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THE COLLAPSE OF NEW RC BUILDINGS DURING THE MARCH 4, 1977 VRANCEA EARTHQUAKE

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Abstract

The seismic hazard of the Romanian territory is determined by the well known Vrancea seismic region, characterized by a high rate of large earthquake occurrence in a narrow focal volume confined to an area of about 30 x 70 sq kilometers. During the last century, four major Vrancea earthquakes happened (1940, 1977, 1986, 1990), the second one leading to a disastrous impact on the Romanian territory. 36 buildings collapsed, 4 among them being new buildings at that time, designed according to the seismic legislation in force. This paper summarizes the behavior of these four RC buildings during this earthquake, showing their main structural deficiencies that contributed to their collapse. All these RC buildings were designed and constructed prior to having a recording of a strong earthquake in Romania and prior to the introduction of provisions for ductile response in seismic design codes (nonductile concrete buildings). After this seismic event, the need for improvement in collapse assessment technology for existing nonductile concrete buildings has been recognized as a high-priority. Once more, it was demonstrated that the failure of such buildings involves partial or complete collapse, together with substantial life and property losses. The most instructive building collapse was that of a new computing center, built in 1967. It was a 3 storey building, comprising a central structure with service towers at both ends, structurally separated from the main building that collapsed, while the service towers did not. The second of the four buildings was a residential block of flats whose structural system was RC cast-in-place structural walls. It was designed in 1972 and completed in 1974. The block consisted of six sections, developed over a length of 225 m, separated by small joints. The height regime of the building consisted of a basement, a ground floor and ten stories. During the 1977 earthquake the last section of the block completely collapsed by cross overturning, rotating relative to the ground structural floor. The third building that makes subject of the paper is also a block of flats that was completed in the period 1961-1962. The height regime of the building consisted of a basement, a ground floor, eight complete stories and a partially retracted top floor. Along its length, the building was divided in three sections. Its structural system consisted in a flexible first storey with RC columns and a more rigid system with cast-in-place RC structural walls for the rest of the building. During the March 4, 1977 earthquake the west part of the building, on a length of about 10 meters, was dislocated, its subsequent demolition being required. The last case refers to a building with mixed structural system, consisting in moment resisting frames and masonry walls. During the earthquake the ground floor – a weak and soft storey – "*disappeared*". All of these case studies will be presented in such a manner, so that everyone can understand what happened.

Keywords: Vrancea; structural failure; collapse; assessment; weak and soft storey

1. Introduction

The subject of this scientific communication seems, at the first sight, a topic that has been treated by many other authors after the incidence of the March 4, 1977 earthquake. It is true and I quote among others: Ambraseys (1977), Ambraseys and Despeyroux (1978), Berg (1977), Berg, Bolt, Sozen and Rojahn (1980), Berg (1980), Fintel (1977), Fattal, Simiu and Culver (1977), Tescan, Yerlici and Durgunoğlu (1977 and 1978), Japan International Cooperation Agency (1977). In 1982 a monograph of this earthquake was published in Romanian language [1]. As time passed, but mainly after 1989, on the Romanian market "*hidden materials*" started to appear: earthquake photos, papers and reports of foreign professionals, and reports prepared by leading design engineers and academics for the Public Ministry and for the Directorate of Romania Safety.

Compared to the seismic disasters produced globally over the past 40 years, the four examples of building collapses that are the subject of this paper are, obviously, less spectacular. However they are extremely important and even topical for Romania, where a future strong earthquake is expected. There are hundreds, or even thousands, similar buildings which were not strengthened after the 1977 seismic event and which exhibit today a high degree of seismic vulnerability, anticipating a great disaster.

The structural methods of analysis in the linear elastic range available at that time (stipulated in the technical legislation in force), even though they were used in the current design practice, could not explain the damage phenomena, the partial collapse, as well as the total collapse of buildings. Moreover, they did not allow the identification of the technical reasons that caused the collapse of a building during an earthquake. In other words, the papers and the reports written on the March 4, 1977 earthquake by different Romanian and foreign professionals presented some "*first hand observations*" related to the performance of various types of reinforced concrete buildings in Bucharest.

Since 1967, in Romania new design methods began to be used, within which the structural analysis to dynamic loads generated by seismic actions in the inelastic range was an important step. The prominent Romanian engineers have realized that the interpretation of the behavior of a structural system based on its deformations in the inelastic range, together with the understanding of the nature of the dynamic phenomena, were the only "*concepts*" that could explain the real behavior of a construction at the incidence of an earthquake. The design engineers also became conscious that while using this new procedure they would have the possibility to determine the technical causes of damage and of partial or total collapse.

After the March 4, 1977 earthquake, some of them began to apply the concepts previously mentioned in the process of the technical assessments of a large number of damaged buildings, to establish the technical causes which led to their damage, and to use them in drawing up solutions for their strengthening. Thus, to the scientific basis of these structural methods of analysis a key issue was also added, namely testing them in real cases.

2. The collapse of the "Computing Center" building

The collapse of the central part of the building "Computing Center" of the "Transport and Telecommunications Ministry", produced by the March 4, 1977 earthquake, was the subject of several technical assessments. These were drafted by well-known personalities of the period at the request of various authorities and served as support materials for various foreign specialists who came at that time in Romania. Their elaborators have not reached the same conclusions regarding the "*technical causes*" that led to the collapse and they could never explain (or they did not want to) the "*phenomena that led to the simultaneous failure of the building columns*". Therefore, the higher authorities of the State ordered then another technical assessment, and some of the elements that are presented in this section are an accurate summary of the information which is currently available [2], [3].

This building was designed in 1967, was constructed between 1967 and 1968 and had a built area equal to 1,230 m^2. It consisted of three bodies separated by expansion joints (a central technological body where the computer units were located, an office and sanitary facilities body on the left, namely a body where the main staircase was placed on the right). The height regime of the building consisted of a ground level (H = 6 m) and two levels (H = 5 m). Since the collapse occurred only at the technological central body, the other two lateral bodies being practically undamaged, in what follows we will refer exclusively to the affected one (Fig.1).

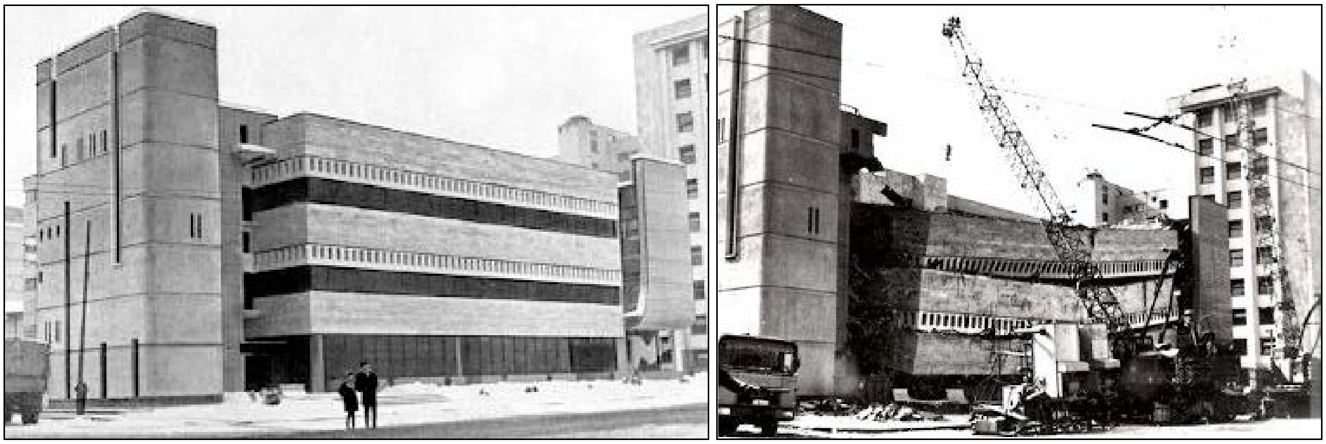
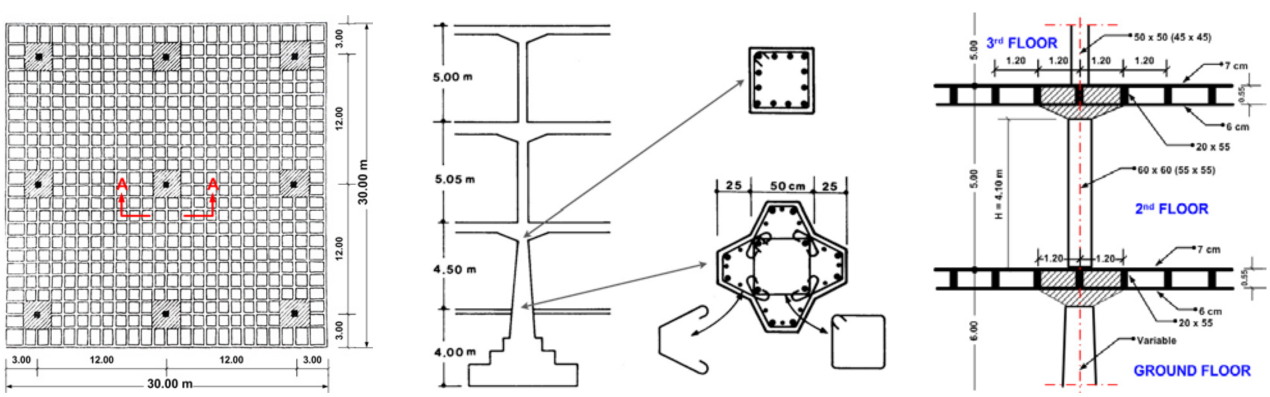


Fig. 1 – The "Computing Center" building before and after the March 4, 1977 earthquake (INCERC)

2.1 The structural system of the central body building

The central body had a square shape (30 m x 30 m), with two openings of 12 m, and two marginal cantilevers, each equal to 3 m. The structural system of the building consisted of 9 columns at each storey and cellular floors with cast-in-place reinforced concrete capitals. The nine columns of a level were set after a modular configuration equally sized (12 m x 12 m): a central column S_1, four intermediate marginal columns S_2 and four corner ones S_3 (Fig.2,a).

At the ground floor the columns were designed with variable sections, a square shape under the capital (S_1: 60 cm x 60 cm, S_2 and S_3: 50 cm x 50 cm) which widened toward its base, to a 1.0 m x 1.0 m cross shape. At the two upper storeys the columns were designed with a constant square section (Fig.2,b and Fig.2,c).



(a) (b) (c) (d)

Fig. 2 – Floor structure, sections and typical column reinforcement (adapted from [1] and [4])

The horizontal component of the structural system consisted of cast-in-place RC cellular floor structures that rested on the columns by means of RC capitals. The cellular floor beams were spaced 1.20 m center to center in both directions, and had the following cross sections: the first two floors 20 cm x 55 cm, and the last one 20 cm x 50 cm. These beams were tied-in by cast-in-place reinforced concrete plates with a thickness of 7 cm at their top and of 6 cm at their bottom. The floor structures rested upon the columns by means of massive capitals, developed on a plane surface equal to 2.40 m x 2.40 m, and having a height of 1.0 m, in order to complete the frame joints (Fig.2,d). The floor cantilevers (of about 3 m) supported exterior precast RC vertical elements at each level, making continuous bands of sash separating the precast wall units (Fig.2,a). The ground story was enclosed by walls set in from the exterior building lines and capped with a continuous band of sash.

As it can be noticed in Fig.2,c, the ground floor columns were reinforced in an unusual manner. A typical ground-story column had 12 ∅25 mm round bars, that have been stopped at short distance from the full heights of the columns, in addition to 12 ∅20 mm round bars that extended from the base up to about 2/3 of its height. 4 of the 12 full-height bars were enclosed in square hoops all the way; the remaining bars were outside the square ties except at the very top of the ground story, and were restrained by hairpin bars serving as auxiliary ties (∅8 mm round bars). The tie sets were spaced at 15 cm in the bottom 1.25 m of each story and at 20 cm for the rest of the free height [5].

2.2 The mechanism of the building collapse

The mechanism of collapse includes the contributions of the following key factors:

- the degree of severity of the seismic motion and its directivity reflected in the path of the cracks in columns;
- the formation of plastic hinges in columns;
- the combined breaking mechanism brought by the insufficient ductility and the inclined cracks in all columns of the building;
- the increase of the energetic absorption processes of the energy induced by the seismic motion at the 2^{nd} and 3^{rd} floors of the building, due to an strength overcapacity of the ground level; this phenomenon was initially identified based on the comparison of the shear force diagram corresponding to the fundamental eigenmode of vibration with the diagram of capable level shear force, corresponding to plastic moments of the columns (the shapes of the two diagrams were not proportional); it was found out that the strength coefficient of the ground floor columns to seismic actions (at the yielding limit) was higher by 25%, compared to the one of the upper floor columns; the larger strength of the ground level to seismic actions was only apparently an advantage, in reality this was a factor with negative contribution;
- from the dynamic point of view, the coupling of the main body building (stiffer and more resistant) to the adjoining left body building (more flexible and less resistant and which didn't collapse) disadvantaged the main body building and favored the connected one.

Considering the above mentioned aspects, the collapse mechanism of the "Computing Center" building can be outlined as follows:

- in the phase preceding the first main seismic shock, the inclined crack formation in columns was initiated (especially in columns characterized by ratios $\tau_0/R_t > 1.0$; τ_0 = average shear unit stress and R_t = strength concrete in tension);
- during the main shocks, the displacements of the floors in the inelastic range (primarily those of the second floor) were more pronounced than those of the ground level; it is therefore probable that the collapse started at the 2^{nd} floor, may have continued at the 3^{rd} floor and, gradually, breaking mechanisms in areas with inelastic behavior of the ground floor columns were developed.

The collapse of the central body of the building occurred by column breaking at its three levels, followed by a gradual collapse of the floor structures. As it was found on site and by studying the available photos, it resulted that the columns of the three stories were broken, in the most cases, at about 50 cm below the capitals. At the ground floor there were columns that were broken at different heights and at the other two stories there were columns broken at their bases. The column breaking was produced by the crushing of the entire concrete section on variable lengths, but there were also present breaks in inclined sections on the direction of the dominant action of the seismic motion. The phenomenon of destruction of concrete in columns was more present at the 2^{nd} floor, as a result that the central column was destroyed at its base. In the breaking areas, the longitudinal reinforcement was strongly buckled, went out completely from the sections and was twisted as a result of a torsion phenomenon produced at the floor levels. The column heights were reduced with about 0.5 …1.0 m, fact that led to the collapse of the floors which didn't break in motion, but were strongly deformed. During their falling they found new bearings, i.e. the broken columns and the exterior reinforced concrete elements (Fig.3).

Fig. 3 – Ground story top and bottom column failures (EERI collection)

3. The partial collapse of residential buildings

On March 4, 1977, a huge number of residential buildings – blocks of flats type – existed in Bucharest and in many other towns. For the most part of them, the heights ranged from 9 to 15 stories. It can be stated that the behavior of the new residential buildings (built between 1950 and 1977) was good. Nevertheless, there were two exceptions: the marginal section of a block with structural walls built up to the foundation (named OD16) and the marginal section of a block with structural walls and soft story (named Lizeanu). These two cases are the subject of the following two subparagraphs.

3.1 The "OD16" block of flats [6]

The OD16 block of flats comprises six sections (A…F), separated each other by small joints and deployed on a total length of 225 m (Fig.4).

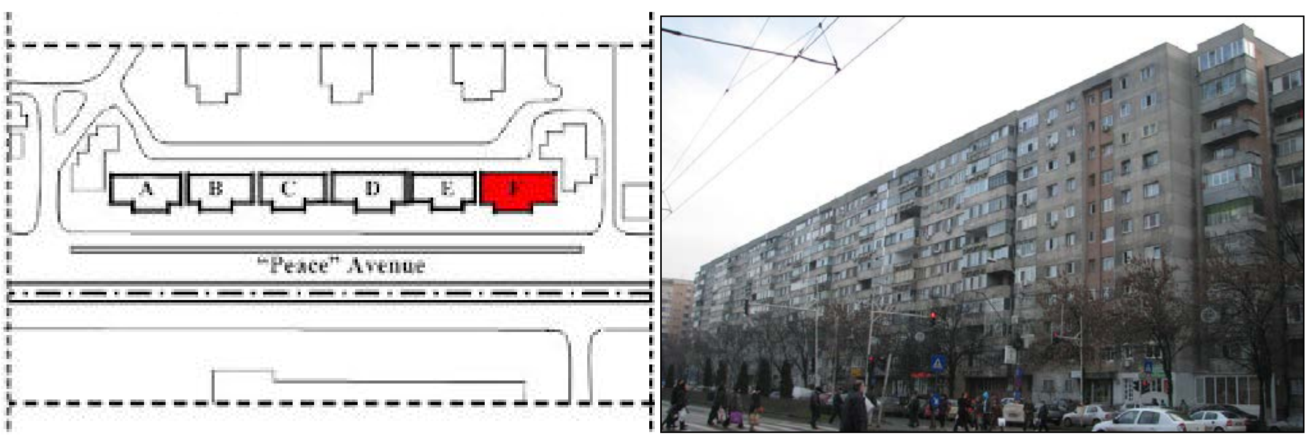


Fig. 4 – The six sections of the "OD16" block of flats

At each end of this building there are other blocks of flats, having different shapes and sizes. Each section of the block, with a rectangular shape, has the following in plane dimensions: 27.47 m x 11.34 m. The block consists of a technical basement, ground floor and 10 stories, with a total height of 31.00 m above ground level. From the point of view of the architectural disposition, there are four apartments on each level, so each section has 44 apartments.

The structural system of each building section is of structural walls type: an assembly of reinforced concrete structural walls with thickness equal to 14 cm and bulbs on their extremities. It consists of a longitudinal wall located along the central axis of the section and a series of structural walls disposed on the transversal direction, those in extremities without openings, and those inside with openings for doors and windows, disposed shifted in plane. The floors are made of cast in place reinforced concrete, have a thickness of 12 cm, being embedded in the interior structural walls and having provided an "*elastic support*" on the facade beam, at the exterior. The building has cantilevered balconies on both fronts, which were cast in place together with the floors.

During the March 4, 1977 earthquake, the last section of the block (F) completely collapsed by transversal overturn towards the middle of the boulevard, rotating relatively to the ground floor (Fig.5).

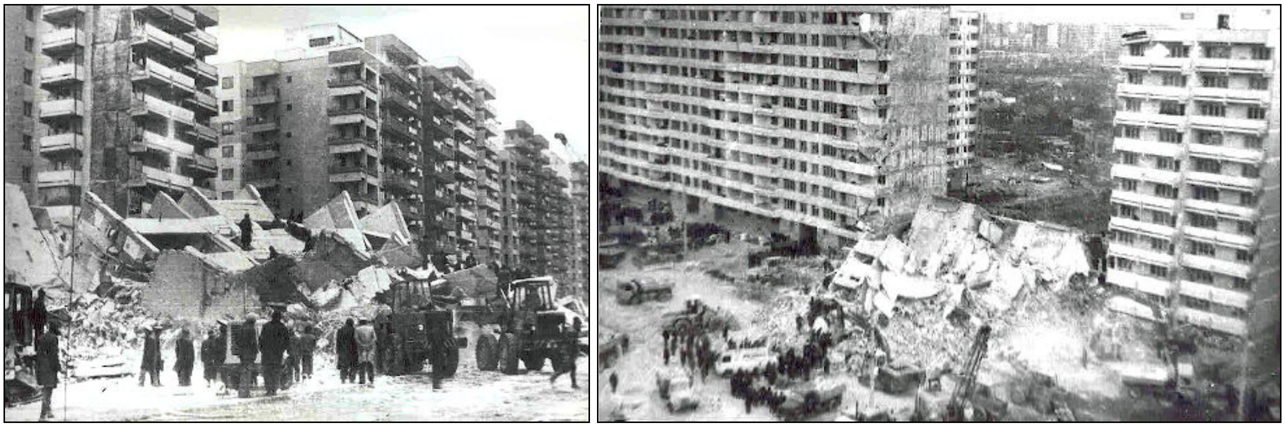


Fig. 5 – Images of the section "F" that collapsed (AGERPRES)

The cause of the collapse was the destruction of the strength capacities of the ground floor RC structural walls which were positioned on the transversal direction of the section to gravity loads. This mode of failure defines the structural collapse form of "*overturning*" type (Fig.6). The phenomenon of "*overturning*" was highlighted by two elements: the first one was the fact that the upper part of the block has reached the middle of the boulevard on the tram rails, and the second one consisted of a proof of rotation recorded on site, by visible traces in the form of circular arcs (scratches) left on the joint separation wall of the "E" section by some existing irregular growths on the unfinished wall of the section "F" that collapsed.

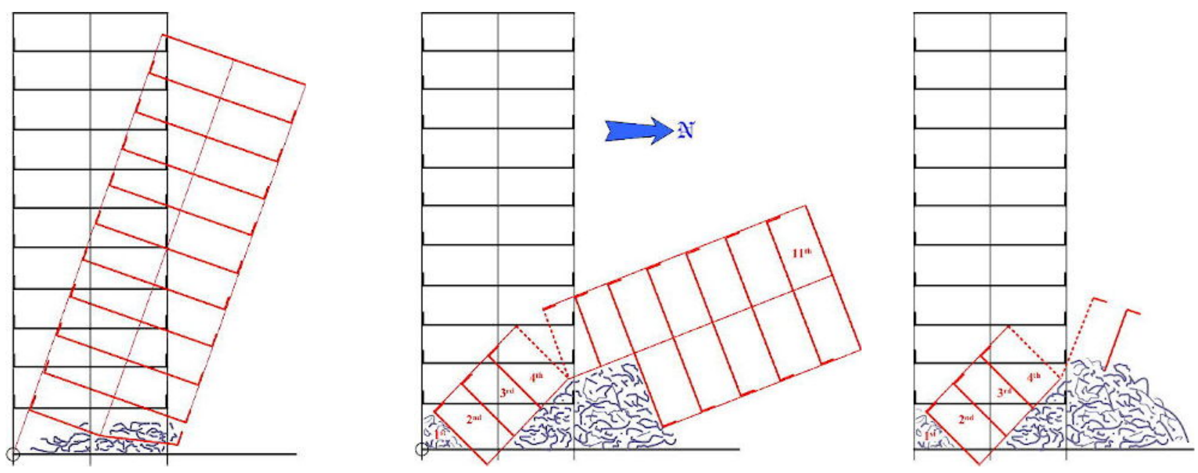


Fig. 6 – The "overturning type" collapse mechanism

It can be said that the overturning always occurs under the action of gravity loads. The action of an earthquake on the structural system of a building takes place until its vital structural elements are destroyed. Then, the structural system can no longer sustain gravity loads and collapses. In the case of section "F" of the block OD16, the seismic action caused the destruction of the compressed areas at the basis of the structural walls located at its ground floor and disposed on the transversal direction (a process of complex breaking, generated by the combined action of the interior generalized forces M+N+T, which caused crashing, shearing and detaching of the concrete, together with the buckling of the longitudinal steel bars of the reinforcement). Under the action of the large forces of compression due to gravity loads, whose values were increased by the seismic action, begun the destruction of the bulbs of the structural walls. The poor transversal reinforcement of the bulbs led to concrete crashing and, finally, to their destruction.

The structural failure of this section was also favored by other causes, of which the followings can be mentioned:

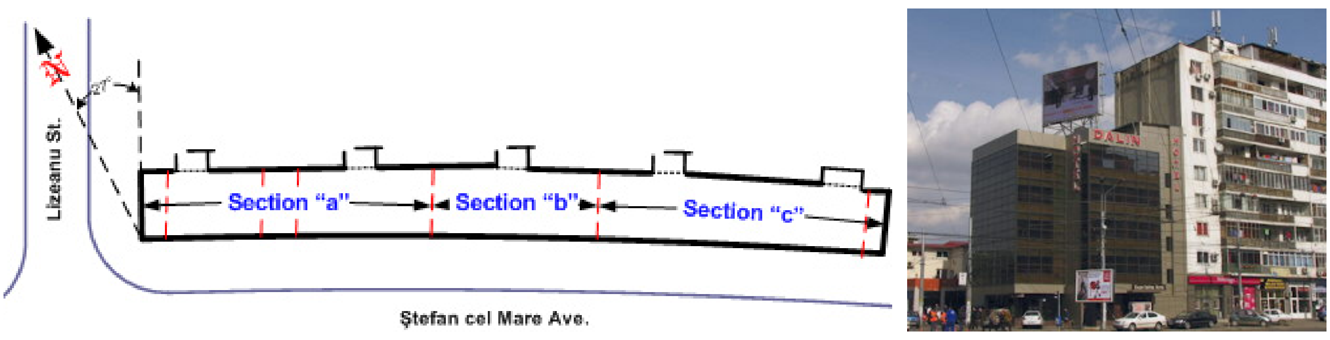
- a *deficient structural conception* which led to a high sensitivity of the building to strong seismic actions (the existence of a single longitudinal structural wall, the shifting of some transversal structural walls, reduced cross sections of the structural walls and, especially, those of the bulbs in the extremities);
- the *lack of ductility* of the structural walls as a result of their insufficient general reinforcement;
- inadequate transversal reinforcement of the structural walls, which prevented the taking over of the main interior generalized forces;
- execution deficiencies (areas with segregated concrete, relatively frequent casting joints and improperly executed, hidden defects, poor working technologies).

At this block of flats failures even to non-collapsed sections have been found, mainly in section “D”. Damages were of brittle type fractures at the base of the structural walls and mainly in the bulbs at their extremities, with concrete crashing and peeling, rebar buckling, fissures and cracks with different configurations. Practically, section “D” was by itself on the verge of total collapse, being saved by the prompt intervention of the authorities who decided to make adequate supports, which proved to be effective.

3.2 The “Lizeanu” block of flats

The block of flats located in Bucharest, 33 “Ştefan cel Mare” Avenue (corner with Lizeanu Street) was the subject of several studies and technical assessments after the March 4, 1977 earthquake. The building was built during 1961-1962, a period in which there were no official design rules to seismic actions [1].

The building has a long linear shape type bar, slightly curved, following the curvature of the “Ştefan cel Mare’ Ave. Its dimensions were: length equal to 117.55 m (orientated towards NW-SE); width equal to 10.85 m; height of about 30 m from the sidewalk elevation. The height regime of the building comprised a technical basement, a ground floor, eight complete floors and a retracted last floor, as it can be seen in Fig.7,b. The total surface of the building was equal to 12,400 m^2. Along its lengths the building was divided into three sections – a, b, c – separated by joints and included five staircases (Fig.7,a). The section “a” of interest for this paper, located towards the “Lizeanu” St., had a greater length and two access stairs. Between them there was a crossing transverse passage with two openings of 3.0 m each. The ground floor was occupied by department stores and the floors were designed for dwellings (a total of 168 apartments).



(a) (b)

Fig. 7 – The “Lizeanu” building

The structural system of the building consists of a flexible ground floor with reinforced concrete columns and a more stiff system above it, with reinforced concrete cast in place structural walls, cellular type. Essentially, the structural system consisted of:

- plain concrete continuous strip footings (ingoing into the foundation medium up to depths of 4.30…4.75 m) and reinforced concrete bearings; isolated foundations for the median columns;
- reinforced concrete columns from the foundations up to the 9th floor, except the columns on the inside median row stopping at the 2nd floor, where they were replaced with structural walls;
- reinforced concrete structural walls on the entire boundary between the exterior columns (40 cm thick) and reinforced concrete structural walls that were placed in-between two interior columns (30 cm thick), at the basement;
- reinforced concrete discontinuous structural walls with a thickness of 15 cm at the 2nd and 3rd floors, connecting two to three columns on the transversal direction and the median row of columns on the longitudinal direction;
- reinforced concrete structural walls with a thickness of 15 cm at the floors 4…9, where there were no columns;
- reinforced concrete floor structures with beams and slabs of 12 cm thickness, at the ground and 1st floors, and with thick slabs (14-16 cm) without beams at the remaining floors;
- reinforced concrete stairs and elevator cages.

During the March 4, 1977 earthquake, the western end of the section “a” has been dislocated over a length of about 10 m, area which was later demolished and where a new hotel was built (Fig.7,b). The area destroyed by the earthquake had about 12% of the total deployed area (respectively about 1,550 m^2 of the total 12,400 m^2).

To identify the real causes of the collapse of this part of the building (Fig.8), a detailed technical assessment was conducted and its main findings are presented in the followings.

a) Due to the characteristics of the structural system, the inelastic deformation and the energy absorption processes induced by the seismic motion were concentrated at the ground floor (a general phenomenon for tall buildings with ground flexible floors); this reality would have imposed increased ductility requirements for this level compared to buildings to which the earthquake-induced energy distribution could have been absorbed through inelastic deformation at multiple levels.
b) The majority of the ground floor columns behaved as if they were fix-ended in the basement structural walls, as well as in the structural walls which started from the 2nd floor upwards, so that plastic hinges formation trend materialized at the extremities of these columns; thus it resulted that during the earthquake the behavior of the building depended on the ductility characteristics and on the resistance to shear of the ground floor columns.
c) The loadings of the columns, even so quite large due to gravity loads, were amplified by the severe earthquake action exerted by the structural walls of the upper floors under the unfavorable influence of the direction of the main motion shocks (that acted more powerful towards the end of the Western section “a”); this led to an increase of the compression unit effort “σ_0” in its columns, compared to the concrete compressive strength “R_c”, respectively to an increase of the ratio “σ_0/R_c”, thus their capacity of ductility being reduced (it is known that the columns ductility is less than that of the beams); consequently, more columns have shown a higher risk of failure by crashing and by expulsion of the compressed concrete in the plastic hinge areas.
d) In some of the columns a greater yielding sensitivity in inclined sections was also emphasized, due to a more reduced shear resistance.

Given the above mentioned aspects, the partial collapse of the section “a” of the “Lizeanu” building could be explained by the failure of some columns at its ground floor where the highest loads were focused. This was caused by a complex mechanism of rupture (bending moment with shear force), phenomenon that was triggered by the low ductility cross-sections of the columns and favored by an insufficient resistance to shear forces.

Following the failure of these columns, other columns were driven in the breaking process, causing the dislocation and the tilting towards South of the destroyed area.

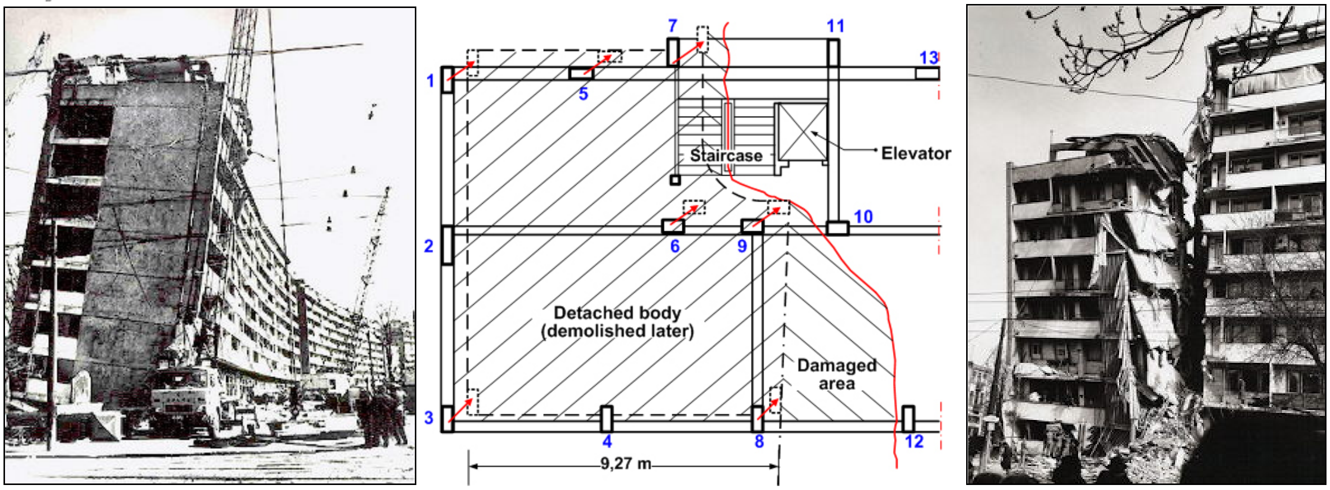


Fig. 8 – The “Lizeanu” building partial collapse photos and the detached body [1]

In the non-collapsed remaining building, damage was found to the ground floor reinforced concrete columns, especially in the staircases 2 and 3. The damage was of concrete crushing and expulsion type accompanied by the buckling of the reinforcement, or shearing of the reinforced concrete sections type, following inclined planes at about 65° from the vertical, which were located at the base and at the upper ends of the ground floor columns.

3.3 The “Valea Călugărească” block of flats

It was a common practice in Romania to have buildings with open first-floor areas, by using columns to support the upper floors. In some cases the spaces between the columns were filled-in with plat-glass windows in order to create ground-floor shops.

The “Valea Călugărească” building was such a construction, with three residential floors and department stores at the ground-floor [1].

Out of functional reasons, the height of the ground floor (3.60 m) was greater than the height of the current floors (which usually was 2.75 m). The building had two staircases with four apartments at each floor, i.e. a total of 24 apartments. The apartments were partitioned and separated from each other by plain masonry panel walls, disposed on the two main axes. At the ground floor there were far less walls which were placed on the rear facade, while on the main facade the exterior walls were of glass type.

The structural system of the building was composed of two subsystems: a main subsystem – reinforced concrete moment resisting frame type – after its two main axes, and a secondary subsystem type – infill panel walls of simple brick masonry. The building floors were built of reinforced concrete slabs with a thickness of 12 cm, using the cast in place solution.

The stiffness and the strength characteristics of this building were imposed by the two already mentioned structural subsystems that responded to the earthquake action. The architectural solution with commercial spaces at the ground floor led to a much higher degree of flexibility for the ground floor moment resisting frames without framed masonry, compared to the degree of flexibility of the reinforced concrete moment resisting frames with framed masonry (located at the three residential floors).

The difference between *the active sections* of the walls of brick panels disposed on both main directions of the building at the upper floors and the *active sections* of the ground floor walls (especially of those used for closing) led to the situation of a relative stiffness to lateral displacements of the ground level much lower compared to the relative stiffness to lateral displacements of the residential floors. The total areas of the horizontal sections of the walls of the upper levels were much higher than those of the ground floor, both on longitudinal and transversal directions. This led to a very weak “*global stiffness*” to lateral displacements of the ground floor masonry walls compared to the “*global stiffness*” of the masonry walls of the upper floors. At the same time, the strength capacities to seismic forces of the columns of the ground floor were significantly lower compared to the strength capacities of the columns of the building floors, due to their greater height. As a result of these stiffness and strength characteristics obtained during the architectural and structural design process, it was arrived to a "*soft and weak*" ground floor situation.

In relation to its main axes, the block was driven into motion under an inclined path, predominantly transversal, resulting thus a displacement of the building following this direction.

During the seismic motion, the ground floor of the "Valea Călugărească" block completely vanished, the upper three-story ensemble "*vertically fell*" practically without serious damage, and the second floor structure reached the ground (Fig.9).

Fig. 9 – The "Valea Călugărească" building with its "vanished" ground floor (ICCPDC-INCERC and JICA)

Basically, the ground floor yielding started from the bottom up, and stopped at the floor above it. The ground floor columns necessarily yielded at their bases (at the finished floor level, ± 0.00 m). At their top, the yielding produced as a result of the fact that the frame joints composed of the second floor columns and the beams of the structural system at the ground floor were much stronger than the ties of these joints with the ground floor columns.

In other words, the 2nd floor building columns together with the beams of the second floor structure, taking into account the favorable effects given by the strength and the stiffness of the 2nd floor brick masonry walls, formed a much stiffer and significantly more resistant storey, in comparison with the ground floor. In these circumstances, only the ground floor columns were deformed in the inelastic range, the structural system members above them remaining undeformed. In the plastic hinges formed at the ends of the ground floor columns yielding moments have developed, which resulted in a value of a shear force at the ground floor much higher than the shear forces of the upper building floors.

It follows that the plastic hinges of the ground floor columns were subjected to deformations produced by both bending moments and the associated shear forces, thus leading to the exterior columns breakage in inclined sections at their upper extremities. These columns, with their vertical reinforcement pulled apart in the lateral and the stirrups in the damaged area cut through or unfastened, were pushed outwards during the vertical fall of the compact ensemble consisting of the three upper floors. Without the support of the exterior columns, a compression overstress of the interior columns generated by the shock of the vertical falling of the three floors package was reached, having as consequence their total destruction. Some of them penetrated the floor structure above the ground floor, entering in the space of the former second floor (Fig.10).

Thus the ground floor vanished; the ensemble of the three floors fell vertically in a compact manner, without them being severely damaged. The brick masonry walls of the ground floor (few in number and mainly arranged on the rear facade) were completely destroyed. The masonry panels of the three upper floors remained mostly attached to the concrete members of the structural system. Damage was also noticed in some areas of the two staircases of the building.

(a) (b)

Fig. 10 – The “Valea Călugărească” building. (a) Rear facade column pushed outwards.
(b) Interior column that penetrated the floor structure and reached the 2nd floor [ICCPDC-INCERC].

The main causes of the serious damage of the “Valea Călugărească” block can be summarized as follows: (a) the design process of the building (1969) that was performed according to the code P.13-63, which later proved to be without any connection to the Vrancea earthquake characteristics; (b) the poor compliance to seismic actions which led to the situation of a building with “*soft and weak first story*”; (c) execution deficiencies.

4. Some final conclusions

During the March 4, 1977 earthquake the so called modern reinforced concrete buildings of various types, although not conforming to contemporary ductile design standards, was satisfactory, excepting those with major design and execution deficiencies. Structural damage observed in many other monolithic structures could have been avoided to a large extent if proper standards of detailing had been followed. In the column failures, objectionably wide spacing and low strength of the transversal reinforcement were apparent. Sudden reduction in stiffness in the lower stories of multistory buildings was a source of stress concentration [7].

All the papers and the technical reports prepared on the basis of structural analysis in the linear elastic range and taking into account only “*first hand observations*” (personally made or communicated by others) could not scientifically establish the technical causes of the collapse of the previously mentioned buildings [8]. Comprehensive technical assessments could not be carried out after the earthquake, as authorities have ordered the urgent demolition and the removal of the debris from the damaged areas, thus producing significant looses of informative materials.

The present paper *takes a step forward*, by identifying the factors imposed by the seismic motion, and those associated to a behavior in the inelastic range of the studied buildings, as well. The negative technical characteristics of the collapsed buildings were consequence of the insufficient knowledge that existed during their design period, of the low level of the legislation in force and of the lack of appropriate design prescriptions. The collapse of these new buildings during the March 4, 1977 earthquake, as well as the serious damage of many others, highlighted the need for clear and practical effective measures to be included in the technical legislation related to the design process. Glen V. Berg studying the collapse of the “Computing Center” building stated that ‘*in a sense, it was an extreme example of the soft ground story – a source of much grief in one earthquake after another*”. He also remarked “*the lack of a second line of defense; once a column failed there was no other structural component to come to the rescue*”. His final conclusion was the following: “*we ought to consider now* *seriously our building code in the light of this disaster*" [5]. The collapse of the section "F" of the block OD16 should be considered in the context of the serious damage of other sections of similar blocks, which required extensive strengthening works. This reality highlighted the sensitivity of these blocks to strong seismic actions, as a result to insufficient resistant capacities on one hand, and to an unfavorable inelastic behavior, on the other hand. The collapse of the other two block of flats presented in the paper drew the attention on the unfavorable behavior of buildings with *soft and weak* ground story. Each case study presented had the features of a "*unique*" technical accident that should have been followed by immediate measures of strengthening similar buildings.

Unfortunately, the change of political regime that took place in December 1989 gave the start to a new "*fashion*": the architectural remodeling of the apartments in the block of flats, especially in large cities. "*Freedom*" was misunderstood and most of the owners of the apartments in blocks similar to those that collapsed in 1977, began to demolish the walls considered in design without "*structural role*" in order to get more "*generous space*", without understanding that in fact they removed the building's first line of defense against strong earthquakes. The "*fever*" of architectural changes for aesthetical reasons also comprised other categories of buildings, whose original destination has been changed to discotheques, clubs, restaurants, bars, fitness centers etc. The owners of these apartments and buildings had only the profit in mind. In some cases they requested approvals for the changes they intended to make, and they received them immediately with great ease. The design engineers, officially certified as technical experts, made superficial technical assessments and, under the motivation "*partition walls without structural role*", have admitted too easily to sign documents allowing the owners to demolish such walls. What these technical experts did not know, and were not even interested to find out, was the fact that, in the same block of flats, dozens of such walls may have been demolished. None of them made adequate structural analysis and, with a reprehensible easiness, gave approvals for architectural changes. Another aspect that seems to be unknown, and the officials seem not to be interested in, is the amount of such demolitions which were extended even to structural elements, especially when the architectural changes were conducted without approvals. It is the time and place to tell the truth, namely that these buildings have survived during the 1977 earthquake thanks to the "*sacrifice*" of the nonstructural components. At present the vulnerability of the residential reinforced concrete buildings is extremely high to future strong seismic motions, despite the fact that the March 4, 1977 seismic event was for professionals a very "*informative*" earthquake.

At the end of this paper I would like recall the words of Professor Nicholas Ambraseys in a letter addressed to the participant at the 14WCEE, held in Ohrid, Macedonia: "*We realized that earthquakes do not kill people, but our structures do, and that acts of God of today are often tomorrow's acts of criminal negligence*".

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