



SEISMIC STRUCTURAL DESIGN OF AN OFFICE BUILDING IN ROMANIA

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SUMMARY

This paper presents some considerations concerning the design of an office building in Romania. The architectural project states that the shape in the horizontal plane is an equilateral triangle. In each of the three peaks of the triangle “structural cores” with reinforced concrete walls are placed. The three cores are connected by a central steel structural subsystem with steel-deck floors. The building general description and a detailed presentation of the structural system concept and analysis are presented. The software used consisted of the ETABS program and the Romanian program ANELISE for inelastic dynamic analysis.

1. SEISMICITY OF ROMANIA

Romania is one of Europe's most seismically active regions, together with other Balkan and Mediterranean countries (Bulgaria, Turkey, Greece, the former Yugoslavia and Italy). The seismicity of Romania is mainly due to activity from within the Vrancea region: the source zone is at an intermediate depth (about 60 to 200 km), and it delivers more than 95% of the whole seismic energy for the country. This source zone has direct influence over about half the territory of Romania, producing high intensity earthquakes. The upper limit of magnitudes for Vrancea earthquakes is considered to be $M=8.0$.



Figure 1 – Seismic map of Romania

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2. SOME ASPECTS CONCERNING THE SEISMICITY OF PLOIESTI ZONE

Ploiesti town stands among the places with high seismic hazard, as it belongs to the most exposed region to strong Vrancea earthquakes. The fact that this town is characterized by severe seismic conditions is confirmed by several factors:

- its position in relation to the region where the strong ground motions occur;
- the seismic stations, placed in a building next to the one that makes the object of this paper, recorded the following peak acceleration values: August, 30/31,1986 earthquake (at the basement level: $a = 219.30 \text{ cm/s}^2$; 6th floor level: $a = 588.20 \text{ cm/s}^2$) and May, 30, 1990 earthquake (6th floor level: $a = 378.60 \text{ cm/s}^2$);
- the macroseismic data acquired during the 1940, 1977 and 1986 earthquakes;
- the high peak seismic acceleration values recorded in the near vicinity of Ploiesti town (Campina, $a = 270.60 \text{ cm/s}^2$ in 1990, respectively Valeni $a = 186.90 \text{ cm/s}^2$ in 1986);
- the engineering observation on the behavior of masonry buildings (Ploiesti-Nord district) and of industrial buildings (Bucharest and Plopeni), that have a repetitive character, during the Vrancea 1977 earthquake;
- the results of probabilistic hazard analysis performed for several regions subjected to Vrancea earthquakes, stated that towns like Focsani, Buzau, *Ploiesti*, Campina, Valeni are placed in the category with the highest seismic hazard.

Earthquake hazard of Ploiesti town depends not only on the seismicity of the region but also on its population density, economic development and degree of preparedness.

3. GENERAL DESCRIPTION OF THE BUILDING

The history of this building began in 1990. On its site, in the central area of Ploiesti town, on a ground of trapezoidal shape and with an area of approximately 5000 m^2 , a block of flats was meant to be built, and, consequently, it was designed in 1988. In the horizontal plane, this building had the shape of the letter “Y” and in the vertical plane 2 basement floors, ground floor and 19 stories.

The superstructure and the substructure consisted of reinforced concrete structural walls, and the foundation structure (that was also the “Y” shape type and of about 850 m^2) formed a general mat box system. Only the general mat was accomplished until 1990, when the works were ceased, as a result of the change of the political regime in Romania.

In 1997, the new owner decided to realize an office building with basement and 10 stories, located over the existing general mat of the initially designed building.

The architectural project established that the new building should have, in the horizontal plane, the shape of an equilateral triangle, having the sides of 50 m. In each of the three peaks of the triangle, structural cores (A_1 , A_2 and A_3) were placed, each of them being realized with structural reinforced concrete walls. A 4.0 m story height for the current story was fixed.

Within these cores were located stairs for the vertical circulation, sanitary facilities at each floor and panoramic elevators. The central zone, limited by the three cores, was endowed with space for offices (8 stories), a restaurant (9th floor) and accommodation spaces (10th floor). The architectural theme stipulated that the basement should have the following functions: technological spaces and a garage for a limited number of vehicles.

At the ground floor, a four-elevator system was foreseen, which, together with the traffic stairs within the structural cores A_1 , A_2 and A_3 , assure the vertical circulation within the entire building. The access to the restaurant and the accommodation rooms is also achieved by means of the panoramic elevator located within the A_1 core.

4. STRUCTURAL SYSTEM CONCEPT

The building has a *dual* type structural system, consisting of a reinforced concrete subsystem and a steel subsystem (Figure 2).

The superstructure of the building, of composite type, has two different parts:

- a *reinforced concrete structural subsystem* formed by the three structural cores placed in the peaks of the equilateral triangle, their main role being that of assuring the lateral strength of the building to the seismic loadings;
- a *steel structural subsystem*, having a structural role mainly towards the gravity loadings.

The building's substructure: comprises in fact its basement, and is made of structural reinforced concrete walls.

The foundation structure: when establishing the conception for the achievement of the foundation structure, the possibility of including the existing mat box system (partially realized) in the overall structural system of the new building was taken into account, in a favorable manner from the structural, functional, technological and economical efficiency point of view.

4.1 The superstructure of the building

a. *The reinforced concrete structural subsystem* (Figure 3) consists of three multistoried cores (A_1 , A_2 and A_3), having identical in plane configurations, but different heights ($A_1 - 44$ m, $A_2 - 36$ m, $A_3 - 36$ m). The design of the reinforced concrete structural cores had, as a starting point, the architectural layout within which the position of the door and window openings, of the stairs, of the facilities, of the panoramic elevators and of the hallways was imposed. One can state that the positions and dimensions of the window and door openings are maintained the same at all levels. Thus, the constructive layout of each reinforced concrete core resulted in "tube with openings" type. Within each "tube with openings" the structural walls are connected one to another by coupling beams, so that each of the multistoried cores was designed to behave at horizontal forces as a vertical beam fixed in the foundation structure. The thickness of the structural walls is equal to 35 cm, with local increase in thickness around the opening in the symmetry axis. In this area with the in plane dimension of about 5 m the thickness of the structural wall reaches 1 m. In conclusion, the reinforced concrete structural subsystem, consisting of the three multiple linked box-type profiles assure the building's strength to seismic actions, taking over in the same time both the gravity loadings and the reactions given by the main beams belonging to the steel structural subsystem.

b. *The steel structural subsystem* is placed in the central zone of the building and is connected to the three reinforced concrete cores, placed in the peaks of the equilateral triangle (Figure 2). The steel structural subsystem consists of three moment resisting frames and composite type floor structures.

The moment resisting steel frames consist of the following structural elements:

- 4 steel columns, which, from the fire protection reasons, have been encased in reinforced concrete, thus leading to 4 composite – type columns (steel encased concrete columns), of which one column in the center of the building (SM1) and each of the other three on the half of each side of the equilateral triangle; in the vertical plane of the building, the four columns have been realized in four segments being spliced by direct contact (without gap between their sides, and having acceptable deviations of maximum 0.2 mm). *The central column* (SM1) has a compound section carried out by welding, consisting of a central pipe and six branches being positioned along the main beams axis of the floor structures. This column supports 6 main beams disposed in a radial manner (Figure 4). *The three peripheral columns* (SM2) have a compounded section carried out by welding, consisting of four branches being positioned along the main beams axis of the floor structures (Figure 5). This type of column supports 3 main beams (a radial beam and two peripheral beams).
- 3 main steel beams (R1), that link the central column with the three peripheral columns placed on the half of each side of the equilateral triangle; these beams are pinned on the consoles of the

central column (SM1), forming resistant rigid joints with SM2 type columns. The main beams (R1) are provided with cantilever supports for the main beams (R3).

The floor structures are of composite type, being achieved of slabs steel-deck type and steel beams. They are situated at the levels: +3800, +7800, +11800, +15800, 19800, +23800, +27800 and +31800. The steel beams that are part of the floor structures are:

- main steel beams (R2) that are pinned on the central column's cantilever (SM1), thus being realized a simple supported type connection on the special steel devices fixed in the reinforced concrete cores; these beams are designed with cantilevers that support the secondary afferent beams;
- main steel beams (R3 and R4) that are also designed with cantilevers for supporting the secondary afferent beams;
- secondary steel beams (N0, N1, N2, N3, N4) that are simple supported by the main beams (R2, R3 and R4).

All these beams are compounded section, being made of welded steel plates. On the superior flanges of the main and secondary beams connectors for the anchorage of the reinforced concrete slabs were designed. The connectors afferent to the main beams were factory welded, and those afferent to the secondary beams were welded at the building site.

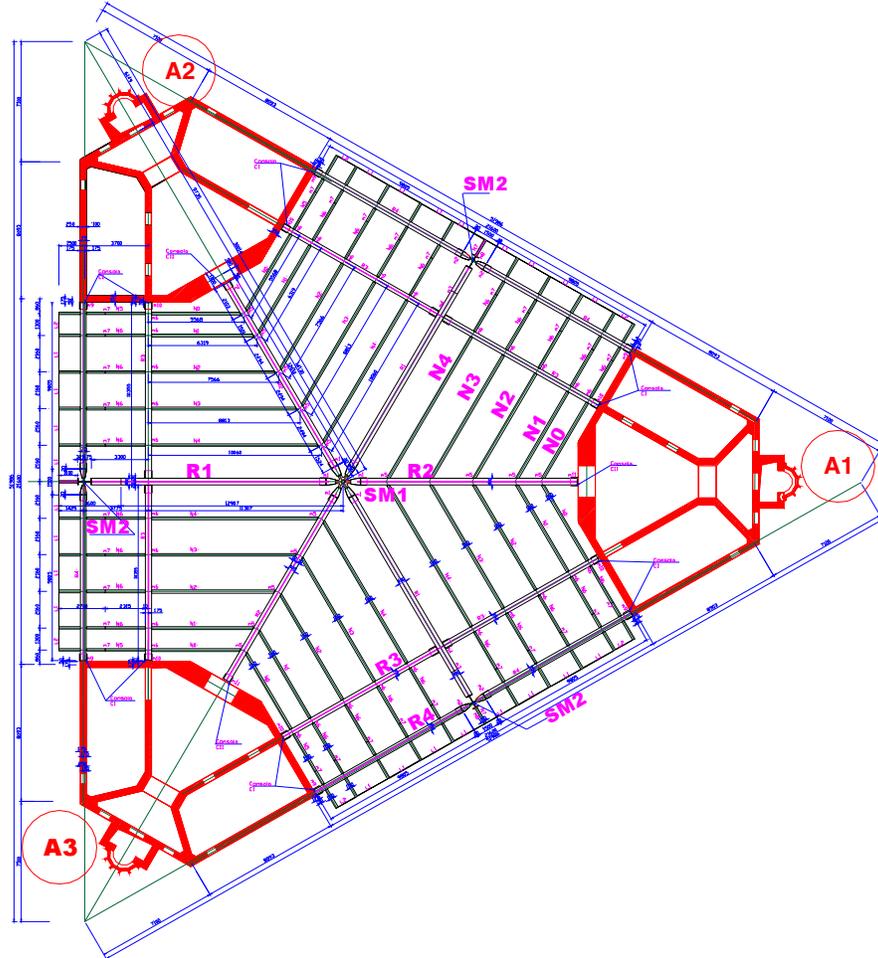


Figure 2 - Overall layout of the structural cores and floor structure of the building (+3.80 and +7.80 m)



Figure 3 – View of the three structural cores, during construction

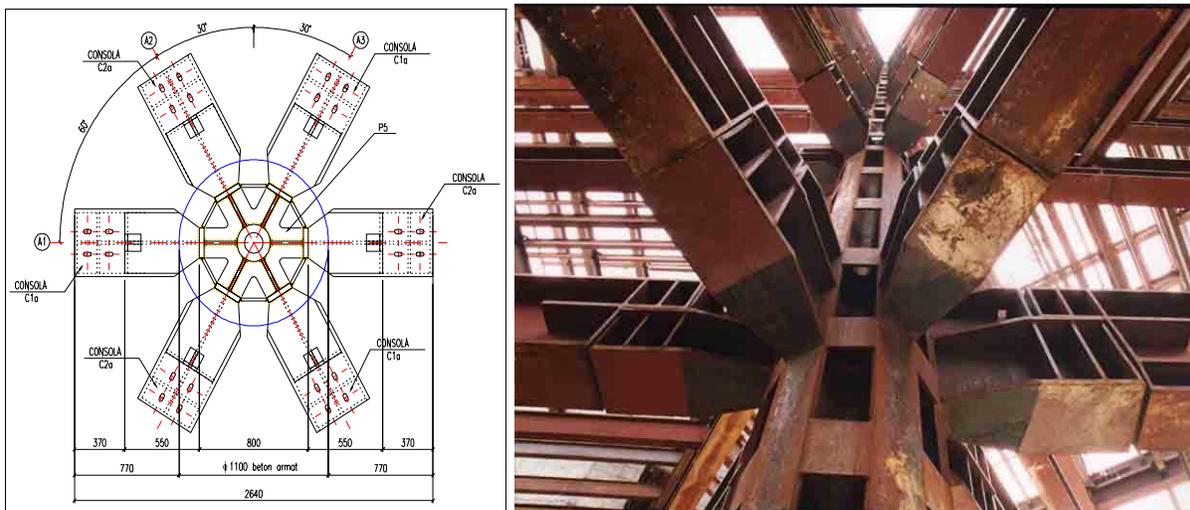


Figure 4 - Cross-section and photo of the SM1 column



Figure 5 – Photo during the construction of the steel encased concrete column SM2

4.2 The foundation structure

For establishing the foundation structure it was taken into account the possibility of integrating into the new structural system of the building (including the foundation structure) the existing general mat box, partially built, in a reliable way from the structural, service, technical and economical points of view. For solving this issue (as possibility itself and as technical solutions) the following features were considered:

- the concept and the structural analysis of the unfinished building;
- the examination on the site of the partially built foundation;
- the evaluation of the existing foundation structure reliability by preliminary computation;
- the main effects induced by the connection of the existing foundation with the structural system of the new building, as a whole;
- studies of various scenarios concerning the behavior of the structure as a whole and on component parts.

The technical assessment concerning the connection of the existing foundation with the new foundation structure led to the conclusion that the whole structural system of the building can have a favorable behavior under gravity and seismic loads.

From *foundation soil condition* point of view, a geotechnical study performed on samples taken from the site revealed that the good foundation soil, consisting of a thick layer of gravels with boulders, is relatively shallow, at a depth of 9 m. For the mat type foundation it was considered a minimum conventional pressure of the foundation soil of 600 kPa. The ground water table was reached at about 12m deep.

It resulted a unitary *foundation structure*, and more precise the existing mat box on the site and the foundations of the structural cores were bond together, as a whole, able to ensure the base fix jointing of the vertical structural elements (the columns and the structural cores).

The four steel columns of the gravitational structural subsystem are supported by the existing general mat box system. Each of the three structural reinforced concrete cores has an isolated massive foundation, adjacent to the existing mat box. The foundations were cast, without joint, in direct contact to the vertical walls of the existing mat box. The foundation of a structural core is made of two components:

- a plane concrete slab, 1.2 m thick, starting at the foundation depth of -9.20 m up to the superior level of the existing mat box (-8.00 m);
- the foundation bearing itself, made of reinforced concrete, between the depths -8.0 m and -4.40m.

The horizontal reinforcement of each foundation was done by 7 meshes of reinforcing steel, three installed on the bottom, two on the midplane and two on the top.

Concerning the *construction technology*, it must be mentioned that the foundations of the cores were built with 1.0m wide casting joints dividing the foundation in four almost equal parts. A *temporary settlement joint* (1 m wide) was prescribed towards the existing foundation. This joint was concreted after the construction of the cores and of the steel structural subsystem.

At the level -4.40 m, all the four structural units (the three core foundations and the existing mat box) were connected together by a reinforced concrete plate, 30 cm thick, thus being obtained a monolithization of the four structural units. The 30 cm thick slab on the central zone was basically “sewed” to the existing foundation by the means of the reinforcing bars, left uncovered by concrete that were lapped along the width of the new raft.

The connection of the three reinforced concrete foundations of the cores with the existing mat box led to a *strong and stiff foundation structure*.

The reinforced concrete cores are largely eccentric with respect to the foundation centers due to the following reasons:

- the general plane configuration of the building imposes the location of each core on the peaks of the equilateral triangle;
- the supplementary contact area of each structural core foundation with the soil may be ensured only towards the exterior of the three peaks of the equilateral triangle.

The floor structure over the basement was realized with monolith reinforced concrete and it assures a general “box effect” of the substructure by its connections with the curved monolith reinforced concrete walls of the substructure and with the basement zones of the three structural cores of the building.

5. GRAVITY INTERACTION EFFECTS BETWEEN THE THREE REINFORCED CONCRETE STRUCTURAL CORES

For the vertical loadings applied to the reinforced concrete structural cores, the gravity interaction phenomenon between the three cores has the following characteristics:

- as a result of the eccentricity of the core in respect with the foundation centroid, the gravity loadings applied to the core yield an overturning moment which may cause a rotation of the foundation;
- due to this rotation tendency, each core has the possibility of rotating in a vertical plane; as a result, each core has a support on the horizontal floors (Figure 6), so that between the cores and floors compression reactions are developed;
- due to the structural symmetry and the gravity loads symmetry, the cores – floors horizontal interaction forces are balanced;
- due to the cores – floors horizontal interaction forces, overturning moments are developed in respect with the bases of the cores, which are acting in opposite direction with the overturning moments developed by the gravity loads associated to the cores.

Taking into account the above aspects, the foundations of the reinforced concrete structural cores will not rotate.



Figure 6 – Photo during the construction of the steel structural subsystem

6. SOME ASPECTS CONCERNING THE STRUCTURAL ANALYSIS OF THE BUILDING

When establishing the building’s structural system, two imposed requirements associated with the architectural concept were had in mind. *The first architectural requirement* refers to the fact that on the

location there was a substructure (two basement levels and a general mat) carried out for a residence building, with the shape of an equilateral triangle in the horizontal plane. The architectural project of the new office building maintained the shape of the equilateral triangle. *The second architectural requirement* refers to the fact that, from the functional point of view, in the peaks of the equilateral triangle the sanitary facilities and vertical circulation (stairs and panoramic elevators) were stipulated.

Taking into account the seismic region where Ploiesti town is placed, the structural analysis had firstly in mind the behavior of the building to future seismic actions. In order to solve certain analysis aspects, several *structural models* were conceived, considering the elastic and post-elastic behavior of the structural elements. The modelling with finite elements and with unidimensional elements, corresponding to the following programs, was used:

- ETABS program (structural system analysis for static and seismic actions in the linear elastic range);
- ANELISE – 2D program (structural system analysis for seismic actions in the post-elastic range, similar to the DRAIN – 2D program).

6.1 The structural model of analysis for the ETABS program

The modelling of reinforced concrete cores has been made with finite elements. As a result of the existence of the window and door openings, each structural core makes a complex structure.

The idea of taking into consideration each reinforced concrete core as a cantilever beam type was kept in mind just from the stage of defining the structural concept. If the achievement of a behavior similar to that of a cantilever beam was desired, for such a complex structure type, it would be necessary to realize certain “compensations” on the opening zones, namely a special treatment for certain coupling beams. Consequently, a high number of finite elements were necessary, next to the coupling beams between the walls and for a reliable description of the window and door openings, both in the vertical and horizontal plane. Departing from this real state a pre-proportioning was performed, which established the distribution of stresses and of deformation, in order to estimate to what extent the behavior of this cantilever beam gets near to the behavior of a beam defined by the theory of strength of materials.

It was necessary to make clear these aspects, as this could be the turning point for establishing different structural models of analysis, in view of the examination of the dynamic behavior of the reinforced concrete structural core in the post-elastic range, together with the application of the reinforced concrete theory for the proportioning of the concrete cross-sections and of the reinforcement.

Within the structural model of analysis with the ETABS program, the general disposal for each story has three rows of finite elements, their total number being:

- 3,186 finite elements for the reinforced concrete structural cores;
- 108 finite elements for the columns;
- 2,379 finite elements for the beams;
- 720 finite elements for the slabs;

that is a total of 6,393 elements.

The checking upon the three laws of the theory of strength of materials was performed by analyzing the results provided by the ETABS program, concerning the stress and deformation of the finite elements, in horizontal cross-sections of the structural cores at each level. The distribution diagrams for the stresses and deformation, show that the distribution of the deformation complies with Bernoulli’s hypothesis and that the stress distribution complies with Navier’s law. Thus, was confirmed the fact that the complex structure of the reinforced concrete cores can be modeled as a vertical cantilever beam, on the one side as a beam, in order to be accepted by the structural model of analysis typical for the non-linear ANELISE – 2D program and, on the other side, as a beam to whom the proportioning methods of the reinforced concrete theory can be applied.

The structural model of analysis with finite elements (ETABS) was used for the following:

- the estimation of the modal characteristics;
- the estimation of the forces of the structural system for the seismic loading, according to the

Romanian earthquake resistant code (P100-92);

- the analysis of the effects of the gravity interaction between the three reinforced concrete structural cores.

The results obtained after performing the analysis for the reinforced concrete structural cores, by means of the finite element model, can be synthesized as follows:

- the distribution of the normal stress “ σ ” in the horizontal section of the cores is linear type, so complies with the Navier law;
- the distribution of the specific deformations in the horizontal section of the cores is linear type, so complies with the Bernoulli hypothesis.

On the basis of the above mentioned elements, the following *conclusion* can be drawn: *for the reinforced concrete structural cores a simplified model for the structural analysis as a type cantilever vertical beam can be accepted.*

The immediate outcome for accepting this simplified structural model of analysis is that it can be used further on for:

- structural system analysis for seismic actions in the post-elastic range (non-linear dynamic seismic response);
- the proportioning of the reinforcement.

Figure 7 presents the first and the third mode of vibration.

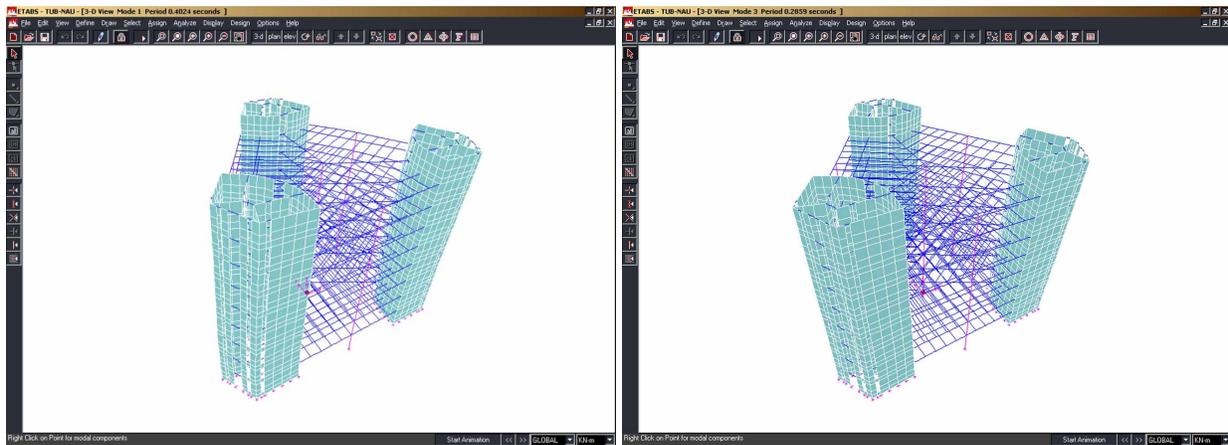


Figure 7 – Mode shapes

6.2 The conception of the earthquake design of the reinforced concrete cores

On the basis of the proportioning of the reinforcement, the locations of the post-elastic deformation zones were imposed, in view of obtaining a favorable energy dissipation mechanism. They were imposed at the bottom zone of the reinforced concrete cores (at the floor level over the basement), keeping the rest of the element within elastic range of behavior. The behavior of the coupling beams located over the door openings will be developed only in the elastic range. Concerning the coupling beams the following remarks are made:

- on the doors and windows opening zones, the coupling beams are subjected both to bending and shear force stresses;
- taking into account the dimensions of the openings and the cross-sections of the coupling beams, the shear force deformations are much more important than those resulted from bending;
- if for such a complex structure type, with door and window openings, a cantilever beam model of analysis is imposed, the compensation of the unfavorable effects of the openings will become necessary; the openings lead to additional shear force deformations and to a disturbance in the distribution of the normal stresses and of the specific deformations in the horizontal section of the structural cores;

- in consequence, the thickness of the coupling beams in the horizontal plane were increased;
- the most important coupling beam is the one placed right to the door opening between the core and the central zone; having in mind that the door opening ($h = 2.10\text{ m}$) is about 50% of the story height ($H = 4\text{ m}$), the coupling beam was increased in thickness up to 1.0 m, so that by its stiffness the unfavorable effect of the door opening may be compensated.

For the inelastic dynamic seismic analysis of the reinforced concrete structural cores the ANELISE-2D program was used. The 2-D structural model of analysis specific to this program was established having in mind the following hypotheses:

- according to the results obtained with the ETABS program, for each structural core a structural model of analysis - cantilever beam type was adopted, with two principal axes of inertia (I-I, II-II), corresponding to the actual structural system (Figure 8);
- the inelastic dynamic seismic analysis – 2D was carried out for two principal directions of the structural system represented by the three structural cores (A – A and B – B, Figure 8);
- when establishing the stiffness and strength characteristics for the 2D structural model of analysis, the effects of the 3D actual system were taken into account;
- in what concerns the control of the inelastic response of the actual structural system, it was imposed that the inelastic zones are restricted only to the bottom zone of the cores, for *any direction* of the seismic action; in a more simplified manner, for the *structural model of inelastic dynamic analysis*, the inelastic zone of deformation at the base of the structural cores was modeled as *plastic hinge*.

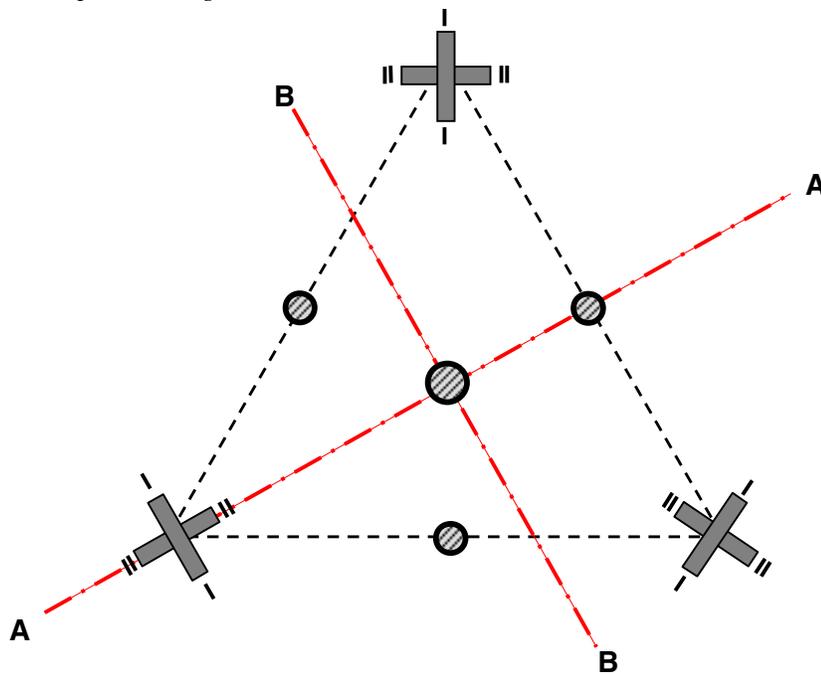


Figure 8 – The -2D structural model of analysis

By applying ANELISE – 2D, the seismic forces (bending moments, shear forces and axial forces) were established for the horizontal sections of the structural cores. In all these horizontal sections, the areas of reinforcement assure an elastic behavior, excepting for the bottom zones of the cores, where an inelastic behavior develops.

For the strength analysis of the coupling beams, the seismic forces in the vertical cross-sections were established in the following manner:

- with the ETABS program, the forces in the horizontal sections at the bottom zones of the structural cores were determined according to the Romanian seismic code (bending moments M_{CODE});
- for these bottom sections, the vertical areas of reinforcement and the yielding moment (M_Y) values were established;
- the seismic forces in the vertical cross-sections of the coupling beams (bending moments M_{CB} and shear forces Q_{CB}) were thus determined:

$$M_{CB} = \frac{M_Y}{M_{CODE}} \cdot M_{ETABS}$$

$$Q_{CB} = \frac{M_Y}{M_{CODE}} \cdot Q_{ETABS}$$

where:

M_{ETABS} = the code value for the bending moments in the coupling beams;

Q_{ETABS} = the code value for the shear forces in the coupling beams.

Figure 9 ... Figure 13 show some photos taken during the construction of the office building and Figure 14 presents an architectural overall view of the building.



Figure 9 – Steel encased column SM1



Figure 10 – View of the steel-deck floors during construction



Figure 11 – View of the steel structural subsystem



Figure 12 – Connection between the peripheral column and beams



Figure 13 – The simple supported type connection between the main steel beam R2 and the structural core



Figure 14 – Architectural view of the office building

7. FINAL REMARKS

1. The design of this particular office building in a country such as Romania, where the earthquakes are of a peculiar nature, has represented a complex process, within which new concepts about the establishment of the structural system together with the achievement of efficient technological solutions were developed.
2. The connection of the former foundation that existed on the site with the new building's foundation structure was performed by introducing four columns, intended to undertake over 40% of the building's floors weight.
3. When designing the reinforced concrete structural cores, favorable structural characteristics were chosen and imposed, in a conscious manner, in view of reducing the effects of the seismic action. Thus a favorable energy dissipation mechanism was imposed, by localizing the zones with post-elastic deformations only at the basis of the three reinforced concrete structural cores. Meanwhile, the unfavorable failure modes in the coupling beams were avoided. For the coupling beams a behavior in the elastic range was assured. These coupling beams, rigid and resistant, that do not have inelastic deformations, make that the whole multiple connect box profile act like an elastic beam on the overall height of the building, the laws of the theory of strength of materials being thus satisfied (Hooke, Bernoulli and Navier).
4. By the conception and proportioning of the three reinforced concrete structural cores an favorable "core effect" for seismic loadings was obtained; one can specify that the three reinforced concrete structural cores are the only components of the superstructure with an aseismic role, and by means of cooperation with the floor structures the elimination of the eccentricity effects given by the gravity loading transfer from the floor structures to the structural cores was assured.
5. The floor structures of the central zone, realized in steel-deck solution, may perform, in optimum conditions, the two structural roles that they have:

- the transmission of the gravity loadings to the vertical supports;
 - the transmission of the horizontal seismic loadings to the three resistant and rigid vertical reinforced concrete structural cores.
6. For each structural unit a technological solution was adopted, in order to simplify the activity on the site and to assure a high quality of all categories of works. The structural and technological advantages are reflected best in the economical ones. One of them resulted from the integration of the old existing general mat box foundation on the site in the foundation structure of the new office building. Thus a structural unit of about 3000 m³ reinforced concrete was totally used. This volume of materials comprised a direct net economy of about 2,000,000 \$. Moreover, the embedding of the old foundation in the new foundation structure led to indirect savings, by means of the effects generated by a rational structural concept.

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