ASSESSMENT OF THE SOIL-STRUCTURE INTERACTION EFFECTS ON PREFABRICATED STRUCTURES

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SUMMARY

This paper describes the results of experimental and theoretical studies of the soil-structure interaction effects on two prefabricated apartment buildings. The experimental findings are correlated with theoretical analysis based on a generalized three-degree-of-freedom model and are compared with the pertinent stipulations of the ATC-3 Seismic Regulations (ATC, 1978).

INTRODUCTION

Dynamic response characteristics of structures are essential parameters in their earthquake resistant design. For conventional frame type reinforced concrete buildings resting on relatively stiff foundation soils, structures are flexible enough so that soil-structure interaction may be neglected for analytical purposes. In such cases conventional rigid-base idealization yields sufficiently accurate engineering solutions for earthquake resistant design. In cases where the structure is more rigid than usual and/or the foundation soil is medium to soft type, conventional "rigid-base, flexible-structure" type an idealization should not be expected to yield a satisfactory solution for dynamic response. Panel-prefabricated buildings resting on soil foundations are good examples exhibiting strong soil-structure-interaction phenomena.

For the last two decades a number of full scale dynamic tests on structures have addressed to the problem of the assessment of the dynamic characteristics of such structure-soil systems (Öner and Erdik, 1980; Bouwkamp and Stephen, 1980; Jurokovski, 1980).

In order to gain information about the dynamic behaviour of two concrete panel type prefabricated apartment buildings full scale field tests have been carried out on the 7-story Betonsan-Karşıyaka and 10 story Betonsan-Atatürk Sitesi Buildings located respectively in Karşıyaka, İzmir and Antakya, Hatay, Turkey. These buildings will be respectively termed as the "7-story" and "10-story" structures. The theoretical analysis of the dynamic response of these structures has been carried out on the basis of the model developed by Öner and Janbu(1976) and are reported in Öner and Erdik (1980).

This study will include a review of the experimental studies, the experimental results and theoretical comparisons on the basis of a generalized three-degree-of-freedom approach considered in the "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC, 1978). In the conclusions a critical review of the pertinent findings is provided together with an empirical equation to determine the fundamental period of vibration of such structure-soil systems.

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DESCRIPTION OF THE BUILDINGS AND FOUNDATIONS

The 7-story structure in the Betonsan-İzmir Karşıyaka Apartment Complex has a height of about 21.0 m with overall plan dimensions of approximately 20.0 by 21.0 m as indicated in Fig.1. The 10-story structure in the Betonsan-Antakya Teachers' 2nd Housing Complex has a height of about 30.0 m with overall plan dimensions of approximately 17.0 by 13.8 m as indicated in Fig.2. The "Mischek Fertigbau" structural system is used with vertical and horizontal load-carrying reinforced concrete shear walls in the transverse and longitudinal directions. The foundations are of reinforced concrete mat type. The shear wave propagation velocities in the foundation soil media of the 7-and 10-story structures are respectively 400 and 200 m/s as determined by the in-situ seismic surface refraction tests.

EXPERIMENTAL INVESTIGATIONS

The experimental setup utilized in the study consists of a rotating - mass type vibration generator mounted on the center of rigidity of the second floor from the top of each building (i.e. the 6th and the 9th. floors). The directions of the excitation are indicated on the typical floor plan of the buildings given in Figs.1 and 2. Strain gage type linear accelerometers are utilized as transducers to measure the horizontal and vertical accelerations on each floor and on the foundation mat. Detailed description of the test set up can be found in Erdik et.al. (1981). For the theory of such forced vibration tests the reader is referred to Wiegel (1970). Frequency-response curves were determined by changing the frequency in small steps and recording the structural response at each step. The resonance points are determined on the basis of the amplitude maxima and phase changes. The equivalent viscous damping is determined from the normalized frequency-response curves on the basis of the "Half-Power Width" method. The modal vibration shapes are determined on the basis of the maxima of the frequency-response curves obtained at each floor and at the foundation for the same loading. The dynamic excitation frequency are only considered to cover the first mode of vibration where the soil-structure interaction effects are expected to dominate the response.

Typical frequency-response curves for the 7.and 10-story structures are provided respectively in Figs.3 and 4. In these figures the horizontal axis denotes the frequency of excitation and the vertical axis denotes the average floor acceleration divided by the square of the frequency of excitation, to account for the frequency dependent harmonic excitation force. The tables on these figure yield the first mode frequency of vibration and the associated equivalent viscous damping values. As it could be seen the 7-story structure exhibits approximately 3.9 Hz and 15% as the fundamental frequency of vibration and the damping ratio and the 10-story structure exhibits approximately 2.84 Hz and 1.7% for the same. The frequency-response curves for the foundation mats of the 7-and 10-story structures are provided respectively in Figs.5 and 6. In Fig.5 the vertical axis represents the rocking angle of the foundation mat corresponding to the various excitation force levels as indicated on the figure. The rocking angle and the lateral displacement of the foundation mat of the 10-story structure are represented as the vertical axis of the Fig.6 for different excitation force levels.

The data represented in Figs. 3, 4, 5 and 6 are further utilized to obtain the rocking and lateral stiffness characteristics of the foundation media for the 7-and 10-story structures. For this purpose the total shear force and the overturning moment at the foundation level, obtained at each excitation frequency on the basis of the measured flood responses, are divided by the measured lateral displacement and the rocking angle of the foundation mat. The foundation stiffnesses together with their standard deviation are summarized in Table I.

TABLE I. FOUNDATION STIFFNESSES

	7- Story Structure		10- Story Structure	
	Rocking 10^{11} N.m	Lateral 10^{10} N/m	Rocking 10^{11} N.m	Lateral 10^{10} N/m
Experimental	$1.2, \sigma = 0.2$	$0.2, \sigma = 0.05$	$0.9, \sigma = 0.1$	$0.8, \sigma = 0.1$
Theoretical (Eq.8,10)	1.9	0.5	1.8	1.3

In Fig. 7 and 8 the experimental first mode shapes respectively for the 7-and 10-story structures are provided. The foundation rocking angles, indicative of the soil-structure interaction effects, are worthy of noting.

THEORETICAL INVESTIGATIONS

For the theoretical analysis of the dynamic response of the two prefabricated apartment buildings a simple three-degree-of freedom generalized structure-soil model will be utilized. The same model has previously been considered by several researchers (i.e.Jennings and Bielak, 1973; Roesset et al., 1973 and Veletsos and Nair, 1975) and has consequently provided the backbone of the relevant stipulations in the ATC-3 Seismic Design Provisions (ATC, 1978). In this model the structure is represented in its generalized fixed base first mode quantities of mass ($\bar{m}$), height ($\bar{h}$), damping coefficient ($\bar{c}$), stiffness ($\bar{k}$) and total rocking moment of intertia($\bar{J}$) and the foundation is represented by its mass (m_b), rocking moment of intertia(J_b), lateral stiffness (K_b), lateral damping coefficient (C_b), rocking stiffness (K_ϕ) and the rocking damping coefficient (C_ϕ). The three degrees of freedom are the lateral deformation of the generalized mass (u), the lateral deformation of the foundation (x_b) and the rocking angle of the foundation (ϕ_b) as indicated in Figure 9. For a ground acceleration of $\ddot{x}_g$ the response of the model can be given by the following equations:

$$m\,(\ddot{u} + \ddot{x}_b + \bar{h}\,\ddot{\phi}_b) + c\dot{u} + ku = -\bar{m}\,\ddot{x}_g \tag{1}$$

$$m\,(\ddot{u} + \ddot{x}_b + \bar{h}\,\ddot{\phi}_b) + m_b\ddot{x}_b + C_x\,\dot{x}_b + K_x\,x_b = -(\bar{m} + m_b)\,\ddot{x}_b \tag{2}$$

$$m\,h\,(\ddot{u} + \ddot{x}_b + h\ddot{\phi}_b) + (\bar{J} + J_b)\,\ddot{\phi}_b + C_\phi\,\dot{\phi}_b + K_\phi\,\phi_b = -\bar{m}\bar{h}\ddot{x}_g \tag{3}$$

The fixed base structure generalized first mode parameters can be determined through following equations (Clough and Penzien, 1975)

$$\bar{m} = (\{q_1\}^T[M]\{I\})^2 / (\{q_1\}^T[(M)]\{q_1\}) \tag{4}$$

$$\bar{h} = (\{h\}^T[M]\{q_1\}) / (\{q_1\}^T[M]\{I\}) \tag{5}$$

$$\bar{k} = \bar{m}\omega_1^2 \quad \text{and} \quad c = 2\xi_1\omega_1\bar{m} \tag{6, 7}$$

where $\{q_1\}$, ω, and ξ, are respectively the fixed base structure first mode shape, frequency and the damping ratio and $[M]$ is the mass matrix. It can be shown that for a uniform shear beam, m is equal to the 81% of the total mass and h is equal to the 64% of the total height.

Veletsos et al.(1971) provide the following expressions to determine the foundation stiffnesses and the damping values :

$$K_X = k_1\ 8V_S^{\ 2}\rho R\ /\ (2-\nu) \quad ; \quad C_X = C_1\ 8V_S\rho R^2\ /\ (2-\nu) \qquad (8,\ 9)$$

$$K_\emptyset = k_2\ 8V_s^{\ 2}\rho R^3/\ (3-3\nu) \quad ; \quad C_\emptyset = C_2\ 8V_s\rho R^4\ /\ (3-3\nu) \qquad (10,11)$$

where R is the equivalent radius of the foundation, ρ is the mass density of the soil, V_s is the shear wave propagation velocity in the soil, ν is the Poisson's ration and k_1, c_1, k_2, c_2 are the frequency dependent coefficients.

The modal characteristics of the vibration can be determined through the eigen-value analysis of Eqns. 1 , 2 . and 3. ATC (1978) provides the following simplified equation for the determination of the first mode frequency of vibration (f_1) for cases here the foundation mass could be neglected.

$$\tilde{f}_1 = f_1\ [\ 1-\ (\bar{k}\ /\ K_x)(1\ +\ (K_x\bar{h}^2\ /K_\emptyset))\ ]^{\ -1/2} \qquad (12)$$

For systems where the structure can be assumed as a rigid body with respect to the foundation media, the foundation has an approximately square plan and the ratio of the apparent building density to the soil density is about 0.5 the Eq.12 can be further simplified to yield the following equation (Erdik et.al, 1981)

$$\tilde{f}_1 = 0.4\ (V_s/H) \qquad \text{for} \qquad (H/a)\ <2 \qquad (13)$$

where H/a is the aspect ratio of the building.

The first mode frequencies of vibration of the 7-and 10-story structures are provided in Table II.

ATC(1978) provides the following equation for the determination of effective damping ($\tilde{\xi}_1$) for the flexibly supported structure :

$$\tilde{\xi}_1 = \xi_o + \xi_1\ (\tilde{f}_1\ /\ f_1)^3 \qquad (14)$$

where ξ_o is the contribution of the foundation damping and ξ_1 is the first mode damping ratio of the fixed based structure. The experimental and theoretical results of the damping ratios for the flexible base structures are also included in Table II.

TABLE II. FIRST MODE FREQUENCIES AND DAMPING RATIOS

	7-Story	10-Story
Theoretical Fixed Base Freq.	12.2 Hz.	4.0 Hz.
Exp.Flexible Base Freq.	3.8 - 3.9 Hz.	2.8 - 2.9 Hz.
Flexible Base Freq.(Eqs.1.2 and 3)	4.0 Hz.	3.1 Hz.
Flexible Base Freq.(Eq.12)	4.1 Hz.	3.3 Hz.
Flexible Base Freq.(Eq.13)	4.2 Hz.	-
Experimental Damping Ratios (Flexible base)	12 - 16 %	1.6-2.6 %
Theoretical Damping Ratios (Flexible Base) (Eq.14)	20 %	5 %

The theoretical mode shapes obtained on the basis of Eqs.1, 2 and 3 are $\{u,\ x_b,\ \bar{h}\ \emptyset_b\}^T = \{1,\ 2.37,\ 4.15\ \}$ and$\{\ 1,\ 0.07, 0.45\}$ respectively for the 7-and 10-story structures. These mode shapes are plotted in Fig.7 and 8.

Eq.13, which approximately provides the first mode frequency of vibration of the flexibly supported rigid structures, indicates that the fundamental period of vibration of (T_1) such structure-soil systems will be directly proportional to the number of stories (N) for given soil groups. In fact, Fig.9, where the frequency data provided by Jurokovski(1980), Bouwkamp and Stephen (1980) and by this study plotted against the number of stories confirms this finding, at least for soft-to-medium stiff soil foundations.

CONCLUSIONS

The results presented clearly indicate that the full-scale forced vibration studies are valuable means of obtaining the low-amplitude dynamic parameters of structure-soil systems.

The response of prefabricated structures supported by medium-to-soft soil foundations is found to be dominated by the soil-structure interaction effects. For the structures considered herein the fundamental frequencies decrease upto 70% and the equivalent damping ratios increase upto 500% compared to the similar but rigidly supported system. The foundation flexibilities contribute upto 90% of the lateral displacements.

As it could be assessed the theoretical findings on the mode shapes and the frequencies of vibration indicate excellent agreement with the experimental findings. However, there exist appreciable differences between the experimental and theoretical values of the foundation stiffnesses and the equivalent damping ratios. The approximate equation provided in ATC(1978) for the determination of the first mode frequency of vibration of an elastically supported structure is faund to be quite adequate.

It should be concluded that the dynamic response of such rigid structures depends strongly on the foundation soil characteristics rather than the structural parameters and details and in the design of such - systems the effects of soil-structure interaction should be properly accounted for.

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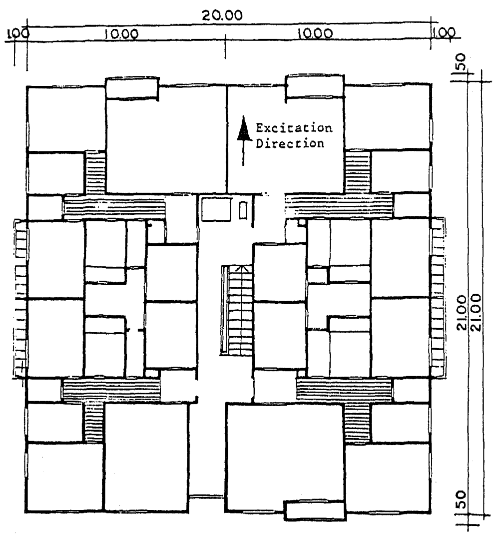

FIGURE 1. Typical Floor Plan for the 7-Story Structure

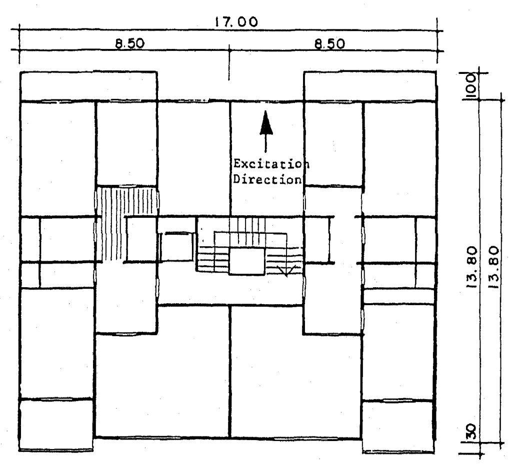

FIGURE 2. Typical Flor Plan for the 10-Story Structure

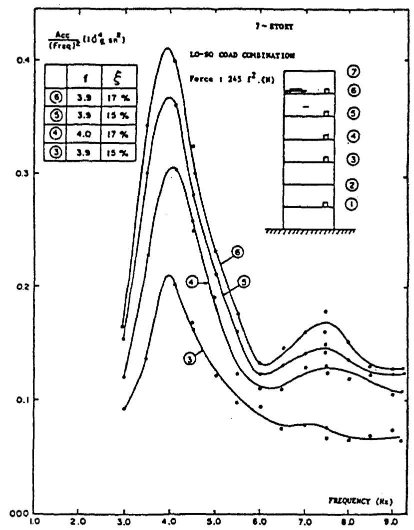

FIGURE 3. Typical Frequency-Response Curve for the 7-Story Structure

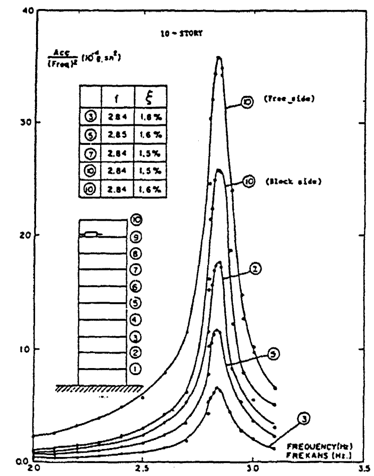

FIGURE 4. Typical Frequency-Response Curve for the 10-Story Structure

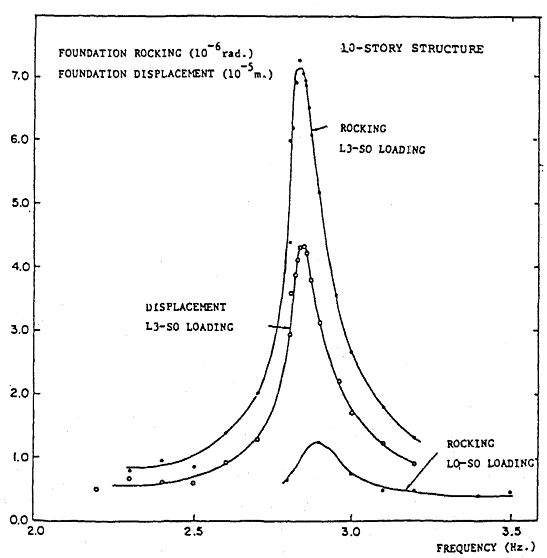

FIGURE 5. Frequency-Foundation Response Curve for the 7-Story Structure

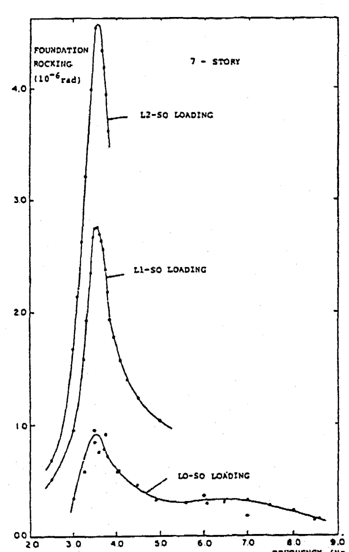

FIGURE 6. Frequency-Foundation Response Curve for 10-Story Structure

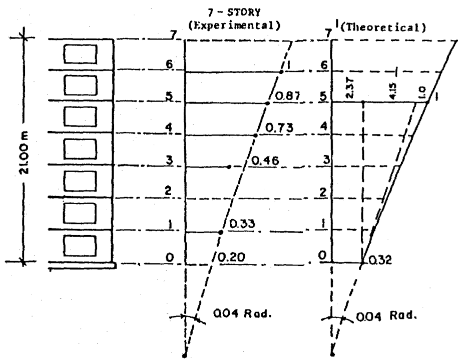

FIGURE 7. First Mode Shapes for the
7-Story Structure

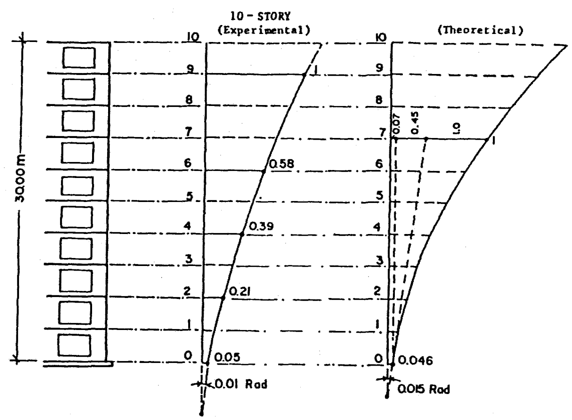

FIGURE 8. First Mode Shapes for the
10-Story Structure

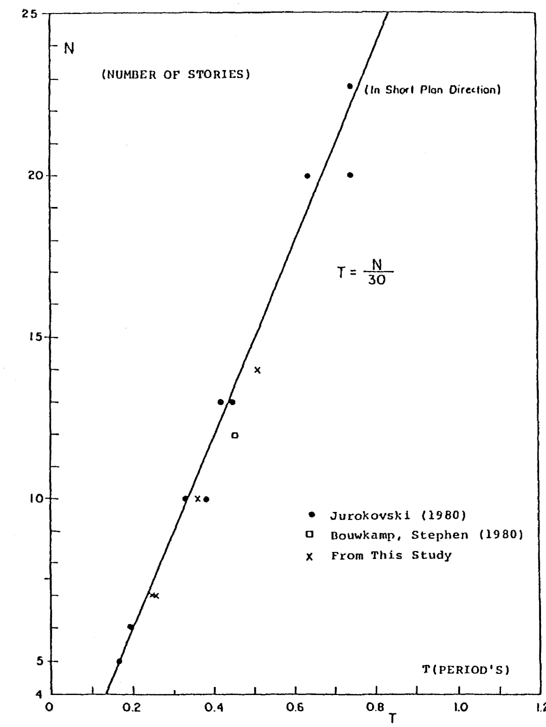

FIGURE 9. Fandamental Period of Vibration of
Flexibly Supported Structures