DYNAMIC BEHAVIOR OF AN ELEVEN STORY MASONRY BUILDING

by

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SYNOPSIS

The results of a forced vibration study of an eleven story reinforced masonry structure are compared to a dynamic analysis. The experimental work determined the natural frequencies, mode shapes, and damping values. Considerable flexibility of the foundation was noted in the experimental studies. The analytical model was developed with both a fixed and flexible base. Experimental and analytical resonance data are compared.

INTRODUCTION

The design of multistory structures subjected to dynamic forces resulting from foundation motions require a consideration of both the characteristics of the ground motion and the dynamic properties of the structure. The availability of high speed digital computers and the sophistication of the idealization of structures and the computer model formulation of the structure have made available the elastic, and in certain structural systems, the inelastic response of structures when subjected to earthquakes. However, the accuracy of the results in large measure depend upon the computer model formulation of the structure and its foundation. In order to determine the accuracy of the calculated results and to accumulate a body of information on the dynamic properties of structures, especially when these structures have novel design features, a number of dynamic tests have been conducted on full-scale structures (1).

For the above reasons a dynamic test was performed on the Oak Center Towers, Oakland, California (2).

DESCRIPTION OF BUILDING

The Oak Center Towers is an eleven story structure located in Oakland, California. The 100 foot high building has an overall plan of 85 feet by 200 feet. The building is offset in the middle by approximately 16 feet so that it does not have a pure rectangular plan. It is constructed with reinforced concrete block shear walls and prefabricated prestressed concrete slab elements. The elevator shaft is located in the center south section of the structure with stairwells at either end. Figure 1 shows the east elevation of the building.

The building is designed as a housing development for the elderly and is therefore modular in concept. The building is serviced by two elevators in the center south section of the structure. Stairwells are located on either end of the building. Figure 2 shows a typical floor plan for the second through eleventh floors.

The vertical and horizontal load carrying systems are reinforced concrete masonry shear walls in both the transverse and longitudinal directions. These walls rise from the first floor and run up to the roof except in one section on the south end of the first floor where the dining

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room is located. In this location the walls terminate at the second floor and a reinforced concrete frame system carries the loads to the foundations.

The foundations are in general spread footings under each of the walls from 4 feet to 6 feet in width and eighteen inches thick.

The compressive strength of the masonry unit is 3000 psi below the eighth floor and 2000 psi above the eighth floor. All of the cells in the masonry were grouted with 4000 psi hard rock concrete.

The transverse walls are made up of eight inch wide blocks from the first floor to the roof. The longitudinal walls, which basically run down each side of the corridor, are twelve inch wide block up to the fifth floor and eight inch block from the fifth floor to the roof.

The minimum reinforcement consisted of two number 4 bars at 24 inches on centers vertical and the same horizontal for the twelve inch block and in the eight inch block one number 4 bar at 24 inches on center, both vertically and horizontally. Special reinforcement is added at wall ends, corners and where two walls connect. This consisted mainly of number 8 bars up to the eighth floor then number 6 bars from the eighth to the tenth floors and number 5 bars from the tenth floor to the roof.

The floor system consists of precast prestressed planks 6 inches deep and 40 inches wide spanning between the transverse walls. A 2 inch lightweight concrete topping is placed over these planks.

EXPERIMENTAL PROGRAM

Forced vibrations were produced by two rotating-mass vibration generators or shaking machines mounted on the 11th floor of the building and oriented so as to induce the maximum forces in the East-West and North-South directions as shown in Figure 2. A complete description of the vibration generators is given elsewhere (1,3).

The transducers used to detect horizontal floor accelerations of the building were Statham Model A4 linear accelerometers, with a maximum rating of ± 0.25g. The electrical signals for all accelerometers were fed to amplifiers and then to a Honeywell Model 1858 Visicorder. For the translational motions the accelerometers were located near the center of the floor and oriented so as to pick up the appropriate East-West or North-South accelerations. For recording the torsional motion accelerometers were properly oriented near the north and south ends of the building. To determine the resonant frequencies of the building the accelerometers were located on the 11th floor. In addition vertical accelerations were taken at 6 locations on the 1st floor to determine the foundation motion. The mode shapes were evaluated from records taken at all of the floors including the roof.

MATHEMATICAL MODEL

A mathematical computer model of the Oak Center Tower Building was formulated to assess its dynamic characteristics. The model was formulated using both a rigid base and a flexible base. TABS, a general computer program developed by the Division of Structural Engineering and Structural Mechanics of the Department of Civil Engineering at the University of California, Berkeley, was used to calculate the frequencies and mode shapes of the building. A complete description of this program is given in reference (4).

The program considers the floors rigid in their own plane and to have zero transverse stiffness. All elements are assembled initially into planar frames and then transformed, using the previous assumption, to three degrees of freedom at the center of mass for each story level (2 translational, 1 rotational). Coupling between independent frames at common column lines is ignored. The basic model of the building was formulated as a system of independent frames and shear wall elements interconnected by floor diaphragms which were rigid in their own plane and fixed at the 1st floor level.

The story masses are obtained from the approximate dead loads per floor. These lumped weight values include the floor slabs and masonry walls for each floor level.

During the experimental phase of the work significant vertical motion was recorded at the first floor level in the building. Therefore as a second basic approach the model was allowed to have a flexible base.

Based on the measurements of the ground accelerations at the first floor of the building the following approach was used.

It was assumed that the accelerations measured were due primarily to first mode response. As such, a first mode shape was assumed and acceleration values at each mass point computed. These were used to calculate an effective overturning moment which when compared with the measured ground accelerations allowed an assessment of the base rotational and translational stiffness. An additional basement story was added to the structure with the stiffness values as determined above assigning to the elements as a means of modeling the rotational and translational flexibility.

RESULTS

In the forced vibration tests, two translational modes in the East-West and North-South directions were excited, as well as, the one torsional mode. Typical frequency response curves in the region of the resonant frequencies are shown in Figure 3. The typical vertical and horizontal modes shapes are shown in Figure 4. The resonant frequencies and damping factors evaluated from the experimental data along with the analytical results are summarized in Table 1.

CONCLUSIONS

In comparing the forced vibration as well as the analytical solution it is noted that there is reasonable agreement in the first two modes, however, this does not hold as true for the higher modes. The analysis indicated that in the first E-W mode there was a significant contribution of torsion in this mode. This is also noted in the forced vibration study where the first E-W mode and the first torsional mode (2.78 and 2.83 cps, respectively) were very close together.

The predominant feature which came out of the analytical solutions was the effect of the foundations on the response of the structure. The fundamental frequency was almost half for the flexible foundation as for the rigid foundation (2.45 versus 4.13 cps, respectively). The analysis of very rigid structures on flexible foundations must consider the soil-structure interaction phenomena or the solution could be as much as 100 percent off.

It is apparent that for structures where the in-plane stiffness of the floor system is less or comparable to the stiffness of the lateral load resisting system, the assumption that the floors are rigid in their own plane does not seem to hold true.

This same response regarding the flexibility of the foundation and the in-plane bending of the floor system was also noted in some recent forced vibration studies carried out on a building in Sarajevo, Yugoslavia (5).

The damping values determined from the forced vibration studies varied from about 2 percent to almost 9 percent. The higher damping values could be due to the flexibility of the foundations and their contribution to the response of the building.

TABLE 1 COMPARISON OF RESONANT FREQUENCIES AND DAMPING RATIOS

EXCITATION	MODE							
	1				2			
	FORCED VIBRATION		ANALYSIS		FORCED VIBRATION		ANALYSIS	
	FREQ cps	DAMP %	FIXED BASE	FLEX BASE	FREQ cps	DAMP %	FIXED BASE	FLEX BASE
E-W	2.78	6.4	4.50	2.45	5.82	2.1	14.49	4.13
N-S	3.30	8.8	4.98	2.98	5.93	2.8	5.08	--
Torsional	2.83	2.6	--	--	--	-	--	--

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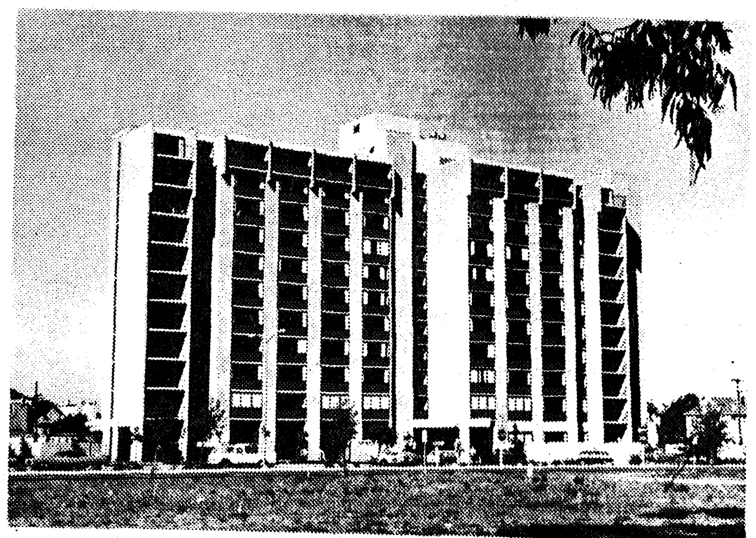

FIG. 1 OAK CENTER TOWERS, OAKLAND, CALIFORNIA
(EAST ELEVATION)

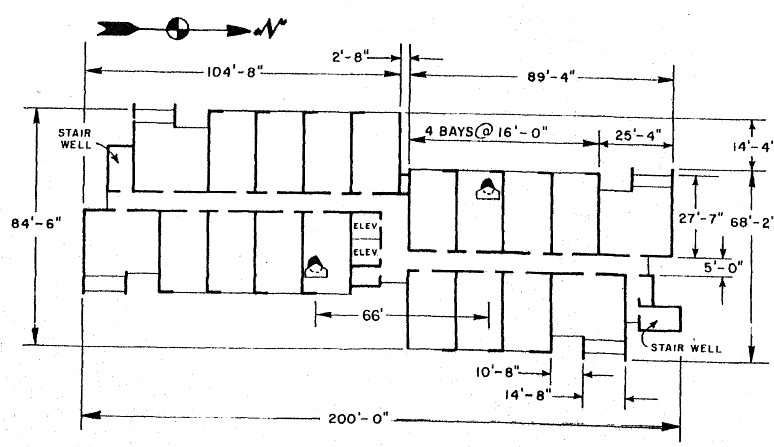

FIG. 2 TYPICAL FLOOR PLAN

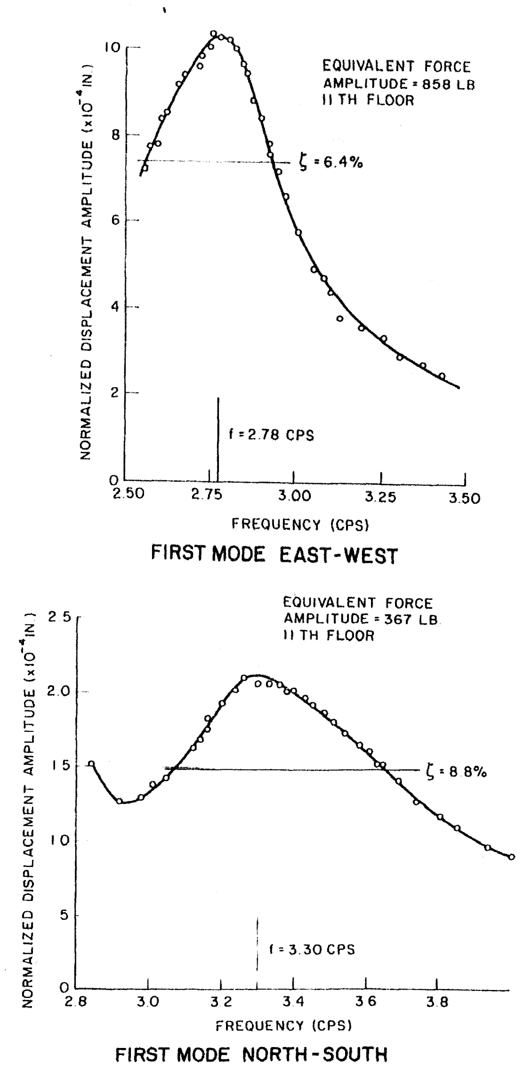

FIG. 3 TYPICAL FREQUENCY RESPONSE CURVES

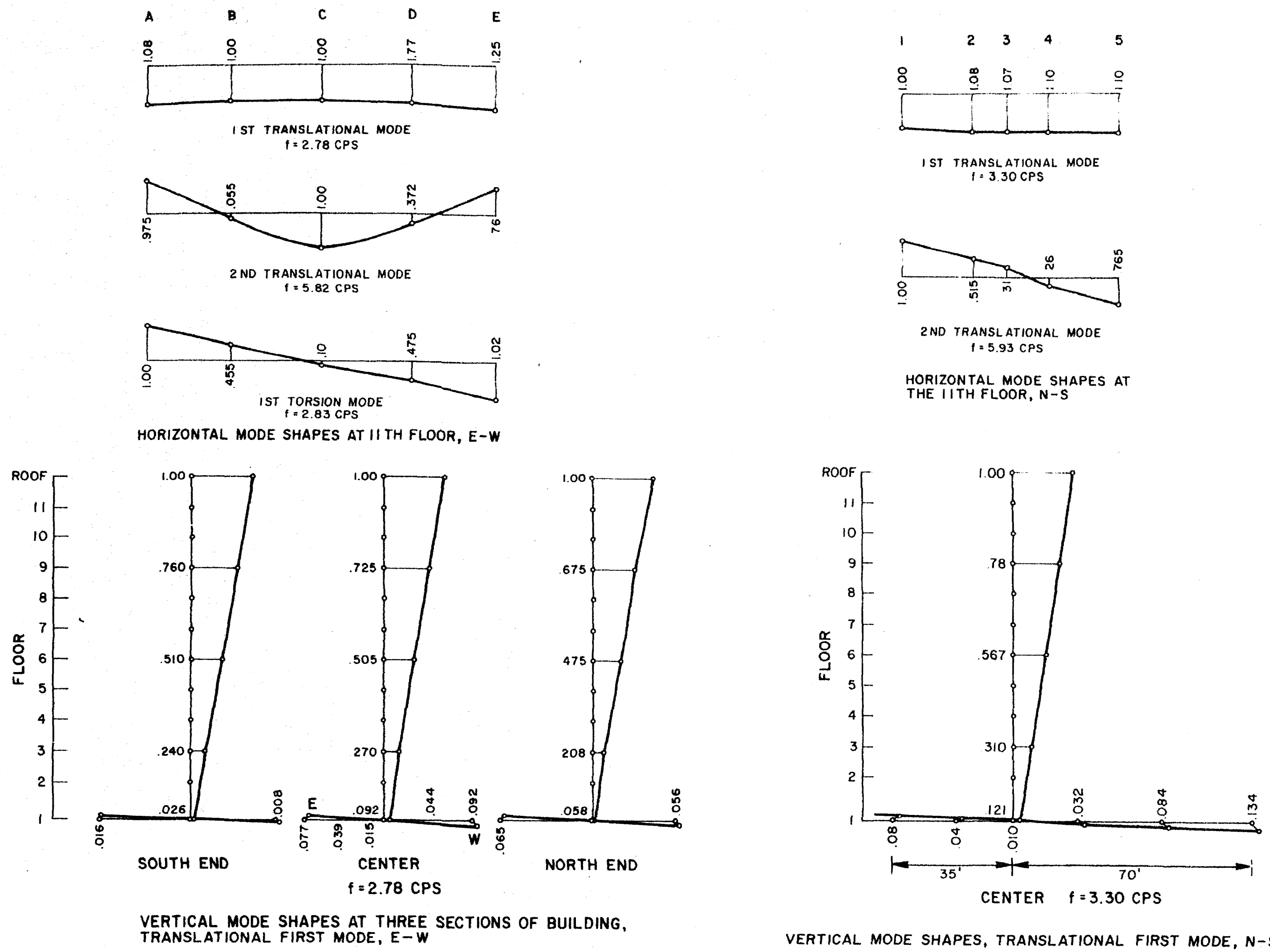

FIG. 4 TYPICAL MODE SHAPES