etc. Most of the designers of tall buildings in Romania use the SAP or ETABS software, but we must never forget that their lead author Eduard Wilson – Professor Emeritus of Structural Engineering at the University of California at Berkeley – stated: “STRUCTURAL ENGINEERING IS THE ART OF USING MATERIALS that have properties which can only by estimated, TO BUILD REAL STRUCTURES that can only be approximately analyzed, TO WITHSTAND FORCES that are not accurately known, SO THAT OUR RESPONSIBILITY WITH RESPECT TO PUBLIC SAFETY IS SATISFIED.” The same very well known Professor and software developer wrote on July 13, 2015 a communication entitled “Termination of the Response Spectrum Method - RSM”, in which stated: “Ray Clough and I regret we created the approximate response spectrum method for seismic analysis in 1962. After working with RSM for over 50 year, I recommend it not be used for seismic analysis.”

Within the EUROCODE 8 and the P100-1/2013 Romanian Code it is specified that “the modal response spectrum analysis is applicable to all types of buildings” (paragraph 4.3.3.3 in EUROCODE 8 and paragraph 4.5.3.3 in P100-1/2013). Considering Professor Wilson’s opinion, then all the tall building in Romania were analyzed by a method that should have been abandoned for a long time.

The company “Computers and Structures, Inc”, CSI, has recently added the ability to generate “time histories” from user specified spectra. Therefore it is now possible for every structural engineer to conduct linear time-history response analysis which satisfies all tall buildings code requirements Wilson (2015). After they perform time-history analysis engineers will realize that the non-linear analysis is easy using the “fast non-linear analysis method” (25 years tested), by replacing a few linear elements with non-linear elements. With the increasing performance of the computers, and as a result of developing the ability to use dedicated software, along with the users’ belief in the infallibility of the adopted structural models of analysis (high level modeling of the detailed features), there is a tendency that a dangerous “sufficienza/exaggerated self-satisfaction” be present, by giving a maximum credibility to the numerical analysis.

The correctness and the efficiency of the structural models of analysis used in the design process can be verified (confirmed) at the moment of the incidence of any of the aforementioned dynamic actions, but it is preferable to validate them prior to the occurrence of such a major event. With regard to “new tall buildings”, the efficiency of the design processes, as well as their practical implementation, can be validated by recording their eigenvibrations using equipment of high sensitivity. The owners and
developers should know the structural performance of the buildings they own, or they manage, and to have the basic data that are absolutely necessary for a rapid and accurate technical assessment after the occurrence of a strong seismic event. The existence of such basic information greatly reduces the term of non-use of the buildings (especially those destined for offices), shows accurately the damaged areas even if they are not visible, eliminates the lack of competence or experience of those called to assess the situation after the event, and reduces to the maximum the restarting of the current activity in these buildings.

5. INSTRUMENTAL INVESTIGATIONS

In order to evaluate the real structural performance of a tall building in Romania there are available two methods, both conditioned by the existence of an instrumental data acquisition system of high performance, of specialized software, and of a competent team with proven experience in performing such works:

- *instrumental investigations by recording ambient vibrations* (a cheap, efficient and fast method);
- *seismic instrumentation of the building* (top approach in the field, costs dictated by its extension).

The *instrumental investigations programs* have highlighted the fact that not for a few times the structural models of analysis based on finite elements were not able to accurately provide the values of the fundamental eigenperiods of vibrations. Most of the times, these eigenperiod values were overestimated. This means that in reality the tall buildings resulted more rigid as a result of the fact that the design process is usually conservative. On the other hand the effects of eccentricity associated to the torsion request, and the connections at the interfaces “superstructure - substructure”, “substructure – structure of foundation” can be incorrectly modeled, aspect that leads to vibration eigencharacteristics with different values than the actual ones. A complete understanding of the structural performance of a tall building, in the linear range of its behavior to lateral actions, can be obtained only by determining experimentally its own vibration eigenmodes.

The “ambient vibration” dynamic excitation can be considered as having components at a range of periods. A structure responds to all periods of excitation but will resonate when some of the excitation component periods coincide with the eigenperiods of the structure. Through resonant amplification a dynamic excitation can generate response one or two orders of magnitude higher than a static load of the same amplitude. Conversely, very small excitations can generate measurable response of office towers, for example one person shifting his body weight Brownjohn (2003). In the following some aspects regarding the correlation between the results obtained by structural analysis of two tall buildings and those obtained experimentally will be presented. The tall buildings instrumental investigations at natural scale are still quite rarely used in Romania, and the correlation of the outcome with the results obtained numerically by the use of the structural models of analysis based on finite elements is even rarer. More than that, in the technical literature “papers or studies” devoted to this important aspect are also rarely published. The basis of this situation consists of the designers’ fear of providing the results of the structural analysis, and the lack of equipment for the acquisition and processing of the instrumental data.

The main objectives of the instrumental investigations that were performed for the two tall buildings that are subject of this paper were related to the following aspects:

- a dynamic characterization of the structural properties of the whole building, with the intent to diagnose its own “dynamic identity”;
- the identification of the eigendynamic characteristics of the building from ambient vibration tests (eigenperiods, eigenshapes, overall damping);
- pointing out the vulnerable potential areas of the building to future seismic actions;
- the influence of the foundation medium on the behaviour of the building to dynamic actions;
- the checking of the accuracy of the structural models of analysis used in the design process;
- the calibration of the structural models of analysis;
- the correlation of the instrumental information with the one obtained by structural analysis;
- the assessment of the floor structures to dynamic actions generated by human activities;
- the identification of possible influences between the investigated building and the adjacent ones;
- the interpretation the results obtained by instrumental investigations.
5.1 “Cathedral Plaza” building

The superstructure of the building consists of a ground level – GL (H = 4.90 m), 18 current levels (18 L) and a technical level (TL). The total height of the superstructure, measured from the quota ± 0.00 m to the cornice quota, is 74.50 m (Figure 3). In the horizontal plane, the superstructure is essentially quasi-rectangular in shape, withdrawn in the south area of the property. Typical floor dimensions of the upper floors are about 42.35 m x 22.10 m. The surface of the current level is equal to approximately 903 m².

The structural subsystem of the superstructure is of dual type. It is composed of a central RC shear core of composite structural walls and perimeter steel moment resisting frames, located on the main directions of the building. The perimeter steel moment resisting frames were introduced in order to balance the lateral stiffness of the whole structural system of the building. For this purpose they are braced with steel diagonals, the diagonal panels being extended over four levels, along the axes “A” and “E”, and over eight levels along the axis “5” (Figure 3).

The vertical components of the structural subsystem of the superstructure have the following characteristics:

- the central core of composite structural walls is eccentrically placed as against the center of mass of each floor structure, at every level; the thickness of the core structural walls is 60 cm, excepting those of axes “2” and “4”, having a thickness of 65 cm; from the functional point of view, the central core ensures the vertical circulation in the building and some technical spaces;
- the steel columns were encased in reinforced concrete in order to ensure them high resistance and fire protection conditions; their cross sections are square (75 cm x 75 cm).

The horizontal component of the structural subsystem of the superstructure is formed by the floor structures. These consist of the main beams of the frame, of the secondary beams (ribs) disposed between the main beams at about 2 m apart, and of reinforced concrete plates of 13 cm thickness. The steel secondary beams are resting on the reinforced concrete structural walls of the central core and on the perimeter frames.

The floor structures can be considered rigid diaphragms at every level of the construction. The two components of the structural subsystem of the superstructure (the central core + the perimeter braced frames) ensure optimal transmission paths for gravity and lateral loads to the “physical basis of the structural system” (the assembly consisting of the substructure and the foundation structure).

The substructure, consisting of four levels, is designed exclusively for vehicle parking and it occupies almost the entire area of the property being substantially expanded compared to the superstructure (Vlad and Vlad, 2017).

The structural subsystem of the substructure consists of the reinforced concrete structural walls of the central core continuing those of the superstructure, of the perimeter molded walls and, in addition, of the 18 columns that sustain the floors of the four basements, located in the substructure areas which do not have correspondence with the superstructure. It should be mentioned that all the substructure columns were achieved in “composite column solution”.

The horizontal component of the substructure consists of reinforced concrete slabs of 35 cm thickness.

To obtain the necessary information for the proposed objectives, several acquisition configurations were performed.

Ten sensors were placed in five sets of measurements within the building, and 4 sensors were used for the identification of possible influences between the investigated tall building and the low-rise adjacent ones (Figure 3).

Typical time domain recordings and the corresponding amplitude Fourier spectra are shown in Figure 4 and 5.

The recorded signals revealed the synchronism of the motions in the various instrumented points, fact that proves that the two subsystems of the superstructure work together.

It was found out that these eigenperiods pertain to a relatively narrow range of variation, which proves the fact that the building has a behavior in the elastic range quite homogeneous on both horizontal directions.
The absence of other higher eigenfrequencies of vibration proves that the building has a well defined dynamic identity. The processing of the recorded signals put to evidence a coupling between the vibrations of the building on the two main directions, which revealed the existence of an overall phenomenon of torsion.

Table 1 summarizes the values of the fundamental eigenperiods of vibration which emphasize a higher degree of flexibility on the transverse direction.
Table 1. Fundamental eigenvalues of vibration.

<table>
<thead>
<tr>
<th>“Cathedral Plaza” building</th>
<th>Fundamental eigenvalues of vibration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>Eigenperiod (s)</td>
</tr>
<tr>
<td>Transversal</td>
<td>1.26</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1.02</td>
</tr>
<tr>
<td>Torsion</td>
<td>0.61</td>
</tr>
</tbody>
</table>

5.2 “Globalworth Tower” building

The structural system of the “Globalworth Tower” (the tall wing) has in its composition a “superstructure” with a height regime \( \text{Ground} + \text{Mezzanine} + 22\text{F} + \text{Retired}\text{F} + \text{Technical}\text{F} \) (26 floors), a “substructure” composed of three underground parking lots (3B) and a “foundation structure” consisting of a general RC mat, supported by drilled piles with enlarged base.

The structural subsystem of the superstructure is of dual type. It consists of two cores made of RC coupled structural walls and of RC perimeter moment resisting frames. The main structural core has the in-plane dimensions equal to 20 m x 17 m, and is 120 m above-ground. The secondary structural core has the dimensions 10 m x 10 m and is completed on the entire height of the building excepting the last floor, where some structural walls were interrupted, in order to arrange a conference room with a large span. The thickness of the two cores exterior structural walls is 80 cm, while that of the structural walls inside them is equal to 40 cm, respectively 25 cm. The coupling beams have widths of 40 cm – 80 cm and heights of 80 cm – 250 cm.

The perimeter moment resisting frame columns have a circular shape with a diameter of 120 cm and are made of reinforced concrete. At some floors, steel encased RC columns were used as follows: for the corner columns steel encased profiles were used, up to the 10th floor, with gradual reduction of their section sizes; for the central columns “Maltese Cross” profiles (only on the height of the first five levels) were used. At the ground floor and mezzanine levels steel encased reinforced concrete columns were used. Part of the mezzanine beams were also of composite type.

The horizontal component of the structural subsystem of the superstructure consists of floor structures (RC slabs 10 cm thick and secondary beams – 20 cm x 60 cm – spaced at a distance of 2.70 m) and of perimeter frame beams. The secondary beams rest on the main inner core and on the perimeter frame beams.

The substructure of the “Globalworth Tower” is constituted of three basements for cars parking, technical facilities and a local bombproof. Its structural system consists of a vertical component (structural walls + columns) and a horizontal component (3 RC floors). The 40 cm perimeter walls are connected to the entire height with the molded walls, on which they are supported. The other structural walls of the basement are 60 cm to 120 cm thick. The thickness of the floor above basement 1 is equal to 35 cm, that of the floor over basement 2 is equal to 25 cm and that of the floor above basement 3 is equal to 40 cm.

The substructure of the highest part of the building was conceived to obtain a “rigid box”, consisting of the interior and perimeter RC structural walls, floors and the general mat.

The foundation structure is a RC general mat with variable thickness, ranging from 2.45 m to 2.95 m, supported by drilled piles (\( \Phi 106 \) cm) with enlarged base (\( D = 2.64 \) m). In order to equalize the bearing capacity and the settlements of these piles, injections with special solutions at their bases were achieved on the penetration depths in the sandy layer.

The data acquisition system used allowed simultaneously arrangements of 9 sensors within the adopted 9 configurations, in order to obtain the necessary information for the proposed objectives (Figure 6). The representations in the time domains (velocities and displacements) were performed in order to check whether the two components of the spatial structural system of the building (perimeter moment resisting frames and the RC structural cores) oscillate, or not, in phase.

The Fourier amplitude spectra and the autocorrelation functions highlighted the frequency content of the recorded motions (Figure 7), as well as the amplifications corresponding to the dominant components, which allowed the identification of the dynamic eigencharacteristics of the “Globalworth Tower” building.
The obtained results are summarized in Table 2 and it is evident there is a significant coupling between the translation and rotation motions.

The vibration eigenshapes did not reveal elastic, inertial and dissipative discontinuities of the building structural system, nor other areas potentially vulnerable to future seismic actions, excepting the ones already mentioned.

Table 2. First two eigenvalues of vibration.

<table>
<thead>
<tr>
<th>Globalworth Tower</th>
<th>Eigenvalues of vibration</th>
<th>Eigenvalue Mode 1</th>
<th>Eigenvalue Mode 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Eigenperiod (s)</td>
<td>Eigenfrequency (Hz)</td>
</tr>
<tr>
<td>Direction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transversal</td>
<td>1.82</td>
<td>0.55</td>
<td>0.37</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1.49</td>
<td>0.67</td>
<td>0.40</td>
</tr>
<tr>
<td>Torsion</td>
<td>0.91</td>
<td>1.10</td>
<td>0.31</td>
</tr>
</tbody>
</table>

The author of this paper didn’t have access to the structural analysis of the building. It was therefore considered necessary to verify the instrumental results (Table 2) through a series of simplified formulas specified by several different design codes (Table 3). It is obvious that the results are very similar.

Table 3. Fundamental eigenperiods of vibration.
The Burj Khalifa building is currently the tallest building in the world with a height above ground of 828 m. Its structural system designer has performed structural analysis to gravity, wind and seismic actions, using ETABS software. The following fundamental eigenperiods of vibration were obtained: $T_{1,x} = 1.90$ s; $T_{1,y} = 1.77$ s; $T_{1,z} = 1.77$ s; $T_{1,x} = 1.79$ s; $T_{1,z} = 1.80$ s (Baker et al. 2008). Applying to this megatall building the same approximate formulas which were used for the fundamental eigenperiods of vibration for “Globalworth Tower” the following values resulted: $T_{1}^{(A)} = C_h \frac{h}{h_0} = 0.020 \cdot \left(828/0.3048 \right)^{0.75} = 7.53$ s. Taking into account the upper limiting coefficient of the fundamental eigenperiod $C_u=1.4$, a fundamental eigenperiod of vibration for use in the preliminary design process was obtained: $T_{1}^{(design)} = C_u \cdot T_{1}^{(approx)} = 1.4 \cdot 7.53 = 10.54$ s, thus complying with the ASCE-SEI 7-10/2010 code provisions, according to which $T_{1}^{(design)} = 10.54$ s $< T_{1}^{(computed)} = 11.3$ s.

Based on the autocorrelation functions of the recorded signals, the fractions of the critical damping obtained by specific processing had the following values: transversal direction – 5.79%, longitudinal direction – 5.90% and vertical direction – 4.77%.

6. CONCLUSIONS

1. The rules of specification of the local ground conditions of few tens of meters, given in several codes, are non-realistic in case of the 1500 m depth Bucharest sedimentary deposit. The most important aspect to be mentioned is that in Bucharest the idea of a dominant period of the seismic motions (as it is the case of Mexico City) cannot be sustained, as the spectral components of “past” and “future” strong seismic motions was and will be determined by the “seismic source” and not by the “location”. In Bucharest there was an important tendency of the variability of the spectral contents of strong ground motions; that’s why an attempt of microzonation of the city is at least questionable.

2. The finite element structural models of analysis for tall buildings are established based on highly idealized properties and on the code restrictions in force that may not truly represent the new buildings that are being designed. Performing actual tests on full scale tall buildings, using as source of excitation the ambient vibrations, is the only sure way of assessing the reliability of the various assumptions employed in formulating finite element structural models of analysis. Under the unique seismic conditions that characterize the city of Bucharest, any tall building (in which hundreds of people work) needs to be instrumentally investigated. Getting usable and reliable information by instrumental investigations is not an easy task in case of tall buildings. In the current seismic design activity, engineers pay special attention for determining the fundamental eigenperiods of vibration for the structural systems of the buildings they design and, in function of these, they will establish the seismic design loads, displacements etc. In conclusion, computer structural models of analysis must be checked by actual building measurements (carried out immediately after the completion of the works), in order to obtain essential information about their elastic behavior. The importance of the “stiffness factor” of the structural system is now widely known in the dynamic behavior theory of the buildings to seismic actions. The current knowledge prove that during strong earthquakes the buildings designed at an usual insurance level are overstressed, meaning that, in certain areas, the stage of elastic behavior is no longer valid. Once this threshold is exceeded a non-linear physical behavior occurs being characterized, inter alia, by a decrease of stiffness on more or less extended areas. Obviously, as a result of this behavior, the eigenperiods characteristic to the stage of elastic behavior lose their significance. While in the stage of elastic behavior there is a significant amplification of the spectral components corresponding to the eigenperiods specific to this stage, during the postelastic behavior stage there
is a tendency of lengthening the predominant vibration periods, lengthening which is even more pronounced as the stress level is higher. For this reason, the designers are interested in knowing “the real eigenperiods”, which occur when the stress level is high. The eigenperiod concept, rigorously defined, is specific to the dynamic behavior in the linear range and loses its rigorous meaning when the behavior becomes nonlinear. However it is basically necessary and important to know the dynamic eigencharacteristics of a tall building in the elastic behavior range for two reasons: to reduce the term of non-use of a building after a strong earthquake and to be able to predict how the dynamic amplification phenomena are influenced by the severity of a seismic event. For these reasons, many designers are interested in using non-linear response spectra without worrying about the largely unsolved issues that the use of this concept rises (incorrect application of the principle for the nDOF structural models and special issues associated with defining non-linear response spectra with different levels of insurance).

3. In the case of the “Cathedral Plaza” building the values of the fundamental eigenperiods of vibration, instrumentally obtained, confirmed the ones that were obtained by linear structural analysis within no less than four different technical assessments, while for the “Global Tower” building these values were confirmed only by the ones based on codes simplified formulas.

4. Tall buildings in Bucharest are one of the few constructed facilities whose design relies upon analytical structural models of analysis based on finite elements within advanced software. They have not yet been validated at natural scale by a Vrancea strong seismic motion. The possibility of occurring damage will be strongly influenced by the energy delivered at frequencies near their resonance frequencies.

5. The experience gained after the 1977, 1986 and 1990 seismic events regarding the decrease in stiffness of some constructions (and the increase of their stiffness after being strengthened) showed that the determination of the fundamental eigenperiods of buildings using ambient vibration recordings is easier, more accurate and surer than the one based on data provided by accelerographic recordings.

7. REFERENCES

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