DYNAMIC PROPERTIES OF FOURTEEN STORY R.C. FRAME BUILDING FROM FULL SCALE FORCED VIBRATION STUDY AND FORMULATION OF MATHEMATICAL MODEL

by

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SYNOPSIS

More than twenty five fourteen story reinforced concrete frame buildings of the same type of structure were constructed according to the program of rebuilding of the down town area of Skopje. In order to check out assumption made for their seismic stability analysis, a full scale forced vibration study was carried out on one of these buildings.

In each of the principle directions of the building the three lowest translational modes were excited. Representative frequency response curves and mode shapes for E-W direction are shown in the figures and values of the resonant frequencies and structural damping coefficients for both directions of all excited modes are given in tables.

Using dynamic properties obtained from the test, effort for mathematical model formulation was made. The comparison of the analytical and experimental results shows good agreement between them.

1. INTRODUCTION

In order to check out the assumptions made in the seismic stability analysis of more than twenty five fourteen-story reinforced concrete buildings in Skopje, full scale forced vibration study of one of them was made. The building is 22,2 by 22,2m. in plan

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and 47.0m high with grill type of foundation laying on dense gravel. Partition walls in the building are constructed of hollow brick in lime-cement morter.

Vibrations of the building in two principal directions were excited by rotating mass vibrator attached on the top floor. In order to determine frequency response curves, frequencies, mode shapes and damping coefficients corresponding to each of the modes, twenty tests have been carried out in both principal directions. For the three lowest translational modes the following frequencies were obtained: E-W direction f_1=1.01 cps, f_2=3.12 cps, f_3=5.58 cps; N-S direction f_1=1.10 cps, f_2=3.40 cps and f_3=6.67 cps.

Representative response curves for different level of excitation and mode shapes for E-W direction are given in figures 3 to 8. The results of frequencies and damping coefficients under different level of excitation for both principle directions are given in Table 1.

Analysing the building as a shear frame structure, effort for mathematical model formulation was made. Using experimental data for the first three modes three diagonal stiffness matrix is obtained . The damping matrix is obtained as linear combination of stiffness and mass matrices(6). Using the formulated mathematical model good agreement is obtained by comparison of the experimental and the theoretical frequencies and mode shapes.

2. DESCRIPTION OF THE BUILDING, VIBRATION EXCITER AND INSTRUMENTATION

The tested building is reinforced concrete frame structure 22,2 by 22,2 m. in plan and 47.0 m. high with grill type of foundation laying on a dense gravel 3.50 m. from the ground surface.

The structural system of the building is a space frame with same geometrical properties in both orthogonal directions. The cross section of the columns is changing according to the height from 65/65 cm. at the ground floor to 40/40 cm at the upper stories. The beams are of constant depth of 45 cm. and the height of the stories is changing from 2.90 m. to 3.10 m. The floor structure is a coffered one with dimensions of the coffer 90/90cm. The total height of the coffered structure is 40 cm. with a slab of 6 cm. and ribs 7 cm. thick. A plan of the typical floor and the cross section of the building are shown on figs. 1b and 2 respectively. All external and partition walls are constructed of hollow brick, strengthened by horizontal shallow R.C. beams. The thickness of external walls is 25 cm. and that of partitions 12 to 25 cm.

The vibration exciter located on the roof(fig la) uses counter-rotating weights so the inertia forces will be addative in one direction while they will cancel each other in orthogonal direction. Changing the loads in the baskets the force produced by the vibration exciterscould be changed according to the required level of excitation by the test program. The maximum force produced by the exciter is 2.250 kg. The frequency control of the vibration exciter is extremely accurate, with an accuracy of about 0.1%. Detailed description of the vibration exciter is given by Hudson(2).

During the tests four fluid damped accelerometers A4-0.25-350 with natural frequency 15 cps and 0.7% damping were used. Signals from the accelerometers were amplified by an amplifier Model 131-2c and recorded by Visicorder 906T with galvanometers M24-350. More detail description of the instrumentation used is given under(7).

3. EXPERIMENTAL RESULTS

In order to obtained the frequency response curves, frequencies, mode shapes and damping, a total of twenty steady-state vibration tests were carried out. In both principal directions of the building the three lowest translational modes were excited at various force levels. First, frequency response curves were taken on the roof, and when they were established, the mode shapes at resonance were determined. The amplitudes on each of all fourteen floors were measured along the same vertical of the structure. Non-normalized frequency response curves for the first three modes of translational vibration in the E-W direction are shown on figs. 3, 4 and 5 for various levels of excitation, and the results for frequencies and damping coefficients are given in the figures and Table 1 for E-W as well as N-S direction. The mode shapes for the lowest three modes in E-W direction are given on figs. 6, 7 and 8. During the test in E-W fundamental mode of vibration the maximum acceleration was 5.8×10^{-3} g and the maximum displacement was 0.15 cm. both observed on the roof.

3.1. Resonant frequencies

The resonant frequencies obtained from the frequency response curves given in Table 1 are showing that for the first mode of vibration they are almost same for all levels of excitation and for the second mode they have a tendency of decreasing by increasing the force of excitation. This indicated that the stiffness of the structure under initially low excitation was larger and it changes by increasing the excitation force. Also, this can be observed from the softening effect obtained on the frequency response curves (fig.4). The same effects are obtained for both orthogonal directions.

As it was mentioned before, the frame structure is absolutely simmetrical and with completely same geometry in both orthogonal directions. From the results given in Table 1 it can be seen that the frequencies of all these modes in N-S direction are about 10% bigger than that in E-W direction. This is due to the effect of the different of the infilled walls and the position of the staircase with elevator shafts. Also it is notable on the frequency response curves for the second and the third mode that there is some torsional effect due to the same reason mentioned above. Because of the torsional effect obtained, particular check of the displacement on the roof along the length of the building was done, using four accelerometers in the orthogonal direction of excitation. It was founded that there is some coupling of the translational and the torsional vibration for the second and the third mode which was not the case for the first one.

3.2. Mode shapes

After the resonant frequencies were established, the mode shapes were taken, measuring the amplitudes at resonant conditions on each floor along the height of the building. The mode shapes were not affected by changes in the frequency of excitation in the frequency range near the resonance. The deformation pattern obtained for the first mode shape shows that shear deformations are dominating along the height of the building. From figs. 6, 7 and 8 it is notable that there is some participation of the translational and rotational deformations of foundation which are 1.5%, 5.7% and 5.2% for the first, second and third mode respectively.

3.3. Damping

The high degree of accuracy in frequency control during determination of response curves for all tests enables obtaining of accurate values of damping. The damping obtained by band-width method from normalized frequency response curves is probably a combination of several types of energy absorbtion.

Since the total damping in the structure is resonably small, complex damping can be treated as equivalent viscous damping.

From table 1 it can be seen that the values of damping for all three modes are in the range of 1-2% and that there is almost no influence of amplitudes change on the damping for the first mode and a rather small influence for the second and the third mode.

4. FORMULATION OF MATHEMATICAL MODEL

Assuming that the structure is presented as a lumped mass parameter system and that the floor slabs are rigid, the system can be represented as shear frame structure in which case the stiffness and damping matrices will be three diagonal.

Using the above assumptions and the experimental results obtained from the vibration tests, mathematical model of the structure was determined. The formulation of the mathematical model consisted of adjustment of the stiffness, mass and damping characteristics until the model's response to harmonic excitation matched that of the real structure. The estimated masses and the stiffness were adjusted in order to match the resonant frequencies and mode shapes of the model and the prototype, while tha damping matrix has been obtained by linear combination of mass and stiffness matrices following Coughey's procedure(6). The obtained stiffness and damping matrices are presented on the following figures.

5. CONCLUSIONS

The investigations represented in this paper enabled a rather accurate determination of the dynamic characteristics of fourteen story reinforced concrete frame structure although the excitation level was rather low. The presented mathematical model is on the first step of its formulation. The theoretical research continues considering shear and bending deformations as well as soil-structure interaction effect. Further experimental investigation of the non-linear properties of the structural elements has been planned in order to get the properties for formulation of non-linear mathematical model.

This year, the tested building has to be instrumented with three strong motion accelerographs in order to obtain response of the structure during future strong earthquakes.

With completion of all experimental and theoretical research better understanding of the response of reinforced concrete frame structures will be obtained.

ACKNOWLEDGEMENTS

The authors are grateful for the assistance of their collaborators from Dynamic Testing Laboratory for their participation in the tests performance.

Appreciation is extended to the Institute of Town Construction of Skopje for the financial support of this study.

BIBLIOGRAPHY

1. Bouwkamp, J. G., Dixon, Rea, "Dynamic testing and formulation of mathematical models", Chapter VIII, from Earthquake Engineering, John Wiley, 1970.
2. Hudson, D. E. (1962), "Synchronized Vibration Generators for dynamic tests of full-scale structures", Pasadena, Earthquake engineering research laboratory, California Institute of technology.
3. Hudson, D. E. (1964), "Resonance testing of full-scale structures", Proc. ASCE, J. Engr. Mech. Div., 92(EMS).
4. Nielsen, N. N. (1964) "Dynamic response of multi-story buildings", Pasadena: Earthquake engineering research laboratory California Institute of echnology.
5. Kuroiwa, J. H., "Vibration tests of multi-story building", Pasadena, California, June, 1967.
6. O'Kelly, M. E. J. "Vibration of viscously damped linear dynamic systems", Pasadena, California, May, 1964.
7. Petrovski J. Jurukovski D., Percinkov S., "Dynamic properties of multi-story frame building and formulation of mathematical models". (Original in Macedonian) IEE and ES, University of Skopje, May, 1971. Publ. 23.
8. Petrovski J., Jurukovski D., Percinkov S., "Dynamic properties of a six story reinforced concrete building with eccentric and side staircase". (Original in Macedonian). IEE and ES, University of Skopje, September, 1971. Publ. 24.
9. Petrovski J. Paškalov T., Jurukovski D., Percinkov S., "Dynamic properties of a multi-story shear wall building", (Original in Macedonian) IEE and ES, University of Skopje, December, 1970, Publ. 22.
10. Shapiro, G. A., and others, "Isledovanie nelinearnoj raboti konstrukcij zilih i opstestvenih zdanij s pomoscu moscnih vibrationih masin", GKGSA, Gostroj, Moskva, 1969.

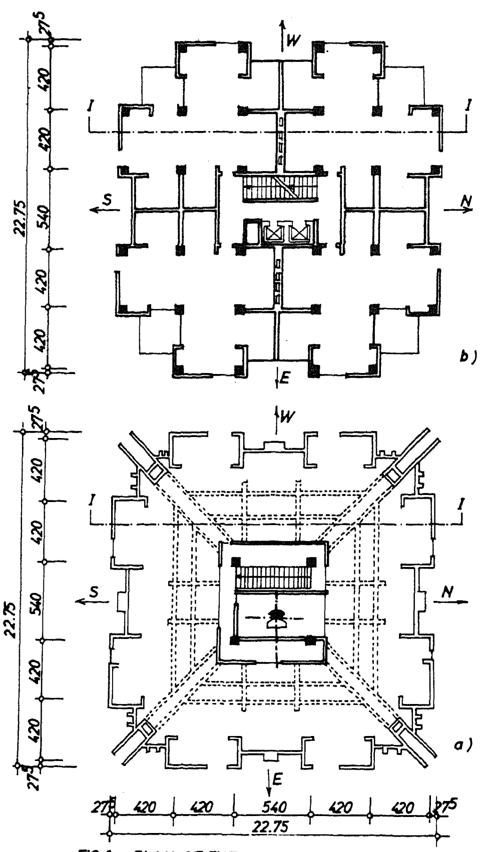

FIG. 1a. PLAN OE THE ROOF WITH THE EXITER LOCATION, 1b. PLAN OF A TYPICAL FLOOR

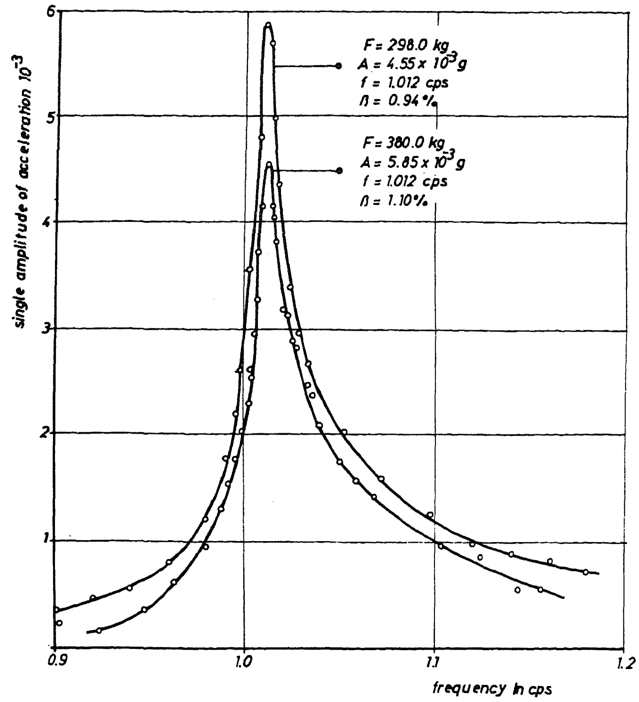

FIG. 3 FREQUENCY RESPONSE CURVES ON THE ROOF FOR THE FIRST MODE

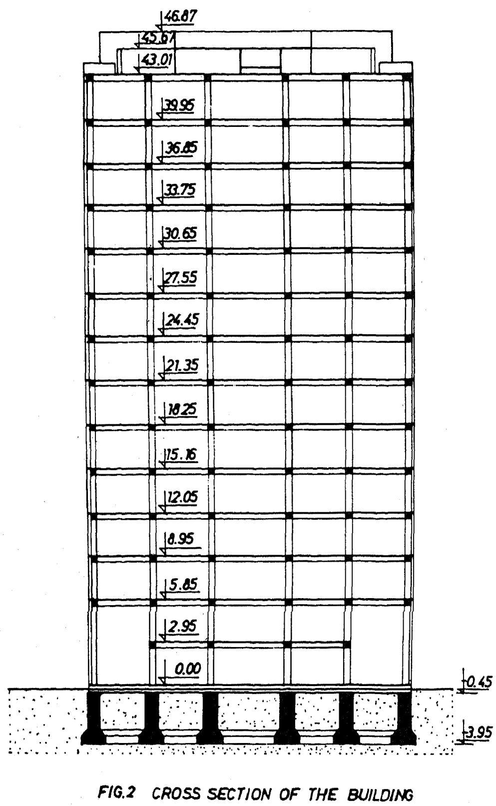

FIG. 2 CROSS SECTION OF THE BUILDING

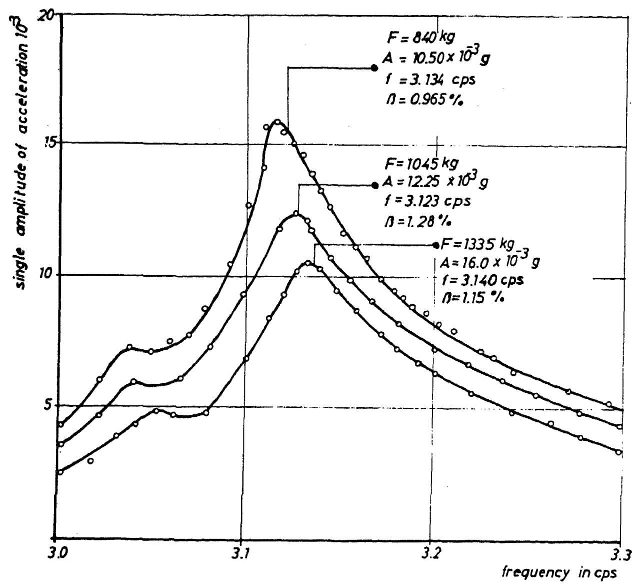

FIG. 4 FREQUENCY RESPONSE CURVES ON THE ROOF FOR THE SECOND MODE

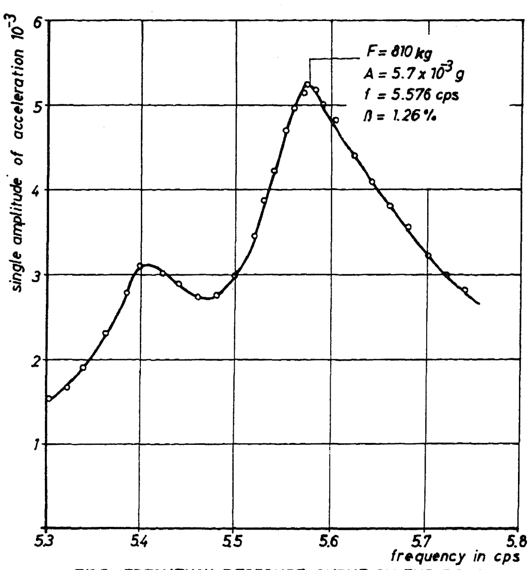

FIG.5 FREQUENCY RESPONSE CURVE ON THE ROOF FOR THE THIRD MODE

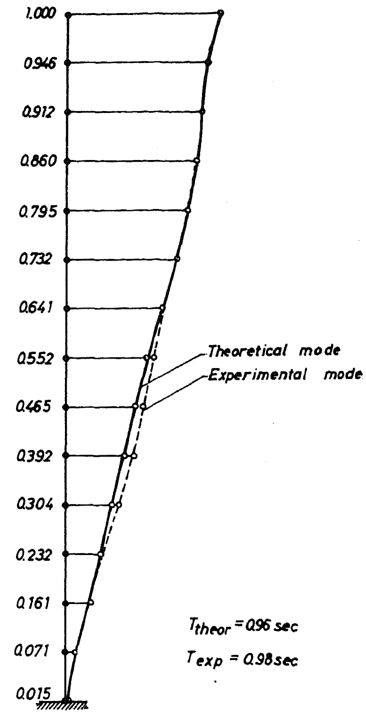

FIG.6 FIRST MODE SHAPE

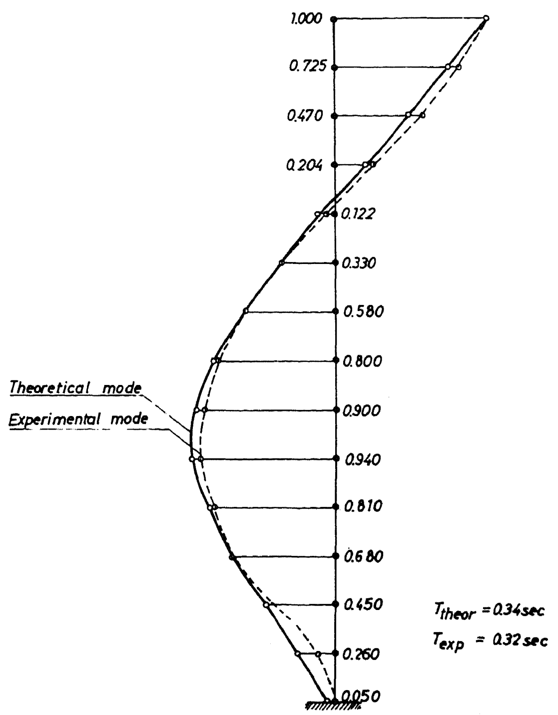

FIG.7 SECOND MODE SHAPE

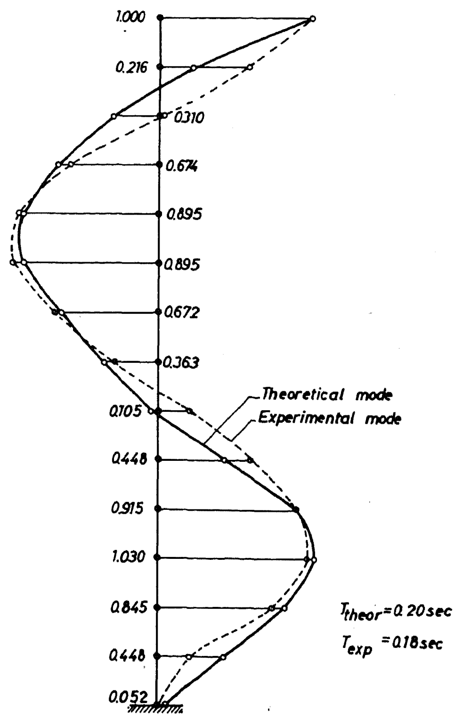

FIG.8 THIRD MODE SHAPE

$$
K = \begin{bmatrix}
3300 & -1400 & & & & & & & & & & & & 0 \\
-1400 & 3600 & -1700 & & & & & & & & & & & \\
& -1700 & 3400 & -1700 & & & & & & & & & & \\
& & -1700 & 3200 & -1500 & & & & & & & & & \\
& & & -1500 & 3000 & -1500 & & & & & & & & \\
& & & & -1500 & 2800 & -1300 & & & & & & & \\
& & & & & -1300 & 2600 & -1300 & & & & & & \\
& & & & & & -1300 & 2500 & -1200 & & & & & \\
& & & & & & & -1200 & 2400 & -1200 & & & & \\
& & & & & & & & -1200 & 2400 & -1200 & & & \\
& & & & & & & & & -1200 & 2250 & -1050 & & \\
& & & & & & & & & & -1050 & 2100 & -1050 & \\
& & & & & & & & & & & -1050 & 2100 & -1050 \\
0 & & & & & & & & & & & & -1050 & 1050
\end{bmatrix}
$$

STIFFNESS MATRIX

$$
C = \begin{bmatrix}
2.999 & -1.260 & & & & & & & & & & & & 0 \\
-1.260 & 3.287 & -1.530 & & & & & & & & & & & \\
& -1.530 & 3.087 & -1.530 & & & & & & & & & & \\
& & -1.530 & 2.915 & -1.350 & & & & & & & & & \\
& & & -1.350 & 2.735 & -1.350 & & & & & & & & \\
& & & & -1.350 & 2534 & -1.170 & & & & & & & \\
& & & & & -1170 & 2364 & -1.170 & & & & & & \\
& & & & & & -1.170 & 2.283 & -1.080 & & & & & \\
& & & & & & & -1.080 & 2.183 & -1.080 & & & & \\
& & & & & & & & -1.080 & 2.183 & 1.080 & & & \\
& & & & & & & & & -1.080 & 2.052 & -0.950 & & \\
& & & & & & & & & & -0.950 & 1.932 & -0.950 & \\
& & & & & & & & & & & -0.950 & 1.932 & -0.950 \\
0 & & & & & & & & & & & & -0.950 & 0.985
\end{bmatrix}
$$

DAMPING MATRIX

Table 1.

Test No	Force at resonans (kg)	Single amplitude at resonans 14 th floor $10^{-3}g$	Frequency (cps)	Damping coeficent (%)
		DIRECTION	E-W	
1.	298.0	4.55	1.012	0.94
2.	380.0	5.85	1.012	1.10
3.	840.0	10.50	3.134	0.965
4.	1045.0	12.25	3.123	1.28
5.	1335.0	16.00	3.116	1.15
6.	810.0	5.70	5.576	1.26
		DIRECTION	N-S	
7.	335.0	5.60	1.101	1.18
8.	435.0	6.85	1.100	1.18
9.	542.0	8.30	1.105	1.22
10.	1056.0	15.60	3.436	1.14
11.	1260.0	19.25	3.404	1.28
12.	1540.0	24.20	3.398	1.18
13.	1015.0	7.40	6.374	1.22

DINAMIC PROPERTES OF THE BUILDING OBTANIED FROM STADY–STATE VIBRATION TESTS