Zeitschrift: IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

Band: 30 (1978)

Artikel: Nonlinear seismic behaviour of Magurele building - Budapest; during the earthquake of March 4, 1977

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DOI: https://doi.org/10.5169/seals-24184

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NONLINEAR SEISMIC BEHAVIOUR OF MAGURELE BUILDING - BUCHAREST, DURING THE EARTHQUAKE OF MARCH 4, 1977

Boris Simeonov *

SUMMARY

During the March 4,1977 earthquake in Romania,the office building IFIN-Magurele underwent moderate damage.For the purpose of definition of a solution for strengthening of the building an elastic and inelastic response analysis was carried out.

The analytical results have shown a good correlation with the real behaviour of the building during the earthquake.

From the analysis carried out a conclusion was made on which elements should be strengthened and their ductility improved.

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1. INTRODUCTION

On March 4,1977 the territory of Romania was subjected,for about 100 seconds,to a severe earthquake with an origin at Vrancea on the arch of Carpati,between the 45th and 46th parallel.The intensity of this earthquake can be compared to the most severe Moldavian earthquakes during the last 500 years(1471,1620,1802.. 1940,1977),and the magnitude was estimated as M=7.2.

Although the earthquake epicenter was 170 km to the north of Bucharest,the city suffered extensive damage.The Office building IFIN-Magurele,a twelve story building,located approximately 10 km to the south of Bucharest was selected for an extensive investigation between IZIIS-Skopje and I.P.Carpati-Bucharest.In fact, this building consists of RC structural walls and during the earthquake suffered moderate damage.

The objective of this joint study was to identify the characteristics of the strength,deformation and the energy processes for such type of structures as a general property of the system,and through this to make suggestions for the most rational measures for repair.

As a result of the joint study,a report was published [1] ,which brought conclusions for the general behaviour of the structure and decisions on the specific aspects of repair.

2. STRUCTURAL SYSTEM AND DESIGN CRITERIA

The Office building within the Nuclear Physics Institute at Magurele is a reinforced concrete building,square in plan of 26 m side and 44 m height,with basement,ground floor and 10 stories (Fig.1).The main structural system consists of reinforced concrete walls,with and without openings,and floor slabs of precast parts and cast in in place parts.Most of the walls have thickness of 25 cm. The main reinforcement is placed at the ends of the sections of walls and ranged from 0.46% to 0.90%.The horizontal reinforcement consists of two layers of bars 10 mm at 20 cm.The yield stress of reinforcement used was 3600 kp/cm^2,and the specified strength of concrete 200 kp/cm^2.

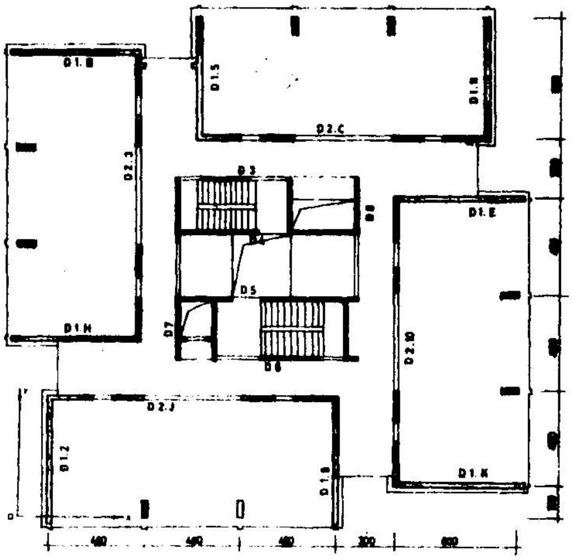

Fig. 1 Typical floor plan

The building was designed by the Design Center of the Ministry of Education in Bucharest and it was analysed for lateral loads according to the earthquake regulations at the time of design(p.13-70).The review of this results showed that the seismic load corresponded to VII degree of intensity,with a shear base factor of c = 2.62% .After the earthquake,at the Design Institute "Carpati"-Bucharest,an analysis was performed and all member forces were obtained,with the conclusion that all reinforced concrete sections are sufficient and consequently the first design was correctly prepared.

3. GROUND MOTION AND STRUCTURAL DAMAGE

Ground motion of March 4,1977 was recorded in Bucharest at the Institute INCERC. The three components of the record(Fig.2) identified an earthquake with characteristics very different from those which we know as classical (earthquakes in California,Japan and USSR). The characteristics of this earthquake for Bucharest can be synthesized as follows:

- the maximum values of the ground motion for the N-S direction were: a_m = 208,2 cm/sec^2 (0,21 g), v_m = 74.9 cm/sec and d_m = 25,4 cm/sec.
- the mode shape of the predominant pulses has a period larger than 1.0 sec.
- the response acceleration spectra has an upper ceiling with periods of 0.5 to 2.0 and a lower ceiling with periods larger than 3.0 sec.These three

characteristics made the conditions of the structures rather unfavourable with periods larger than 0.5-0.6 sec (semi-rigid and flexible structures).The Magurele building has an initial period of vibration of 0.7 sec,according to the design,but during the earthquake the period increased due to the decrease of rigidity and put the structure in an unfavourable condition regarding the periods.

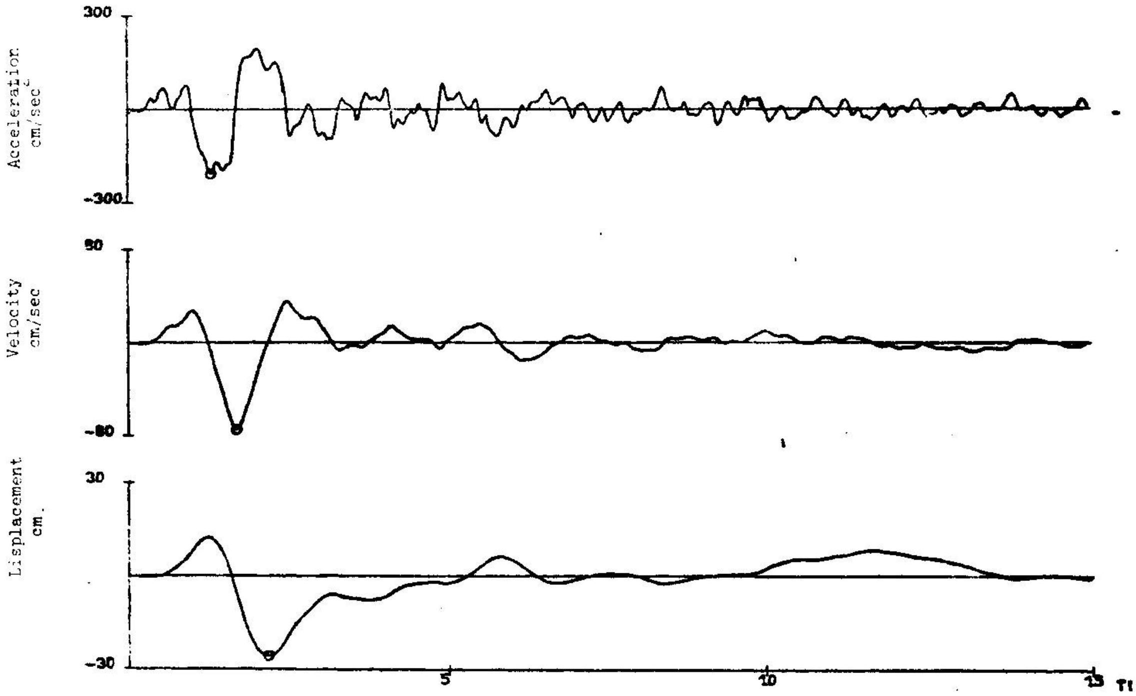

Fig.2 Record of the earthquake of March 4,1977 - INCERC

During the earthquake the main structural system suffered moderate damage.Many cracks in corridor walls and wing walls($D_2.3$ and $D_1.5$ in Fig.1) were observed and less in core walls (D_3 to D_8 in Fig.1).Cracks in walls and deep beams were inclined,whereas flexural cracks at the ends of the shallow beams appeared. Also,some cracks in the floor slabs,near the openings,were observed.Cracks in walls with openings were distributed over all height of the building,whereas in walls without openings were limited to the bottom part only.

4. STRUCTURAL MODELING

The elastic analysis was performed for the whole building,taking into account all the walls in both directions,by application of the TABS [2] programme.The walls with openings are treated as frames with rigid zones,while the walls without openings act as single columns fixed at the base.For the elastic dynamic analysis twelve modes and viscous damping of 4% of the critical in each mode of vibration were taken.The masses are concentrated at the level of floor slabs and are determined on the basis of 1.3 tons on a square meter of gross area.

The inelastic dynamic analysis was carried out applying the DRAIN-2D [3] programme.For the analysis the walls D_{1-5} and D_{2-3},treated separately as plane structures with masses concentrated at the nodes,have been chosen.The wall D_{1-5} is divided into twenty members with lower height in the lower part,and above the fifth storey the members have the storey height (Fig.3).Plastic hinges can be formed at both ends of the members during the dynamic action of the earthquake. The wall D_{2-3} is treated as a three-span frame (Fig.4),consisting of two types of members,columns and beams.The ultimate capacity of beam and column sections was calculated using computer programs,and given as interaction diagram M-N for the columns and yield moment M_y for the beams.

The viscous damping is taken to be 4% of the critical and,based upon the periods for two modes of vibration,the damping as proportional to the mass and rigidity was calculated.

The data from the accelerogram obtained at INCERC have been digitized at IZIIS and used as input for elastic and inelastic dynamic response analysis. In total 875 steps of 0.008 sec,or 7 sec. are taken into account for the time history of the building behaviour.

5. ELASTIC ANALYSIS

The elastic analysis for the whole building was conducted supposing that no collapse of the structural members as a result of the earthquake will occur,i.e the behaviour of the members will be linear.In addition to the earthquake effect ,the building was analysed for vertical load and for statical horizontal forces equivalent to the shear base factor of $c= 8\%$.

The envelope of the maximum floor displacements is shown in Fig.5.The considerable differences between the displacements point out the need for inelastic dynamic analysis,since these displacements cannot be reached in the elastic behaviour of the structural members.

During investigation of the ability of the building to withstand the March 4, 1977 earthquake,the ratio between "the maximum moment at a section from the gravity and seismic loading,divided by the ultimate mument capacity of the section " was required [4] . These relations are shown in Fig.6 as "ductilities" for the beams of the wall D_{2-3}.The maximum moments from the earthquake are considerably larger than the flexural capacity of the sections,which results in the very high "ductilities".

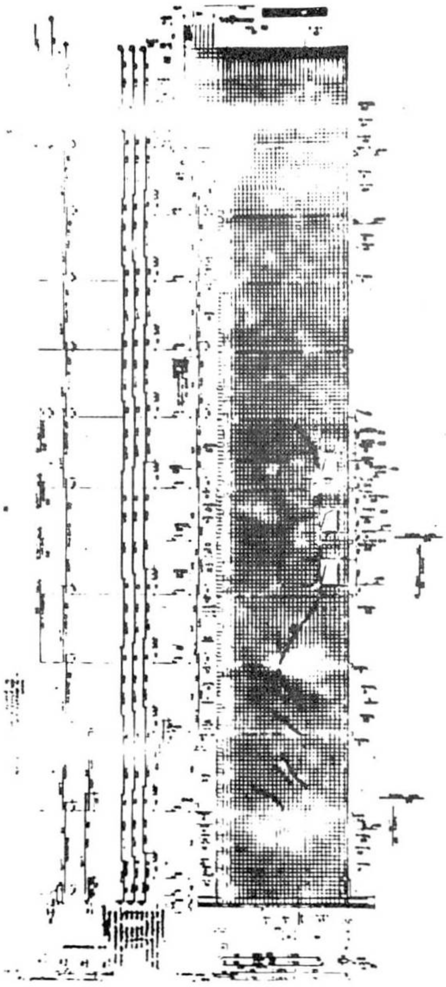

Fig.3 Elevation of wall D_{1-5}

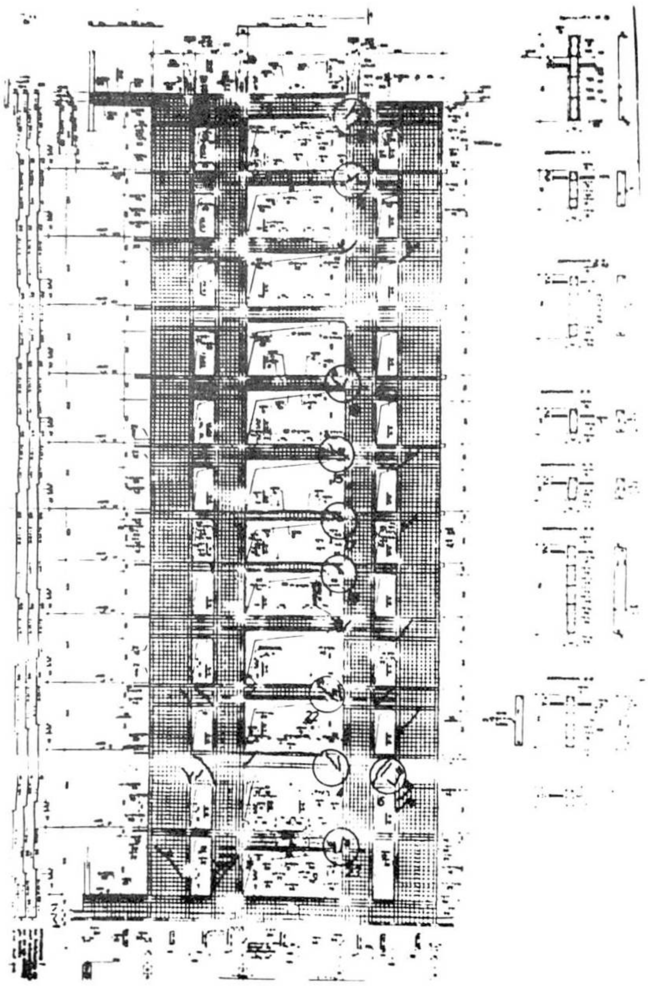

Fig.4 Elevation of wall D_{2-3}

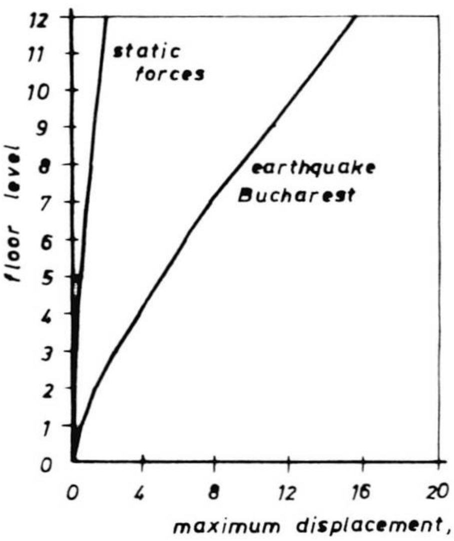

Fig.5 Envelopes of maximum elastic floor displacements

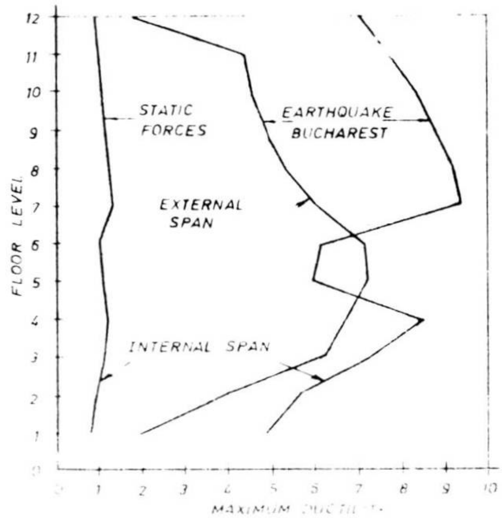

Fig.6 Envelopes of maximum beam ductilities,D_{2-3}

6. INELASTIC ANALYSIS

The inelastic dynamic analysis of the walls D_{1-5} and D_{2-3} is based upon the bilinear histeresis relationship moment-rotation of the wall members, in which the deformations are concentrated at the ends of the members. The version of the DRAIN-2D programme used do not posess the option for stiffness degrading of the beams due to previous yielding, but it was pointed out that the influence of degrading stiffness is not significant [5] .

Before the dynamic action, the gravity load has been applied. For the applied acceleration the dynamic response of the walls for the following values is required:

- horizontal displacement of the nodes
- vertical displacement of the nodes
- rotation of the nodes
- member forces
- plastic rotation at the end of members.

These values are obtained for each 0.2 sec of the adopted time duration of the earthquake. At the end of the analysis, the maximum values for the member forces and the plastic rotations in all members are obtained.

The fig.7 illustrates the maximum storey shears obtained from elastic and inelastic analysis. Although direct comparison is not possible, due to the separate treatment of the walls in the inelastic analysis, it could be stated that the shear forces obtained by the elastic analysis are too high.

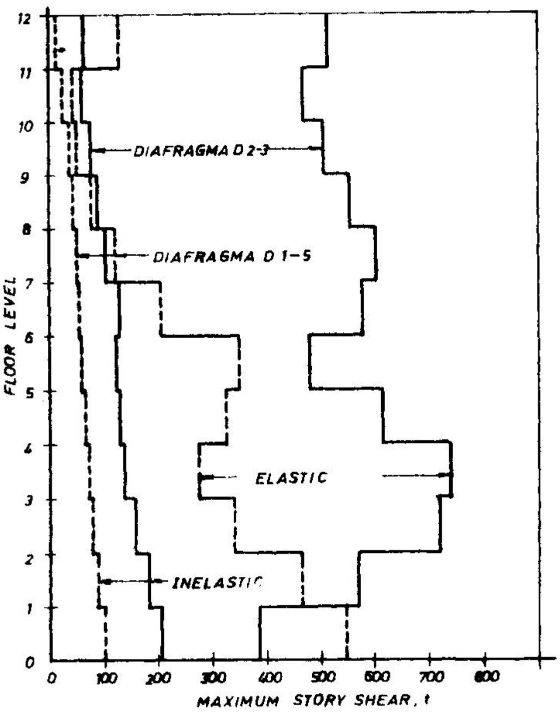

Fig.7 Envelopes of maximum storey shears

The time history of the displacement at the top of the walls D_{1-5} and D_{2-3} is shown in Fig.8.It could be said that the maximum elastic and inelastic displacements were similar.

In the Fig.9 the plastic rotation at the base of the wall D_{1-5} and at the bases of the external and internal column of the wall D_{2-3} are shown.While in the wall D_{1-5} plastic rotations are observed only at the first three stories,in the wall D_{2-3} plastic rotations occured along the whole height,especially at the coupled beams.

The Figs. 10 and 11 show the change of the moments and the axial forces at the base of the columns in D_{2-3} and sometimes the axial forces are larger than those corresponding to the balance point.

Fig 12 illustrates the location and the sequence of forming of plastic hinges in the wall D_{2-3}.The plastic hinges indicated did not occur simultaneously,but generally in groups concentrated at a floor level.

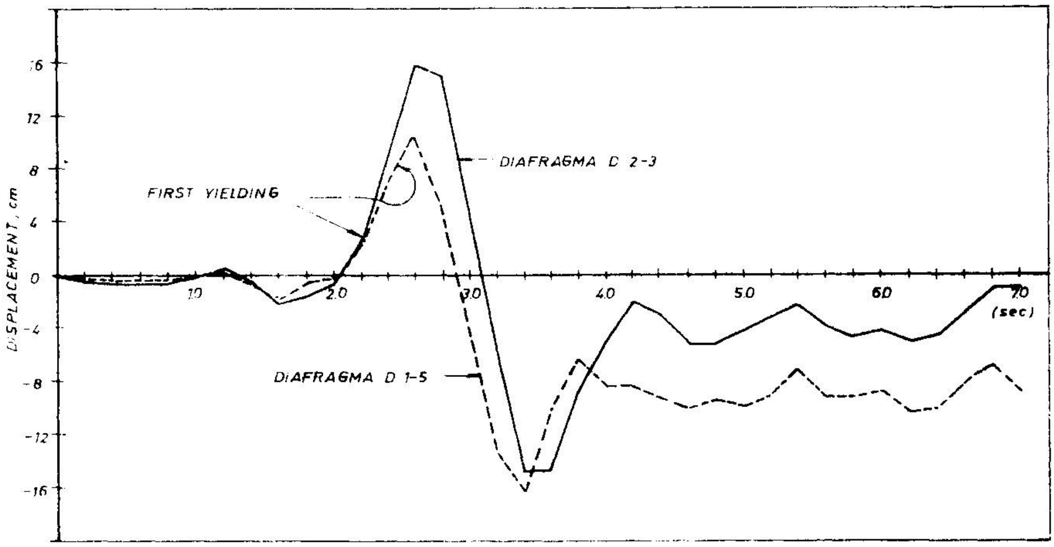

Fig.8 Time history of horizontal displacements at the top of the structure

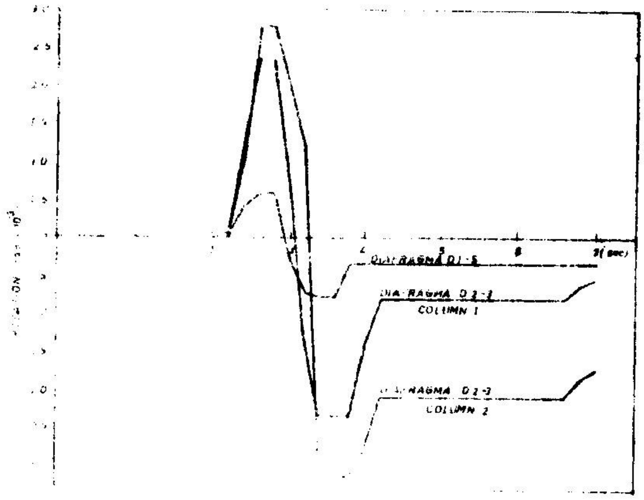

Fig.9 Time history of plastic hinge rotations at base of D_{1-5} and D_{2-3}

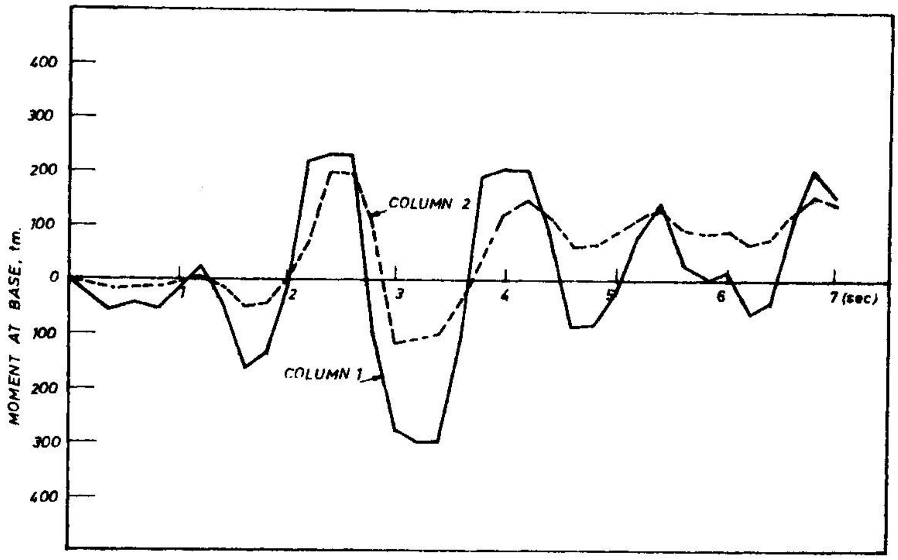

Fig.10 Time history of bending moment at base of D_{2-3}

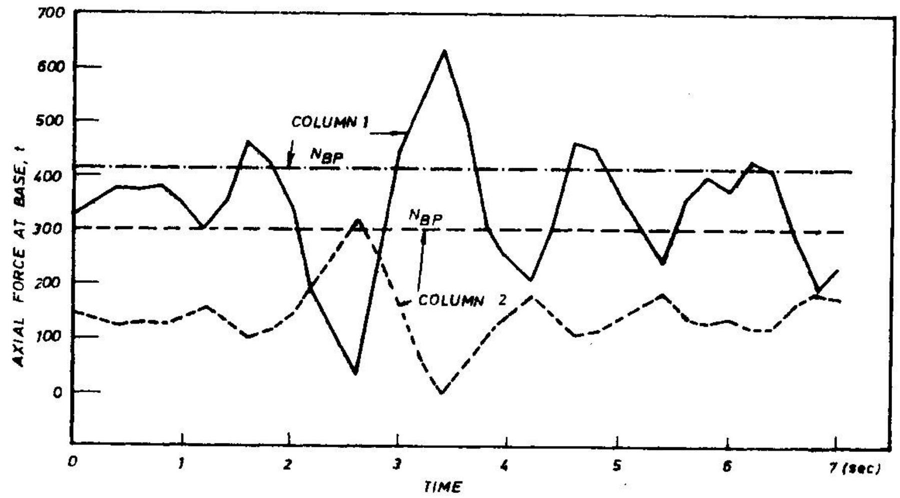

Fig.11 Time history of axial forces at base of D_{2-3}

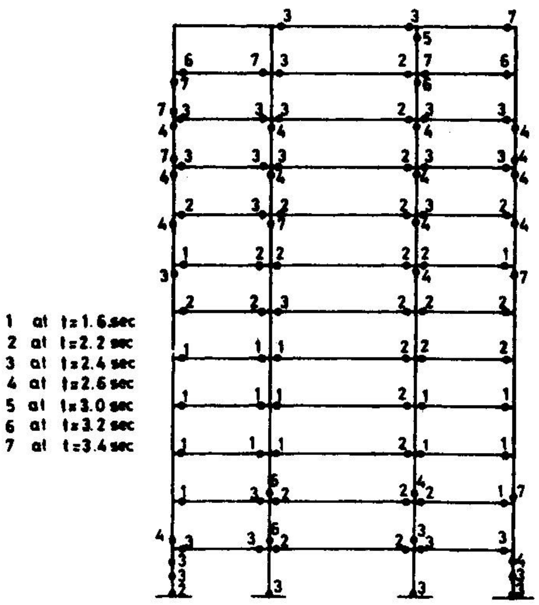

Fig.12 Sequence and location of plastic hinges, D_{2-3}

7. CONCLUSIONS

In summary of the analysis carried out the following conclusions can be drawn:

- The structure of the building was designed according to the Earthquake Regulations in Romania, for an intensity of VII degree, which is not in accordance to the actual intensity and other dynamic characteristics of the March 4,1977 earthquake.

- The most important structural members of the building are the shear walls, frame walls and the core. The specific behaviour of these members points out to the large stresses in the floor slabs, which can be proved by the observed damage. For mixed systems the slab is important structural member as well as the vertical members.

- The comparison of the story shears from the elastic and inelastic response shows that the latter are considerably smaller, which means that the structure acts nonlinearly and requires a source of damping, not only viscous, but also through inelastic deformations.

- The coupling beams of the walls with openings are very suitable members for energy dissipation and the suggestion for strengthening of the building was to make them good elasto-plastic dampers.

- The analysis carried out clearly indicate that the superstructure of the building should be strengthened, but it should not be more than the capacity for overturning moment at the joint of the superstructure and infrastructure.

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