ANALYTICAL MODELING OF SEISMIC STABILITY OF PALASPORT- BOLOGNA USING EXPERIMENTAL DATA

DENEYSEL VERİLER KULLANILARAK BOLOGNA SPOR SARAYININ DEPREM ANALİZ MODELİ

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ABSTRACT

The seismic stability of the Sport Hall (Palasport) in Bologna is investigated by analytical modeling based on experimental data. The mathematical model is considered as a 3D model with 203 nodes and 564 elements and concentrated masses at each node. SAP90 computer program has been used to perform the linear dynamic analysis. In the first part of the paper, some design aspects of the structural system are presented. In the second part, summary of the experimental results related to the dynamic properties of the structure and the tested model, is presented. Comparison between the experimental and analytical results has been performed, first for the physical model and than for the actual structure. The conclusions on the seismic behavior of the structure subjected to the considered ground motion are given at the end of the paper.

1. DESCRIPTION OF STRUCTURAL SYSTEM

The Palasport in Bologna - Italy, is a special structure, almost elliptical in plan, with dimensions of the median axes of 80/120 m. The roof structure has been constructed of lamellar wooden beams linked by steel joints, forming one cylindrical and two half spherical parts. The lower part of this structure consists of 52 reinforced concrete columns, out of which 26 are supported by two steel braces from the external side of the structure. The connections between the wooden beams are made of steel rings (internal joints), while the external, joints between the columns and beams, of vertical steel plates and half rings. The plan of the Bologna Palasport is shown in Fig 1.

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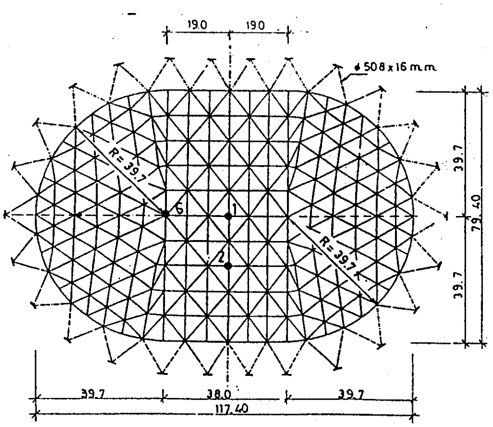

Fig 1 Disposition of plan of the Bologna Palasport

2. SYNTHESIS OF THE EXPERIMENTAL RESULTS

2.1. Experimental results from full scale testing

In order to define the dynamic properties of the structure, two different testing methods have been applied: forced and ambient vibration methods.

The ambient vibration test was performed for preliminary checking of the resonant frequencies of the structural system in two orthogonal horizontal directions and vertical direction, respectively, within the frequency range of 0-30 Hz. The equipment consisted of two Ranger seismometers, model SS-1, Kinemetrics production , USA, Signal conditioner, type SC-1, Frequency analyzer. model 3582A, Hellwet Packard production - USA. The seismometers were placed on the top of the cover structure at points 1, 2 and 6, as shown in Fig. 1. The time history records have been transformed in frequency domain using a two-channel frequency analyzer for obtaining Fourier amplitude spectra. The peak values of the spectra correspond to the resonant frequencies of the tested structure.

The forced vibration test was performed for more precise definition of the resonant frequencies (preliminary defined by ambient vibration method) as well as for definition of horizontal and vertical mode shapes. In this case, a small electrodynamic actuator, type 113 electro-seis shaker- USA, placed on the top of the structure, was used for the excitation of a harmonic force of 150 N within the frequency range of 0-30 Hz. . After definition of the resonant frequencies, the mode shapes were defined by recording the response at several points along the longitudinal and transversal profiles, as shown in Fig. 1. The Fourier amplitude spectra obtained from the ambient vibration test as well as the resonant frequency curves obtained from the forced vibration test are presented in Fig. 2.

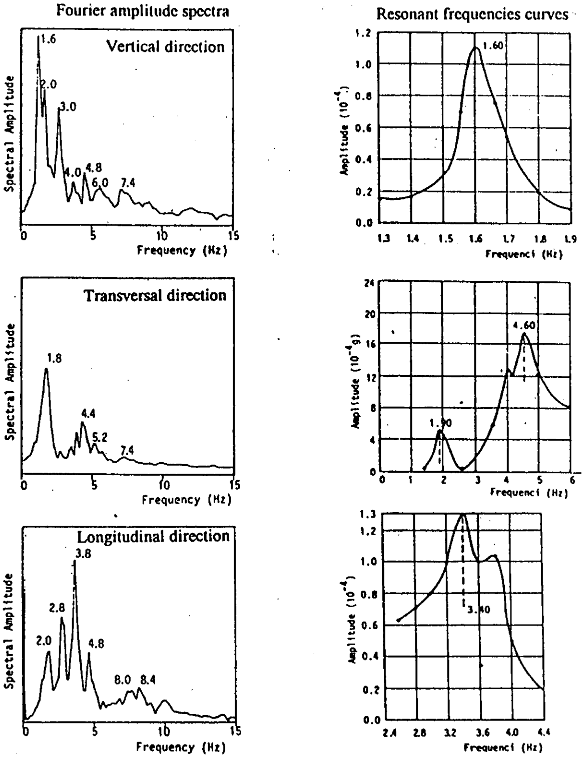

Fig 2 Fourier amplitude spectra and frequency response curves obtained for full-scale testing of the Palasport

2.2. Experimental results from shaking table testing

For shaking table testing, the choice of the linear scale, l_r , is limited by the size of the table. Considering the geometry of the structure and the dimensions of the shaking table at the IZIIS Dynamic Testing Laboratory (5/5 m), a model in scale of 1/30 was adopted as the most convenient one. Taking into account the linear scale, a model with so-called artificial mass simulation was used.

The dynamic characteristics of the model have been obtained by exciting the model with the same small shaker, used for testing the actual structure (Table 1). The results show satisfactory agreement with those obtained from the full scale test of the actual structure, considering the scaling factors. The mode shapes of the model in vertical direction, in comparison with the prototype, are presented in Fig. 3.

In order to estimate the most unfavorable earthquake, several earthquakes have been simulated: Ancona, Breginj, Ulcinj, Petrovac and El Centro. Comparing the response data, the most unfavorable time history is Ancona in vertical, and Petrovac in horizontal direction.

Table 1 Tabular presentation of the fundamental frequencies of the prototype and model

Direction	Prototype Frequency (Hz)	Model Frequency (Hz)		
		Required Model Frequency (Hz)	Obtained Model Frequency (Hz)	Difference (%)
Vertical	1.9	10.4	11.2	7.7
Transversal	1.6	8.8	9.6	9.1
Longitudin.	3.4	18.6	18.0	3.2

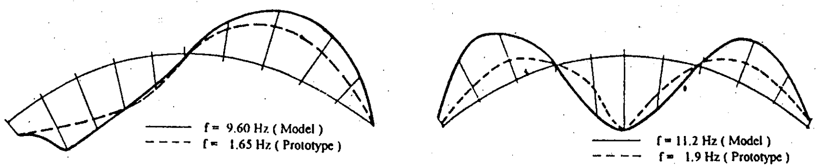

Fig 3 Comparative presentation of prototype and model shapes of vibration

3. FORMULATION OF MATHEMATICAL MODEL

3.1. Modeling conditions

The mathematical model is considered as a 3D model with 203 nodes and 564 elements- rectangular beams as shown in Fig. 4. The model is simplified considering the following assumptions:

- Only the primary truss system is considered. The secondary and tertiary systems are neglected
- The supporting part of the structure is not included in the model. It is assumed as ideally stiff, with fixed end boundary conditions
- Each node has six degree of freedom
- The masses are concentrated at the nodes

3.2. Analytical results

The mathematical model is based on SAP90 computer program. Actually, two mathematical model have been developed:

- Mathematical model simulating the behavior of the reduced scale physical model
- Mathematical model simulating the behavior of the existing structure

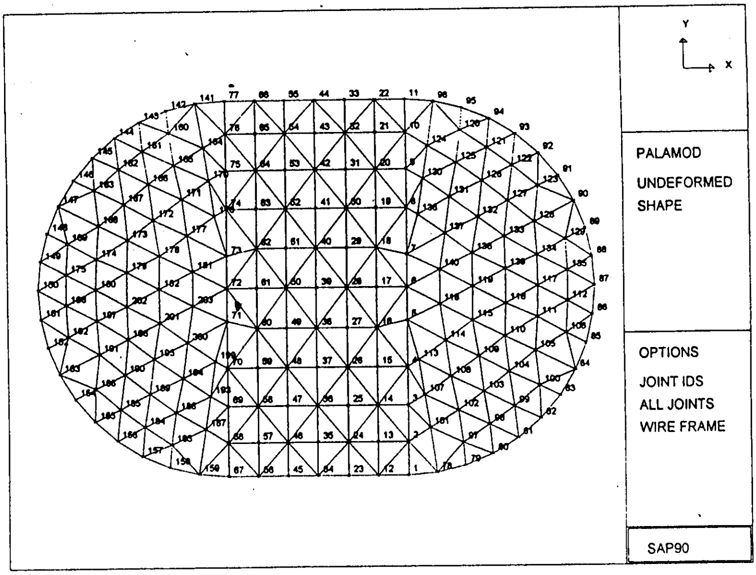

Fig 4 Mathematical model of the structure

Table 2 Tabular presentation of the fundamental frequencies of the prototype and the model

		Fundamental frequencies (Hz)	
		Asymmetrical mode shape	Symmetrical mode shape
Prototype	experimental	1.65	1.90
	analytical	1.75	1.94
Model	experimental	9.60	11.20
	analytical	9.68	10.74

The mathematical model was estimated in respect to the available experimental data. Namely, the mathematical model related to the existing structure, was verified by comparison of the first two resonant frequencies and mode shapes (Table 2). Spatial presentation of the characteristic mode shapes of the prototype structure is given in Fig. 5. Mathematical model related to the reduced scale physical model, was verified not only by comparison of the resonant frequencies and mode shapes (Fig. 6), but also by comparison of the seismic response of the models at characteristic points, for the same earthquake record. For example, the time history response at node 41, was compared with the shaking table response of the reduced scale model at the corresponding point 4 (Fig. 7). The agreement of the time histories and the Fourier amplitude spectra is considered as satisfactory.

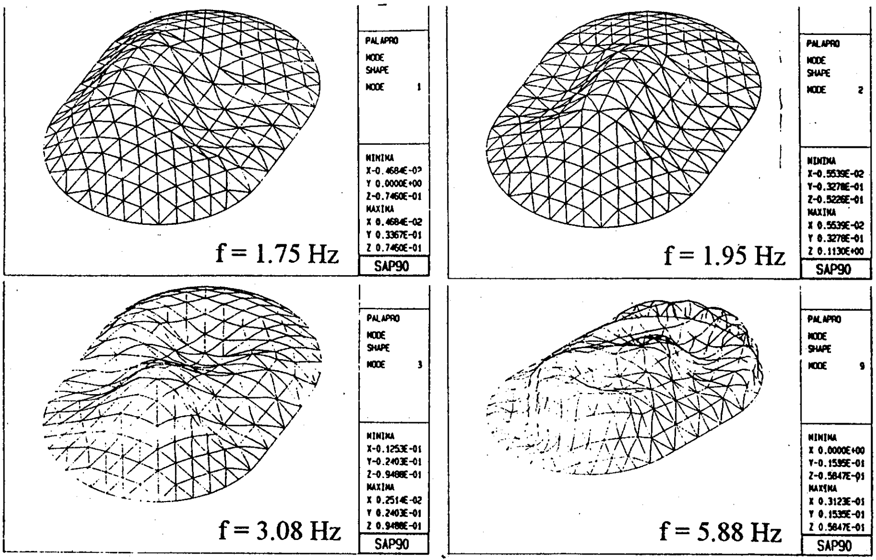

Fig 5 Spatial presentation of characteristic mode shapes of the prototype structure

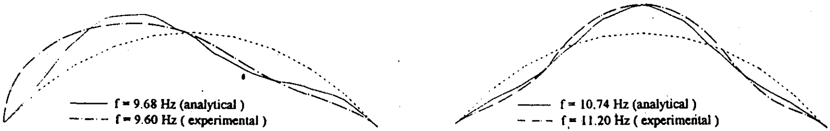

Fig 6 Mode shapes of the model determined analytically, i.e. experimentally

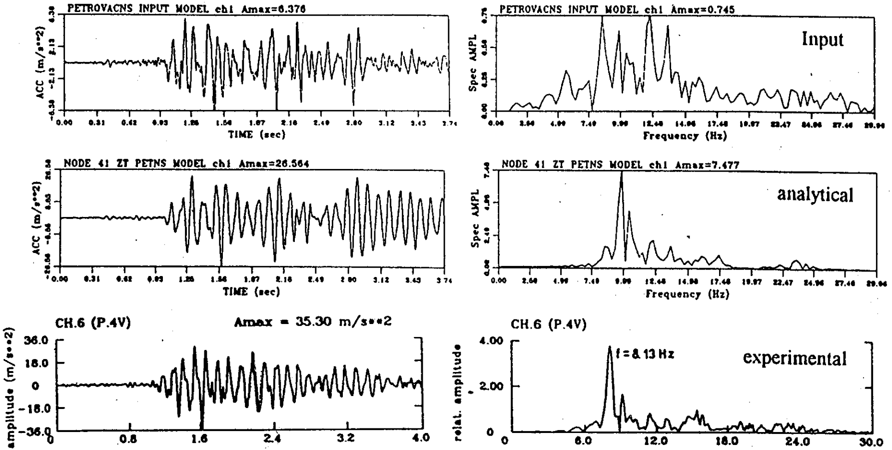

Fig 7 Time histories and corresponding Fourier amplitude spectra of acceleration of the model under Petrovac earthquake in transversal direction

4. SEISMIC RESPONSE OF THE EXISTING STRUCTURE UNDER EXPECTED GROUND MOTION AT THE SITE

The verification of the reliability of the considered simple mathematical model has been discussed in the previous chapter. It was concluded that this model could satisfactorily represent the actual behavior of the existing structure.

Based on the previous experimental and analytical results, it was found out that the cylindrical part of the roof structure is more flexible than the spherical ones.. Consequently, considering the frequency content of the earthquakes as well as the resonant frequencies of the structure, it is expected that the structure will be excited in the first several modes, which correspond to the vibration modes of the cylinder.

The seismicity of the site, at which the existing structure is located, is not a subject of the investigation in this paper. As an example, it is considered that the expected peak ground acceleration is 0.15g . Considering the Ancona record as representative for the site (Fig. 8), the seismic response of the Sports Palace, in vertical direction, was predicted.. It was found out that the most intensive response is recorded at nodes 17,39,41 and 50, which are located on the cylindrical part of the roof (Fig. 4). The acceleration time history and the Fourier amplitude spectrum at node 39 are presented in Fig. 9. The amplification factors of the response, at the specified points ranges between 1 and 3.3 in respect to the peak ground motion. The tabular presentation of the points in which the acceleration response is most intensive, is given in Table 3.

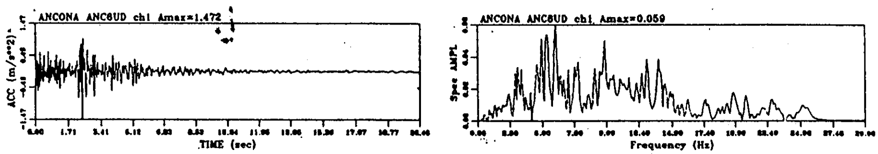

Fig 8 Time history of acceleration and Fourier amplitude spectra for Ancona earthquake

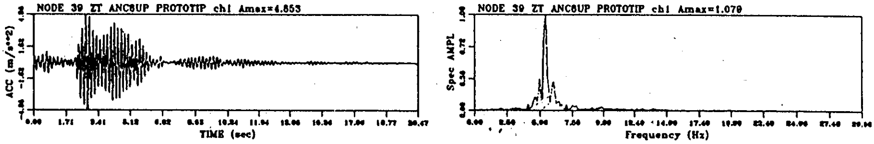

Fig 9 Dynamic response of the structure - acceleration, under Ancona earthquake

Table 3 Acceleration response of the structure in vertical direction obtained by Ancona earthquake (0.15g)

Node	Acceleration (m / sec^2)		Amplification factor
	Input	Response	
17	1.5	4.76	3.24
39	1.5	4.58	3.30
40	1.5	1.22	0.83
41	1.5	1.57	1.07
42	1.5	3.19	2.17
43	1.5	4.02	2.73
50	1.5	4.49	3.05
61	1.5	4.76	3.24
72	1.5	3.62	2.46
182	1.5	3.53	2.40

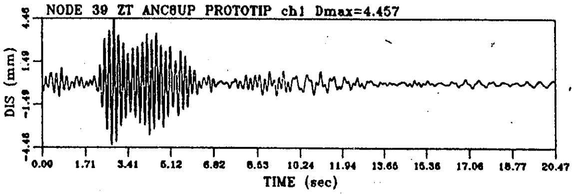

Fig 10 Dynamic response of the structure - displacement, under Ancona earthquake

Table 4 Comparison between static and dynamic displacement of the structure

Node	Vertical displacement (mm)		Dyn ./ Stat (%)
	Static (self weight)	Dynamic Ancona - 015g	
29	21.2	1.5	7.1
39	25.2	4.5	17.9
40	21.1	1.3	16.2
41	13.5	1.9	14.1
42	3.6	2.8	77.8
43	0.01	3.6	-
50	23.2	4.1	17.7
61	21.0	4.4	21.0
72	15.3	3.3	21.6
182	10.4	3.1	29.8

In order to investigate the distribution of the vertical displacements, produced by the expected earthquake, the displacement time histories have been analyzed at different nodes of the cylinder and the spherical parts. The dynamic displacement time history at node 39 is presented in Fig. 10. The peak values have been compared with the static displacements (Table 4). It was found out that the cylindrical part is deflecting more than the spherical parts (both in static and dynamic case). The maximum displacement is obtained in the central part of the cylinder (node 39). The ratio between the dynamic and the static displacement ranges between 7 and 78 %.

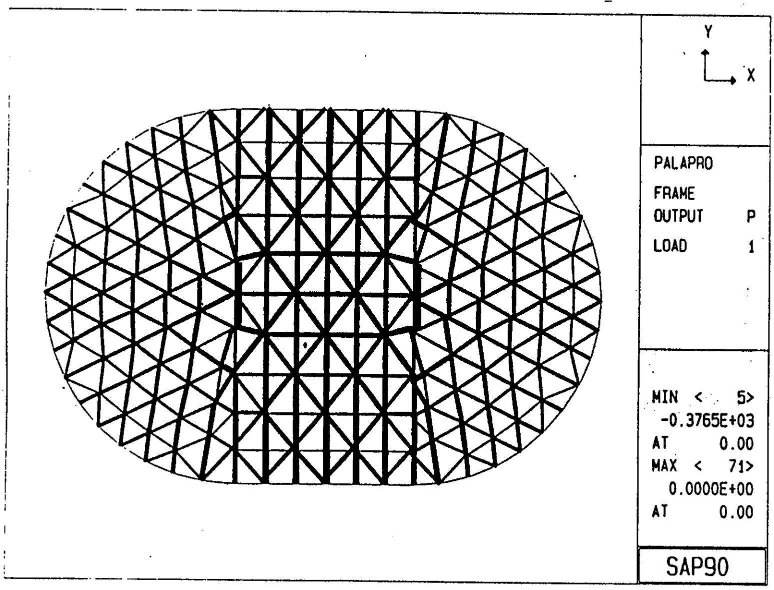

Fig 11 Static force distribution

Table 5 Comparison between static and dynamic forces of the structure

Element	Axial forces (kN)		Dyn ./ Stat (%)
	Static (self weight)	Dynamic Ancona - 015g	
31	329.3	65.0	19.7
32	304.7	68.1	22.4
33	268.5	48.6	18.1
34	256.1	48.5	18.9
35	250.4	43.8	17.4
65	376.5	86.0	22.8
101	194.6	69.9	35.9
102	194.6	71.96	37.0
103	223.6	70.5	31.5
107	256.0	78.3	30.6
108	222.2	65.2	29.3
109	222.2	66.1	29.9

Finally, the force distribution has been investigated in the similar manner. It was found out that larger forces are distributed to the cylinder (Fig. 11). The comparison between the static and the dynamic forces in the most loaded elements is given on Table 5. The dynamic force time history at element 65 is presented in Fig. 12. The highest forces are produced at the contact between the cylinder and the spherical parts (elements 65,66 and 101,102) and at the central arch of the cylinder (elements 38,39). The dynamic/static force ratios ranges between 17 and 37 %.

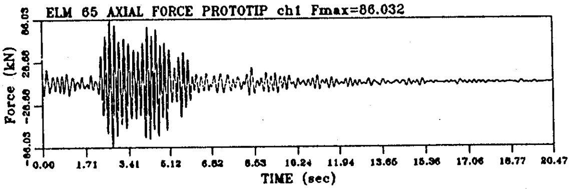

Fig 12 Dynamic force time history at element 65 under Ancona earthquake

5. CONCLUSIONS

The analytical modeling of the structural systems is very important and useful approach for the investigation of the structural behavior under static and dynamic conditions. SAP90 computer programme offers a variety of possibilities for 3D linear analysis, considering different loading and boundary conditions. The graphic options make the analysis very illustrative and easy understandable. However, the lack of experimental data makes the physical interpretation of the analysis difficult, especially in case when some input data errors have been introduced. In this paper, the correlation between the analytical and the physical model in respect to dynamic characteristics and seismic response is satisfactory. The same conclusion could be drawn about the correlation between the analytical model and the existing structure.

REFERENCES

1. Zarri, Lj. Taskov, D. Jurukovski, M. Bojadziev, "Full scale test of the Palasport in Bologna by ambient and forced vibration method", IASS Symposium, Istanbul, March 1993
2. Zarri, Lj. Taskov, D. Jurukovski, M. Bojadziev, "Design and construction of 1/30 scale model of Pala Sport in Bologna", Conference of space structures, Guildorf-England, September 1993
3. Zarri, Lj. Taskov, D. Jurukovski, M. Bojadziev, " Shaking table test of 1/30 scale model of sports palace- Bologna with wooden covering shell structure- Model 1", 10th European Conference on Earthquake Engineering, Vienna, Austria, August 1994