International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:6, No:9, 2012

Dynamic Analysis of the Dome with Arches and Rings from Romexpo Bucharest

V. Precupas, A. Ivan, M. Ivan

***Abstract*—**The dome with ribs and rings, which covers the ROMEXPO pavilion from Bucharest, was designed after the collapse of the single layer reticulated dome. In this paper, it was made the checking of the structure, under the dynamic loads with three recorded accelerograms calibrated according to Romanian seismic design code P100-1/2006. Under the action the dynamic loadings, it was made a time-history analysis to determine the zones where the plastic hinges appear, at what accelerations and their position on the structure. The studied dome is formed by 32 spatial semi arches and three rings: one circular ring located at the top of the dome and another two rings, design as trusses, the first near the supports and the second as an intermediate rings above the skylights. Above the skylights up to the top, the dome is tight together with purlins and bracings.

***Keywords*—**dome, dynamic analysis, plastic hinges, time-history

I. Introduction

THE analyzed structure is a spherical dome with arches and rings, it was built in 1964 in Bucharest, after the collapse of the initial reticulated dome on the same site [10]-[12], in winter 1963-1964, due to snow crowding, malfunction of the deicing and small rigidity of joints. The second dome was designed by [1], more rigid to prevent the loss of stability due to unsymmetrical snow crowding. The geometry of the sphere is given by the quadratic equation, using the Cartesian system of coordinates x, y and z.

$$x^2 + y^2 + z^2 = R^2 \tag{1}$$

The main geometric characteristics of the dome are: the span 93,5m, the curvature radius 70m and the height of 17,90m.

The dome is divided in 32 spatial semi arches, with triangular cross section; pin supported on a pre-stressed concrete ring at the bottom, which prevent the below structure to induce dynamic loadings to the dome; the continuity of the semi arches is assured at the top by a central ring, designed as a caisson.

To overtake the thrust the below structure was completed with a reinforced concrete rigid ring. Above the central ring is executed a small dome for lighting and ventilation.

V. Precupas. PhD Student at „Politehnica" University of Timişoara, Faculty of Constructions, Metallic Constructions and Construction Mechanics Department, Romania (e-mail: vinicius_precupas@yahoo.com).

M. Ivan. Professor Dr. Eng. at „Politehnica" University of Timişoara, Faculty of Constructions, Metallic Constructions and Construction Mechanics Department, Romania (e-mail: adrian.ivan@ct.upt.ro).

A. Ivan. Associate Professor at „Politehnica" University of Timişoara, Faculty of Constructions, Metallic Constructions and Construction Mechanics Department, Romania (e-mail: adrian.ivan@ct.upt.ro).

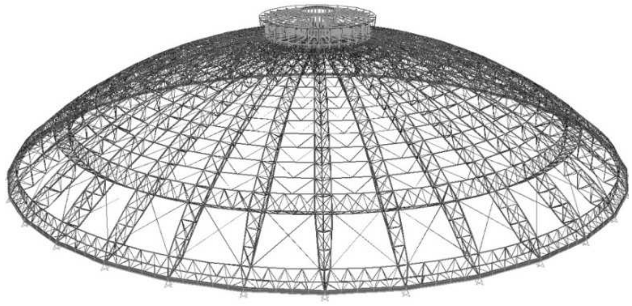


Fig. 1 General view

II. Geometry of The Dome

The dome is made from 32 semi arches and three rings. The arches have a curvature of 70m and they are made as spatial trusses, triangular with the height of 2,1m [3].

The chords of the semi arches are executed curved with the exterior diameter of 146-152mm and with the thickness of 14-18mm, the distance between the two bars of the upper chord, vary from 1, 2 to 2m.

The diagonals are formed from pipes with the diameter of 70-89mm, the thickness of 6-10mm and placed in shape of X and struts between them.

The bracings are arranged in the plane of the purlins in K system, with the cross section of 60x6mm, an in the skylight zone are arranged in X system with same cross section. The quality of the used material for the metallic pipes is OLT45.

The purlins which sustain the cover have circular shape and are made from pipes with the diameter of Ø70-127mm and with the thickness of 6-12mm.

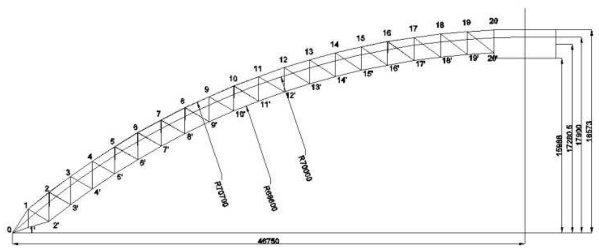


Fig. 2 Longitudinal section of a semi arch

To take over the efforts from the pushing of the arches, are disposed three rings: the ring beam A, at the base of the dome, the ring beam B placed above the skylight and the central ring.

The ring beam A from the base of the dome is formed as a spatial beam with three chords: two from curved pipes Ø121x12mm and one from I profile.

The ring beam B, is made from curved pipes of Ø127x11mm and diagonals from Ø70x6 pipes.

The central ring is the element which makes the bounding of the semi arches at the top;it's cross section is made as a caisson, made by metallic elements riveted together. The radius of the ring is 2825mm.

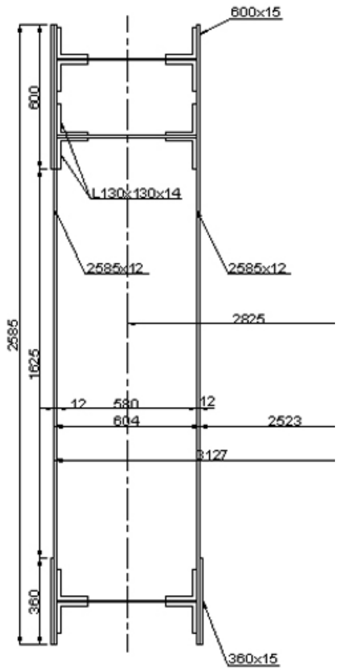


Fig. 3 Central ring

III. Structure Computation

The structure was statically analyzed on three models, in the first models: only with arches and rings; in the second model it was taken into account the cooperation with the purlins, and the third model is completed with the bracings.

The computing was made on the three models, in which it was taken into account the cooperation of the purlins, respectively the cooperation of the purlins and bracings in K and X in the skylight zone.

A. Design loadings after actual codes

The actual computing, after which was made the static analysis:

- Self-weight of the structure, including the cover and the walkway [3];
- The live load of the suspended ceiling [3];
- The live load from the handling of the exhibits: 4 concentrated forces of 50kN [3];
- Uniform snow loading of 160 daN/m^2 [7];
- Exceptional snow loading, was determined experimentally from the wind tunnel [3];
- Wind action according to [6];
- Seismic action [4];
- Temperature variation ±35° [3].
- Coefficients and combinations taken according to Bases of Design Code [5].

B. Static analysis

The modeling and static computation was made with the help of SAP2000 Nonlinear v11 program. On the three models with/without cooperation of the purlins of bracings, were computed the efforts in bars and displacements after the linear static and nonlinear static computation.

The chords of the 32 semi arches are considered continue, pin connected at the base in the pre-stressed concrete ring and up at the top in the central ring. Similar the chords of the ring beams A and B are continue, articulated in contact with the semi arches. The diagonals of the semi arches and of the rings, respectively purlins and bracings are considered all articulated.

The length of the finite elements for nonlinear computation is considered the length between two adjacent nodes.

C. Accelerograms

To determine the plastic hinges of the structure under seismic dynamic action it was made a nonlinear dynamic analysis (time-history). For the analysis it was needed three data sets, each containing two horizontal components and one vertical component, of ground motion acceleration records. To satisfy these criteria, it was chosen the following seismic movement recordings:

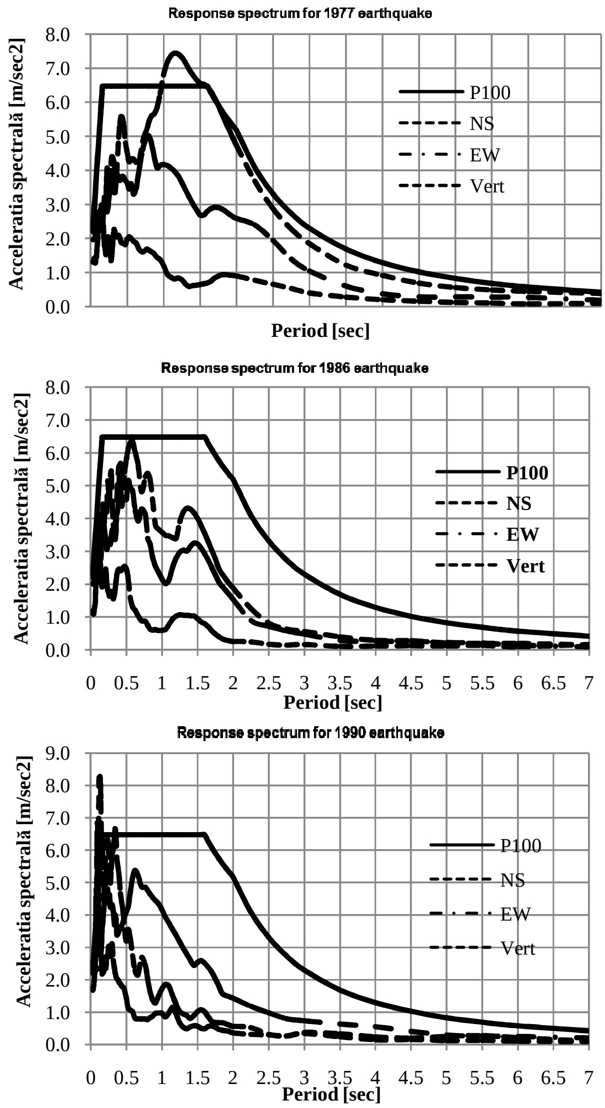


Fig. 4 Recorded acceleration spectrum used in the dynamic analysis a) 1977; b) 1986 c) 1990

International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:6, No:9, 2012

a) Vrancea earthquake from 4 March 1977, directions NS, EW and Vertical, recorded at INCERC Bucharest, peak acceleration = -194.927 cm/sec^2 at 6.12sec, length: 40.14sec;
b) Vrancea earthquake from 30 August 1986, directions NS, EW and Vertical, recorded at Bucharest-Magurele, peak acceleration = 135.45 cm/sec^2 at 19.73sec, length: 57.55sec;
c) Vrancea earthquake from 30 may 1990, directions NS, EW and Vertical, recorded at Bucharest-Magurele, peak acceleration = 89.588 cm/sec^2 at 15.68sec, length: 56.28sec.

All the accelerograms were recorded in Bucharest, where is the location of the dome. The recorded accelerograms were scaled, so that the peak acceleration of the accelerogram to be the same with the ground acceleration given by the seismic code P100-1/2006 (0.24 g).

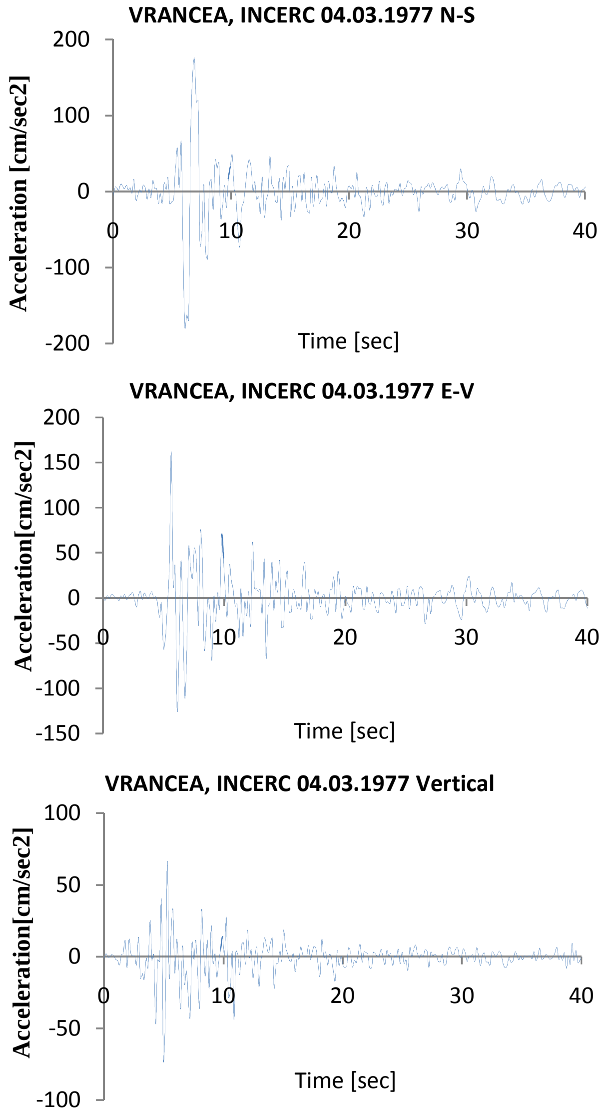


Fig. 5 Recorded acccelererograms on N-S, E-V and vertical direction, used in analysis from Vrancea earthquake, 4 March 1977, INCERC Bucharest

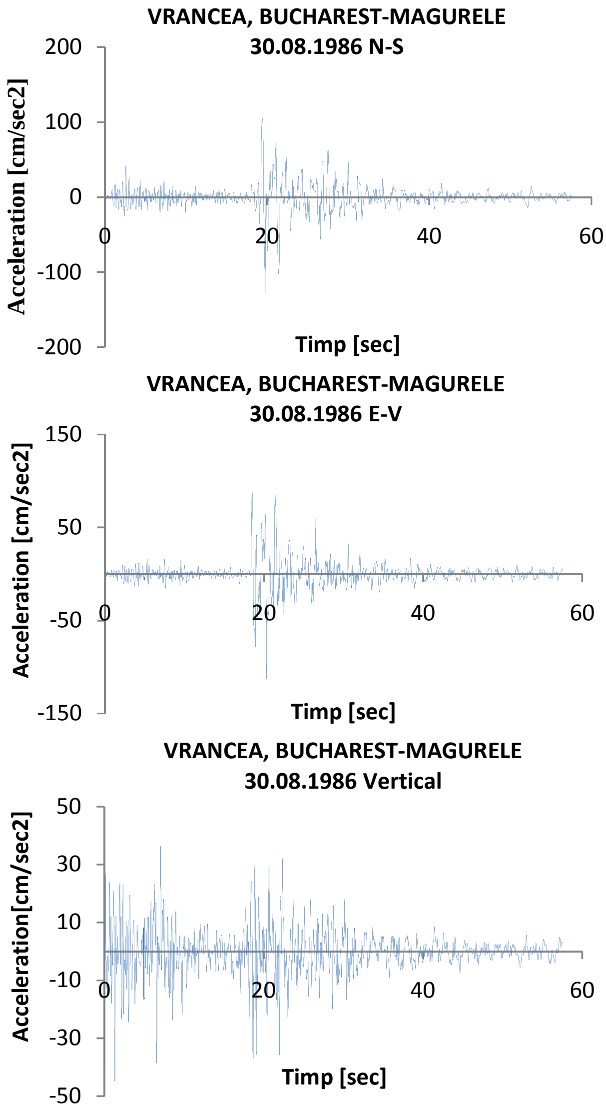
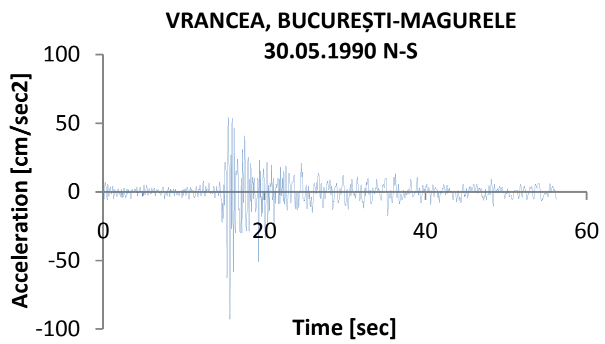


Fig. 6 Recorded acccelererograms on N-S, E-V and vertical direction, used in analysis from Vrancea earthquake, 30 august 1986, Bucharest-Măgurele



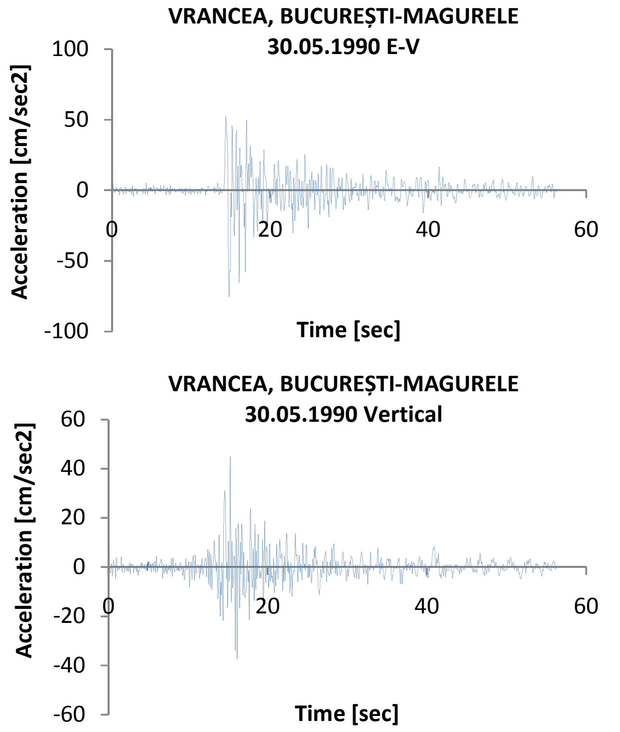


Fig. 7 Recorded accelererograms on N-S, E-V and vertical direction used in analysis from Vrancea earthquake, 30 May 1990, Bucharest-Măgurele

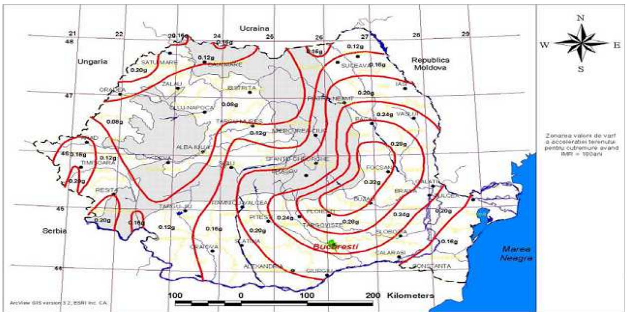


Fig. 8 Ground acceleration zones [4]

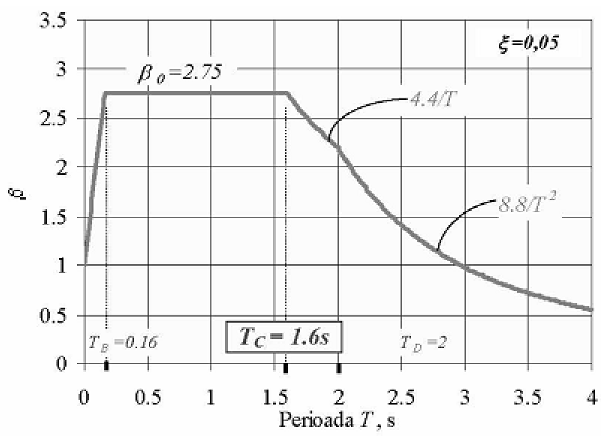


Fig.9. Elastic response spectrum for acceleration for horizontal component of ground motion for Bucharest [4]

$$0 \le T \le T_B \qquad \beta(T) = 1 + \frac{(\beta_0 - 1)}{T_B} T \tag{2}$$

$$T_B \le T \le T_C \qquad \beta(T) = \beta_0 \tag{3}$$

$$T_C \le T \le T_D \qquad \beta(T) = \beta_0 \frac{T_C}{T} \tag{4}$$

$$T_B \le T \le T_C \qquad \beta(T) = \beta_0 \frac{T_C T_D}{T^2} \tag{5}$$

where:

$\beta(T)$ – nominalized elastic response spectrum;

β_0 – maximal dynamic amplification factor of horizontal acceleration by the structure

T – vibration period of a structure with a dynamic degree of freedom and elastic response

D.Combinations

The combination of the seismic effects was taken as prescribed according to Romanian seismic standard [4]:

a) The evaluation of the structure response separately for every direction of seismic action.

b) The combination

$$0{,}30E_{dx} \text{ “+” } 0{,}30\ E_{dy} \text{ “+” } E_{dz} \tag{6}$$

$$E_{dx} \text{ “+” } 0{,}30E_{dy} \text{ “+” } 0{,}30\ E_{dz} \tag{7}$$

$$0{,}30E_{dx} \text{ “+” } E_{dy} \text{ “+”} 0{,}30\ E_{dz} \tag{8}$$

Where:

“+” means “combined with”

E_{dx} represents the effects of the action due to the application of the seismic movement on the horizontal direction of axis X chosen on the structure;

E_{dy} represents the effects of the action due to the application of the seismic movement on the horizontal direction of axis Y chosen on the structure;

E_{dz} represents the effects of the action due to the application of the seismic movement on the vertical direction of axis Z chosen on the structure;

E. Dynamic analysis

The used analysis for dynamic response of the structure was time-history, where it was used three sets of accelerograms (1977, 1986 and 1990) with recorded accelerations on three directions N-S, E-W and vertical, according to fig. 5-7. Because the peak of the real recorded earthquakes is smaller than the ground acceleration given by [4] (0,24g), they were scaled so that the peak accelerations are the same as the ground’s acceleration.

For the analysis it was used the program SAP 2000 v11, program which allows the modeling of the plastic hinges. The plastic hinges were modeled according to [15], in the potential plastic sections of elements; for the trusses, purlins and bracings, the plastic hinges were modeled due to axial loading, while for the central ring and chord A from the first ring beam, the plastic hinges were modeled due to moment-rotation.

The geometric nonlinearity was taken into account by considering the P-Delta effect.

From the dynamic time-history analysis it was obtained the plastic hinges, the appearance of the yielding in bars and their collapse. Also with this analysis was obtained the order of appearance of the hinges and the time when they occurs.

IV. Bars Plasticization

Time History method involves a time step by time step evaluation of building response, using discretized recorded earthquakes as base motion input. The mathematical model takes into account both the plan and vertical spatial distribution of the energy dissipation. The design displacements are determined directly through dynamic analysis using ground motion time histories[15]

In dynamic time history analysis, the structure is first analyzed with initial loadings: 1 Dead Load + 0.4 Live Load + 0.4 Uniform snow Load; including P-delta effect, after which the structure is analyzed nonlinearly using direct integration time history type. In this type of analysis, the equations of the entire structure are solved each time step.

The bracings from the skylights are already in immediate occupancy IO position, when the earthquake starts, and they reach the collapse and failure in point E, for the horizontal direction ground movement and their combination. These bracings reach the collapse point C, in the zone of the peak ground acceleration and fail entirely afterwards, dissipating an important part of the seismic energy.

When the skylights bracings are in collapse prevention point CP, plastic hinges start to appear at the bracings above the skylights, reaching the failure in more points. The plastic hinges above the skylights appear in bracings up to the corresponding nodes 9-10 of the semi arch.

The vertical component of the ground movement alone, without the combination of the horizontal directions has no effect on the structure, due to lower accelerations on vertical compared with horizontal direction. Combining the vertical ground movement accelerogram with horizontal ground movement accelerogram, where the vertical component is the primary component, the bracings yield up to CP position.

Beside the bracings, the diagonal bars from the upper chord of the semi arch trusses start to yield in B point, and in some points reaching yielding to immediate occupancy IO point, having small effect on the structure after the earthquake passes. These yielding bars are located between nodes 1-2 and 4-5 on the semi arch top chord. Another zone where the bars start to yield in point B are the diagonal bars of ring beam B, in the section next to the semi arch.

On the typical load-deformation relation for plastic hinges, the line DE represents the residual strength of the member, which is zero in all cases, after collapse the bars jumps directly to total failure.

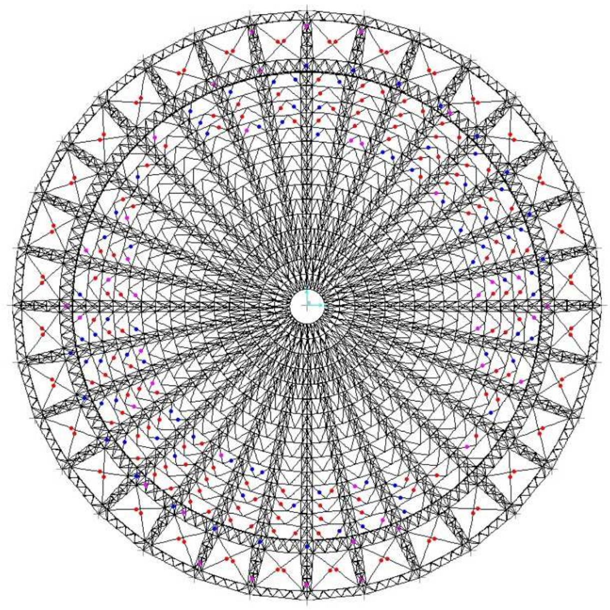


Fig. 10 Distribution of plastic hinges on the structure

TABLE I
ELEMENT'S PLASTICIZATION
The time when the first plastic hinges appear in different states [sec]

Accelerograms	B	IO	LS	CP	C	D	E
1977 – NS	0	0	5.36	5.44	6.04	-	10.96
1977 – EW	0	0	4.96	5.00	5.56	-	10.64
1977 - Vertical	0	0	-	-	-	-	-
1977 – NS+0.3EW+0.3V	0	0	5.32	5.44	6.04	-	10.52
1977 – 0.3NS+EW+0.3V	0	0	4.92	5.00	5.56	-	10.12
1977 – 0.3NS+0.3EW+V	0	0	5.52	5.64	-	-	-
1986 – NS	0	0	2.42	19.36	19.4	-	21.72
1986 – EW	0	0	18.44	18.48	18.52	-	21.48
1986 - Vertical	0	0	-	-	-	-	-
1986 – NS+0.3EW+0.3V	0	0	2.48	18.6	19.44	-	21.52
1986 – 0.3NS+EW+0.3V	0	0	3.80	18.48	18.52	-	21.88
1986 – 0.3NS+0.3EW+V	0	0	18.52	18.6	-	-	-
1990 – NS	0	0	14.76	14.80	15.64	-	23.48
1990 – EW	0	0	14.32	14.36	14.44	-	17.12
1990 - Vertical	0	0	-	-	-	-	-
1990 – NS+0.3EW+0.3V	0	0	14.44	14.76	15.64	-	22.88
1990 – 0.3NS+EW+0.3V	0	0	14.32	14.36	14.44	-	17.52
1990 – 0.3NS+0.3EW+V	0	0	14.44	14.52	-	-	-

In the fig. 10 and table 1, we have the notations [15]

B - Steel yielding
IO - Immediate occupancy
LS - Life safety
CP - Collapse Prevention
C - Collapse
D - Residual strength
E - Deformation limit

V. Conclusion

- The skylight bracings are already yielding, before the earthquake begins, being in immediate occupancy state;
- The first elements which reach the collapse and failing are the skylight bracings;

- All the elements which collapse during the earthquake are the bracings, starting from base up to the middle span of the semi arches (node 9-10);
- The diagonals of the top chord between node 1-2 and 4-5, as well as the diagonals on the ring B, near the semi arches need attention after an earthquake, because after the passing of the earthquake they remain deformed.
- For further analysis is proposed to study the dynamic response of the dome with the introduction of additional elements in the zone of the skylight. These additional bars will prevent the buckling in-plane and out of plane of the skylight bracings. The purpose of this further research is to get elastic response of the structure according to actual seismic design code requirements.

REFERENCES

[1] D. Mateescu and collaborators, Welded metallic dome of National Economy Exhibition Pavilion of R.P.R.
[2] A. Ivan, *Single layer reticulated metallic dome instability*, Romanian Academy Publishing, Bucharest, Ph. D Dissertation, 2001.
[3] Technical design „*Remaking of the National Economy Pavilion roof*" developed by collective of I.P.C.M.C. Bucharest, 1963
[4] Seismic design code – Part I – *Design provisions for buildings*, code P100-1/2006.
[5] Design code CR 0-2005, *Bases of design for structures in construction*.
[6] Design code NP-082-04. *Bases of design and actions on constructions. Wind loading.*
[7] Design code CR 1-1-3-05 *Evaluation of snow loadings upon constructions.*
[8] STAS 10108/0-78 *Steel elements computation*.
[9] Eurocode 8 – *Design provisions for earthquake resistance of structures – Part 1-1*;
[10] E. Baiculescu, M. Enescu, *The design of the pavilion of National Economy Exhibition of R.P.R.* in "Construction and construction materials magazine". 1962
[11] M. Soare, N. Pătrîniche, *The metallic reticulated dome from National Economy Exhibition pavilion of R.P.R.*, in "Construction and construction materials magazine". 1962.
[12] T. Dinescu, M. Anastasescu, *The mounting design of the dome from National Economy Exhibition pavilion of R.P.R.*, in "Construction and construction materials magazine". 1962.
[13] F. DINU, *Methods of nonlinear computation of metallic frame structures loaded by seismic action*, "Edituraorizonturiuniversitare" editure, Timişoara, 2006
[14] D. Dubină, D.Lungu (coordinators), authors collective Aldea A., Arion C., Ciutina A., Cornea T. Dinu F., Fulop L., Grecea D., Stratan A., Văcăreanu R. Constructions placed in strong seismic movement zones, "Edituraorizonturiuniversitare" editure, Timişoara, 2003
[15] FEMA 356, *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Acency, Washington D.C., 2000