Experimental Assessment of the Soil-Structure Interaction Effects for a Research Reactor

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Abstract

This paper describes the dynamic characteristics of an embedded research reactor building as revealed by the in-situ forced vibration testing. Eccentric mass vibrators were used to provide steady-state harmonic input to the structure. Foundation stiffnesses resonant frequencies, damping ratios and the mode shapes of the structural system were determined. The experimental results were compared with those obtained on the basis of a simplified two-dimensional three-degree-of-freedom mathematical model.

1. Introduction

Forced harmonic vibration tests were carried out for the determination of the dynamic characteristics of the reactor building of the Turkish Atomic Energy Authority-Çekmece Nuclear Research and Training Center located near İstanbul, Turkey. Such tests have been undertaken in several nuclear power plants in U.S.A., France, West Germany and Japan (e.g. Hornbuckle et al., [7], Hart and Ibanez, [6], Masao et al., [10] and Ibanez et.al.,). The forced vibration tests conducted on several structures in Turkey can be found in Erdik and Gülkan [4]. The steady-state forced vibration test uses one or more structural vibrators placed at appropriate points on a structure. The response of the structure is measured at points of interest for different frequencies and forces of excitation. To interpret the experimental results, one compares the experimental response curve to theoretical response curves obtained from the analysis of an appropriate mathematical model of the structure.

2. Reactor Building

The reactor building is a reinforced concrete frame structure with brick and concrete block infill walls. As it can be seen in Fig.1 it has an overall plan dimensions of 23.54m x 32.58 m, an under ground depth of 9.12 m and an above ground height of 11.25 m. The structure is founded on a thick base mat, encircled by peripheral walls also acting as retaining walls against the soil pressure. The above ground enclosure structure consists of one-way frames supported directly on the peripheral wall. The underground part of the structure consists of three levels identified on the Fig.1 as base mat level, intermediate floor level and ground level. The enclosure structure also supports a moving crane.

3. Forced Vibration Tests

For the forced vibration tests a Kinemetrics VG-1 and a locally made vibration generator were employed to excite the structure. The Kinemetrics VG-1 model vibration generator was manufactured to the specifications of California State Division of Architecture and the U.S. Earthquake Engineering Research Institute (Hudson, [8]). The design configuration is based on the counter-rotating eccentric weights to provide a sinusoidally varying unidirectional horizontal inertia force. The frequency of vibration can be varied from 0 to 10Hz with an accuracy of 0.1%. The inertial force can be controlled by changing the lead weights in the rotating baskets to reach a maximum value of 22 kN. For excitation at frequencies above 10 Hz. a locally made small vibration generator is used.

The data acquisition in the test was performed with Kinemetrics SS-1 Ranger seismometers used as velocity transducers. The outputs of seismometers were connected to Kinemetrics SC-1 signal conditioner where the signals are low-pass filtered and amplified. The data at the output of signal conditioner were recorded to Honeywell 1885-SCC strip chart recorder and also observed and photographed through an HP 3582 Spectrum Analyzer. Statham 0.25 g strain gauge accelerometers were used as acceleration transducers.

4. Experimental Results

The structure was excited in its short direction with the exciter mounted at the ground floor and the response is measured at several points on the structure as indicated in Fig.1 Excitation up to 15Hz was used. Excitation beyond this range has not been practical since the local response of the floor slabs and beams started to contaminate the overall response of the system. The reduced response data in the form of the plots of normalized absolute acceleration (normalized by the square of the frequency of excitation are presented in Figs. 2-7 for different series of experiments. As it can be seen from these response-frequency curves there are two modal frequencies in the excitation range: First modal frequency = 6.5Hz. (Observed clearly at the winch level response and vaguely in the ground and intermediate floor response), Second modal frequency = 10-12 Hz. (Although clearly seen in all level responses, the frequency is not clearly defined due to limited data and high damping). The half-power bandwidth method (Clough and Penzien [2]) yields ξ_1 = 6% and ξ_2 = 15% as the damping ratios for the first and second modal vibrations. The experimental mode shapes are plotted in Fig.8. On the basis of the experimentally measured first mode vibration characteristics of the soil-structure system the foundation stiffnesses can be obtained as: Lateral Stiffness (k_b) = 4×10^{10} N/m and Rocking Stiffness (k_ψ) = 3×10^{12} Nm/rad.

5. Theoretical Modeling

The soil-structure interaction phenomena can be analysed on the basis of a simple three-degree-of-freedom (D.O.F) mathematical model. In this model the embedded part of the structure is modeled as a rigid body with one rotational (rocking) and one translational (lateral motion) D.O.F. and the free part of the structure is modeled with one lateral D.O.F. and simulating its generalized fundamental fixed-base mode (Fig.9). The use of such simple models are sanctioned in ATC-3 [1] for the analysis and design of buildings influenced by interaction. The symbols used in Fig.9 denote the following physical quantities:

$\bar{m},\bar{k},\bar{c}$: First fixed-base mode generalized mass, stiffness and damping coefficient of the superstructure.

$\bar{h}_{A,B}$: Effective height of $\bar{m}$ to yield the fixed-base first mode overturning moment.

I : Moment of inertia of the superstructure about its centrodial axis.

m_b : Total mass of the embedded part of the structure.

$(I_b)_A$: Moment of inertia of the embedded part about the center of stiffness

k_x, c_x : Lateral stiffness and damping coefficients of the base.

k_ψ, c_ψ : Rocking stiffness and damping coefficients of the base.

h : Height of the center of stiffness above the base.

x_b : Relative lateral displacement of the embedded part of the structure with respect to x_t.

Ψ_b : Relative rocking angle of the embedded part of the structure with respect to Ψ_f.

x_{fA} : Lateral ground motion at the level A.

Ψ_f : Rocking ground motion.

$P(t)$: External force applied on the structure at a distance ℓ above A.

The equation of motion can be given by the following coupled equations, derived after the methodology given by Clough and Penzien [2] and Gutierrez and Chopra [5].

$$[\underline{M}]\,\{\ddot{\underline{u}}\}+[\underline{C}]\,\{\dot{\underline{u}}\}+[\underline{K}]\,\{\underline{u}\}=\{\underline{F}\} \tag{1}$$

where $[\underline{M}]$, $[\underline{C}]$ and $[\underline{K}]$ are respectively the mass, damping and stiffness matrices, $\{\underline{u}\}$ and $\underline{F}$ are displacement and forcing vectors given as follows:

$$[\underline{M}]=\begin{bmatrix} \bar{m} & \bar{m} & \bar{m}\bar{h}_A \\ \bar{m} & \bar{m}+\bar{m}_b & \bar{m}\bar{h}_A \\ \bar{m}\,h_A & \bar{m} & h_A \quad \bar{m}\bar{h}_A^2 + I + (I_b)_A \end{bmatrix};\ [\underline{K}]=\begin{bmatrix} \bar{k} & 0 & 0 \\ 0 & k_x & 0 \\ 0 & 0 & k_\psi \end{bmatrix};\ [\underline{C}]=\begin{bmatrix} \bar{c} & 0 & 0 \\ 0 & c_x & 0 \\ 0 & 0 & c_\psi \end{bmatrix} \tag{2, 3, 4}$$

$$\{\underline{F}\}=\begin{Bmatrix} \bar{m} \\ \bar{m}+\bar{m}_b \\ \bar{m}\,\bar{h}_A \end{Bmatrix}\ddot{x}_{f_A}-\begin{Bmatrix} \bar{m}\,\bar{h}_A \\ \bar{m}\,\bar{h}_A \\ \bar{m}\,\bar{h}^2+\bar{I}+(I_b)_A \end{Bmatrix}\ddot{\Psi}_f+\begin{Bmatrix} 0 \\ 1 \\ \ell \end{Bmatrix}P(t)\ ;\{\underline{u}\}=\begin{Bmatrix} u \\ x_b \\ \Psi_b \end{Bmatrix}$$

The properties of the fixed base superstructure can be obtained through routing structural analysis techniques. The base stiffness and damping coefficients can be determined through the use analytical expressions (Veletsos and Wei [11], Erden [3], Kausel et al.[9]).

6. Analytical Results and Correlation

The soil profile under the reactor building consist of a 50m deep sandy-clayey soil layer overlying a relatively more rigid sand-stone lime-stone medium. Following assessments have been made regarding the soil layer: Density (ρ) = 1900 kg/m^3, Shear Wave Velocity (C_s) = 350 m/s and Poisson's Ratio (ν) = 0.4 .

For purposes of the mathematical simulation of the soil-structure system during testing, it would be appropriate to choose the subgrade stiffnesses that would correspond to a frequency range between 6 and 11 Hz., where the first two experimentally determined modal frequencies are located. In this frequency range the subgrade stiffness coefficients can be determined to be approximately $k_x = 3.92 \times 10^{10}$ N/m and $k_\psi = 4.66 \times 10^{12}$ N.m/rad on the basis of the procedure in Kausel et.al., The experimental subgrade stiffnesses determined before ($k_x = 4 \times 10^{10}$ N/m and $k_\psi = 3.2 \times 10^{12}$ N.m/rad) indicate excellent correlation with these analytical values.

The results of the modal analysis based on the undamped free-vibration form of the eq. (1) yields the following modal frequencies (f_i) and mass-matrix-normalized modal shapes $\{\emptyset_i\}^T = \{u_1,\ x_{b1},\ \Psi_{b1}\}$:

First Mode : $f_1 = 6.36$ Hz , $\{\emptyset_1\}^T = \{1.7 \times 10^{-3},\ 2.9 \times 10^{-5},\ 3.1 \times 10^{-6}\}$

Second Mode : $f_2 = 12.9$ Hz , $\{\emptyset_2\}^T = \{-5.1 \times 10^{-4},\ 4.1 \times 10^{-4},\ -1.7 \times 10^{-6}\}$

Third Mode : $f_3 = 18.1$ Hz , $\{\emptyset_3\}^T = \{-8.9 \times 10^{-4},\ 1.1 \times 10^{-5},\ 5.2 \times 10^{-5}\}$

The experimental and analytical mode shapes are compared in Fig.10. It should be noted that the analytical modal vibration frequencies and the modal vibration shapes correlate satisfactorily with those based on the experimental techniques.

7. White Noise Analysis

White Noise type ground acceleration can be used with the frequency domain analysis to obtain the modal frequencies of vibration of the structural systems with non-Rayleigh type damping characteristics. Since such ground motions will have uniform energy in all frequency regions the structural frequencies that correspond to the modal frequencies will be amplified compared to the others and will be discernable on their respective Fourier Amplitude Spectra (F.A.S.). In Fig.11a the F.A.S. of the horizontal free field ground acceleration ($\ddot{x}_g$) is provided. As it can be seen the F.A.S. is relatively flat over the frequency range of 0.7 - 50 Hz. The F.A.S. of the relative accelerations at the three degrees of freedom ($\ddot{u}$, $\ddot{u}_b$ and Ψ_b) resulting from the excitation by $\ddot{x}_g$ are indicated respectively in Fig.11 b,c and d. As it can be assessed the prominent frequencies of these F.A.S. indicated by 1, 2 and 3 respectively correspond the first, second and third modal frequencies obtained before.

8. Conclusions

The basic conclusions of this study are that the in-situ forced vibration test is a feasible procedure for the assessment of dynamic parameters and the two-dimensional three-degree-of-freedom analysis can be used for the theoretical estimation of the dynamic parameters of the soil structure interaction phenomena.

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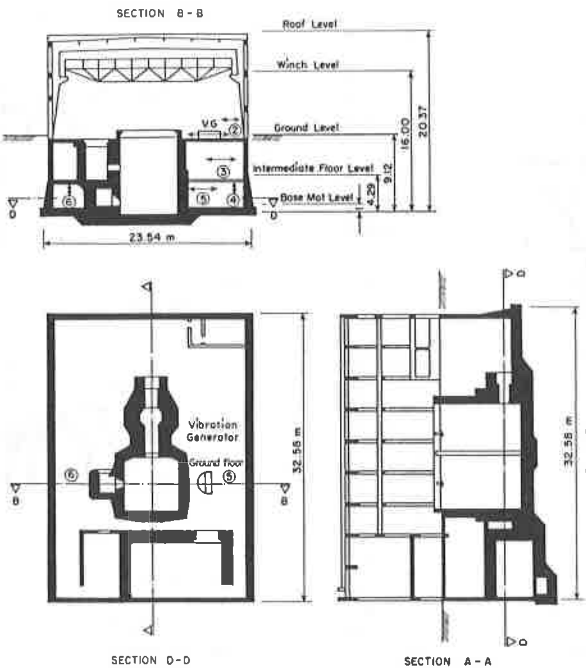

Figure 1. Sectional and Plan Views of the Reactor Building Structure and the Experimental Setup (Number in Circles Indicate Motion Transducess)

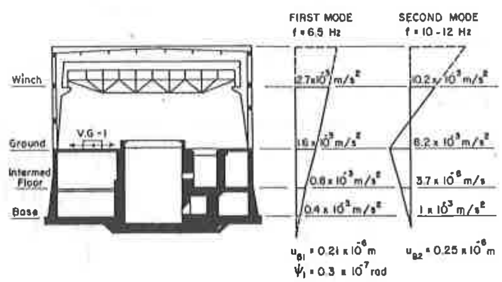

Figure 8. Experimental Mode Shapes (Relative Scale)

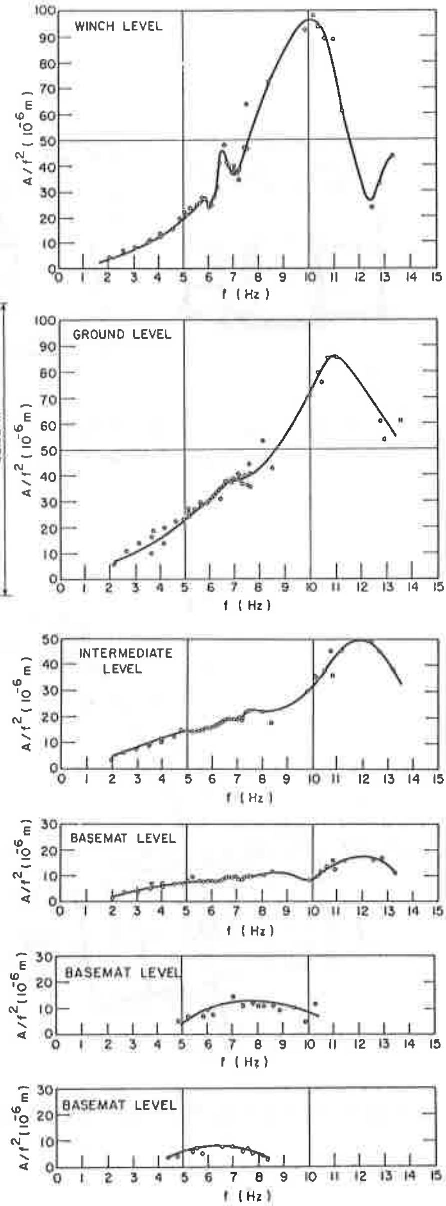

Figure 2-7. Response-Frequency Curve at different Levels.

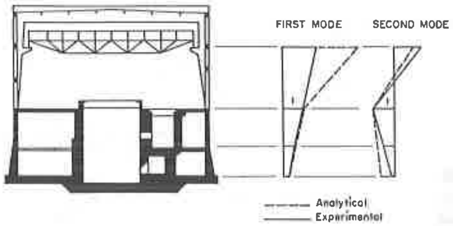

Figure 10. Comparison of Analytical and Experimental Mode Shapes

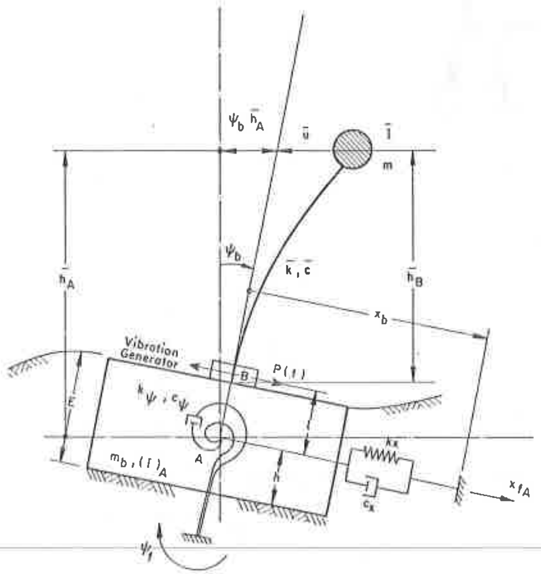

Figure 9. 3 D.O.F. Soil-Structure Model

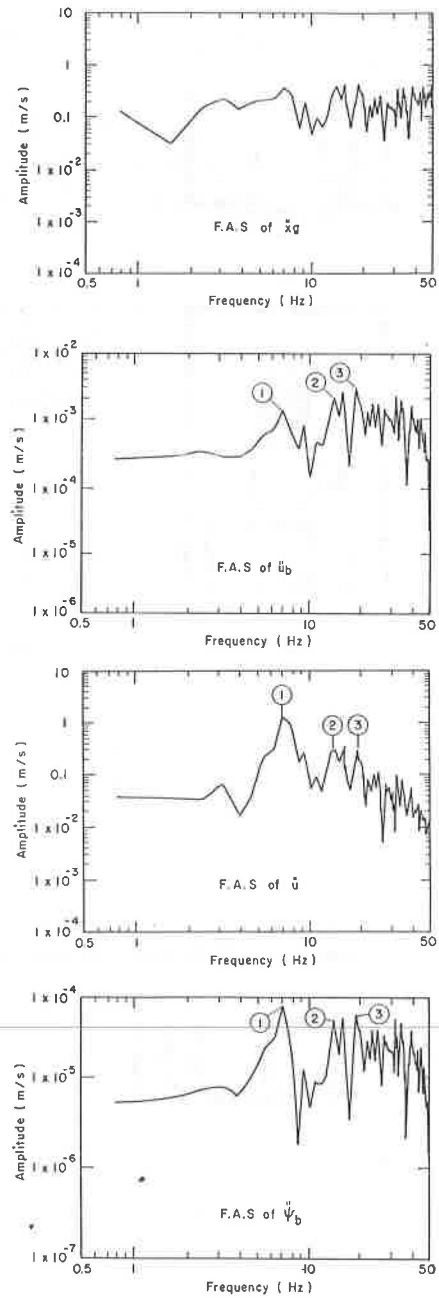

Figure 11.a, b, c, d Fourier Amplitude Spectra