MATHEMATICAL MODEL FORMULATION OF A FOURTEEN STOREY RC BUILDING USING STRONG MOTION RECORDS AND PARAMETER SYSTEM IDNETIFICATION

D. V. Jurukovski(I)
Lj. A. Tashkov(II)
V. K. Trajkovski(III)
Presenting Author: D. V. Jurukovski

SUMMARY

In this paper the results of mathematical model formulation of a fourteen storey building using records obtained during strong earthquake motion and parameter system identification are presented. The form of considered mathematical model is a lumped mass system with rotational and translational springs at the base of the structure. Using a strong motion record at the base as an input, and records at the seventh and thirtheenth floor as known responses,the vector of three unknown parameters was identified by the concept of system identification. Based on this analysis, conclusions for soil-structure interaction effect are given.

INTRODUCTION

During the last ten years, intensive programmes of strong motion instrumentation of buildings with relatively small number of instruments, form 3 to 5, have been carried out in many countries. The strong motion data obtained by such a small number of instruments are insufficient for performing the complex investigations aimed at definition of significant parameters which define the dynamic behaviour of the considered structure. In this research, by using only three records taken on the building, with the instruments being placed at the basement, the 13-th and the seventh floor, applying the parameter system identification technique and by using records of actual earthquake, a mathematical model was formulated which provided data on the dynamic behaviour of the building. A linear lumped mass model flexible at the base was considered assuming that the soil-structure interaction and the viscous damping coefficient are unknown parameters which will have to be established with the parameter system identification concept. The results of these investigations have shown that in flexible structural systems also the soil-structure interaction effect is significant, especially during strong ground motions.

DESCRIPTION OF BUILDING AND AVAILABLE EXPERIMENTAL DATA

Two forced vibration full-scale tests were carried out on a fourteen storey building, the geometry of which are shown in Figs 1 and 2, constructed

(I) Professor in Earthquake Engineering, IZIIS, University "Kiril and Metodij", Skopje, Yugoslavia

(II) Assistant Professor in Earthquake Engineering, IZIIS, University "Kiril and Metodij", Skopje, Yugoslavia

(III) Graduate Student of Earthquake Engineering, IZIIS, University "Kiril and Metodij", Skopje, Yugoslavia

in precast system. This building was studied before and after installation of the nonstructural partition walls and in addition, three strong motion accelerographs were installed to record the response under future earthquakes. The locations of these instruments are shown in Fig. 2. It should be mentioned that the building foundation is in favourable soil conditions with considerable gravel deposit.

From the second forced-vibration full-scale test in which a state of the building was similar to the expected one during the serviceability period of the building, the dynamic characteristics in the two orthogonal directions and torsion were defined. It is evident that there is certain coupling between the translational and torsional frequencies. The corresponding natural frequencies in E-W direction are $f_1 = 1.31$ Hz, $f_2 = 5.57$ Hz, in N-S direction $f_1 = 1.30$ Hz and $f_2 = 5.06$ Hz, while in torsion $f_1 = 1.50$ Hz and $f_2 = 6.08$ Hz. The viscous damping coefficient is 2.0% in average. From these experimental studies the authors constructed different forms of fixed base mathematical model.

During the Banja Luka, Yugoslavia, earthquake of August 1981 with magnitude M = 5.4, structural response at foundation level and on the 7-th and the 13-th floor were recorded. Acceleration record in N-S direction of the building has been shown in Fig. 3. Several days after the earthquake detailed inspection of the building was carried out and neither structural nor nonstructural damages were observed which shows that the building worked in the elastic range during the strong earthquake. Also, no traces of nonlinear soil nehaviour were observed close to the building.

One year later, ambient vibration test was carried out on the same building and the following resonant frequencies in the two orthogonal directions and torsion were obtained, respectively: E-W, $f_1 = 1.28$ Hz, $f_2 = 5.12$ Hz; N-S, $f_1 = 1.22$ Hz, $f_2 = 4.48$ Hz; torsion, $f_1 = 1.40$ Hz, $f_2 = 5.90$ Hz. It is evident from these results that the dynamic characteristics of the building have not changed. However, the absolute acceleration response spectra defined from the records taken at the 7-th and the 13-th floor have shown that certain softening of the building took place during the earthquake and that it basically vibrates with two dominant frequencies: $f_1 = 0.87$ Hz and $f_2 = 4.0$ Hz (Fig. 4 and Fig. 5). Based on these results, the basic assumptions for mathematical model formulation described hereunder, have been made.

MATHEMATICAL MODEL FORMULATION

In selection of the form of the mathematical model and performing the phases of the research, the following assumptions were made:

- During the earthquake, the superstructure behaved linearly. Based on this assumption, a lumped mass parameter model was suggested in N-S direction having threediagonal stiffness matrix, while each mass has only one degree of freedom. After that threediagonal stiffness matrix has been identified from the full scale experimental data considering fixed base model.
- During the earthquake, considerable increase in the soil-structure interaction effect took place. This assumption means treatment of flexible base model which in this case is presented with one rotational and one translational spring. For this model the stiffness matrix of the superstructure has been taken as a solution from the first assumption.

For providing the first assumption, using the data from forced and ambient vibration full-scale test for a fixed base model, the stiffness matrix was defined and the solution of the free vibration equation gives $f_1 = 1.37$ Hz, $f_2 = 5.24$ Hz and $f_3 = 8.63$ Hz and the corresponding mode shapes.

In considering a flexible base model (Fig. 6), it is assumed that the stiffness and mass matrices from the superstructure are known, that is as defined for the fixed base model, while for this model only interaction parameters will have to be defined. Following the G.A.Bekey's concept (Ref. 2) for parameter system identification of mathematical models, a model form has been selected in which the foundation mass has two degrees of freedom defined with the springs of rotation and translation. The vector of parameters $\underline{\beta}$ has been selected from the parameters of the equation of motion and it has to be identified based on this concept. So, the final parameter value of $\underline{\beta}$ should provide maximum correlation between the analytically predicted responses and the responses recorded on the 7-th and 13-th floor. Besides the interaction parameters, the viscous damping coefficient γ is presented as an unknown parameter, that is, an assumption has been made that the damping matrix depends only upon the stiffness matrix and the viscous damping coefficient. In this way the $\underline{\beta}$ parameter vector consists of :

$$\underline{\beta} = \{\gamma, k_\xi, k_x\}^T$$

This solution is justified and mathematically defined with the criterion function which is the sum of the squared errors between the measured responses on the 7-th and 13-th floor and analytically predicted responses at the same levels, accumulated in a period of excitation. The criterion function used has a form

$$J(\underline{\beta}, T) = \int_0^T [\underline{x}(\underline{\beta},t) - \underline{y}(t)]^T [\underline{x}(\underline{\beta},t) - \underline{y}(t)]\, dt$$

where $\underline{x}(\underline{\beta},t)$ represents a vector of calculated displacement histories at the 7-th and 13-th floor, while $\underline{y}(t)$ represents the recorded displacement time histories at the same levels.

The parameter adjustment algorithm is based on the modified Gauss-Newton procedure in which the sensitivity coefficients have been defined as a response difference for sufficiently small parameter increments:

$$\frac{\partial \underline{x}(\underline{\beta},t)}{\partial \beta_j} \equiv \frac{\underline{x}(\underline{\beta}+\Delta\beta_j,t) - \underline{x}(\underline{\beta},t)}{\Delta\beta_j}$$

The initial parameter values are assumed to be:

$$\underline{\beta}_o = \{2.5\%;\ 1\text{x}10^{11};\ 2\text{x}10^5\}^T$$

so that during the fifth iteration the final value of the $\underline{\beta}$ vector was obtained as follows :

$$\underline{\beta}_o = \{10.65\%;\ 1.67\text{x}10^{14};\ 0.92\text{x}10^6\}^T$$

The change in the value of the criterion function is given in Fig. 7.

RESULTS OF THE ANALYSIS

The correlation between the recorded displacement responses and the analytically defined vector parameters $\underline{\beta}$ as obtained during the fifth iteration has been shown in Figs 8 and 9, for the 7-th and 13-th floor, respectively. As shown in these figures, rather good correlation exists in respect to time, however, some differences in amplitudes are observed especially at the 7-th floor level. For flexible base model, frequencies of the first and second mode which are rather close to the predominant frequencies of the absolute acceleration response spectrum, have been obtained. Thus, the spectral frequencies are $f_1 = 0.87$ Hz and $f_2 = 4.0$ Hz, while for the model response they are $f_1 = 0.91$ Hz and $f_2 = 4.14$ Hz, respectively. By reducing the criteria for adopting of such a solution, improvement of the responses can be obtained so that the error level will be decreased, however, it is evident that some changes in the form of the mathematical model should be introduced. Namely, the assumption that each mass can bave only one degree of freedom should be corrected so that each mass should be allowed to have three degrees of freedom, two translational and one rotational. Also, the assumption that there is no rotations at the baams and joints can not be fully accepted, so a model which has a multi diagonal or full matrix will be more adequate.

All these improvements of the mathematical model form will contribute to meeting more successfully the main objective of these investigations. The basic idea of this paper was not to study the problem of formulation of an ideal mathematical model for the considered building which is subjected to strong ground motion, but to point out that during strong earthquakes soil-structure interaction effect has significant order in the dynamic behaviour of structures and it should not be neglected.

CONCLUSIONS

During strong earthquakes, the response of structures is influenced by the soil which modifies the frequency content of the structural response, causing thus softening of the dynamic system resulting in considerable changes in the response intensity. Certain time after the earthquake, the dynamic system is strengthened and brought to its original position, again. Such a phenomenon of behaviour of structures during earthquakes should be further investigated since it comprises significant advantages which could be introduced in the structural design practice.

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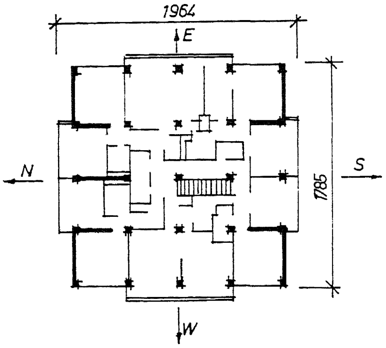

Fig. 1. Typical plan of building

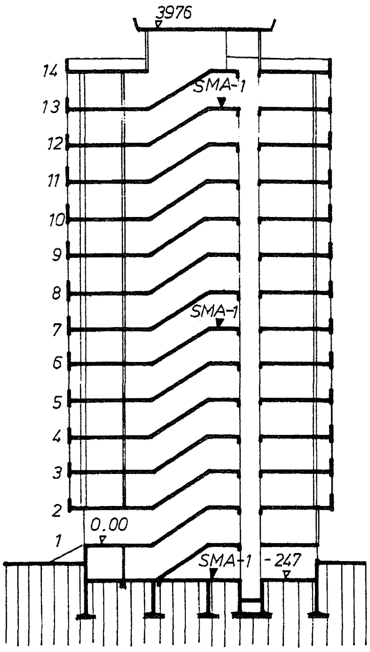

Fig. 2. Cross-section of building with position of instruments

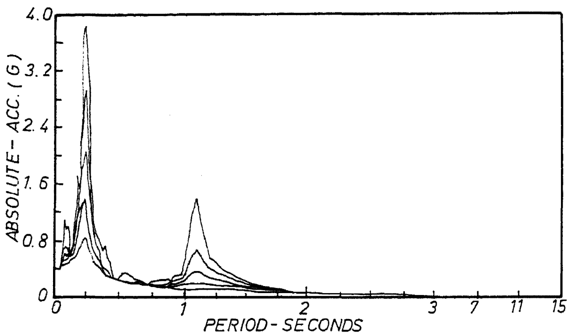

Fig. 4. Absolute acceleration response spectrum, 7th floor, N–S direction

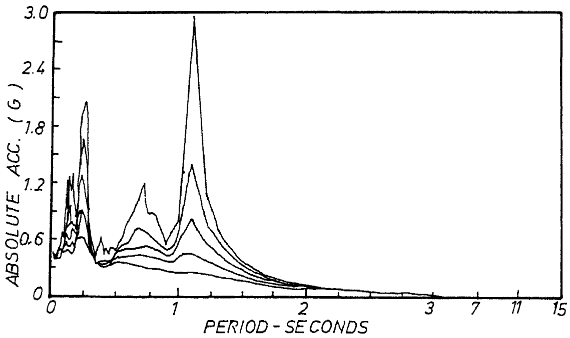

Fig. 5. Absolute acceleration response spectrum, 13th floor, N–S direction

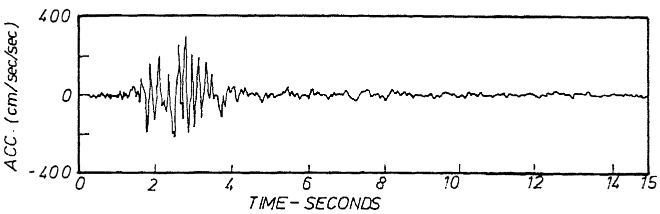

Fig. 3. Earthquake record obtained at the base in N–S direction

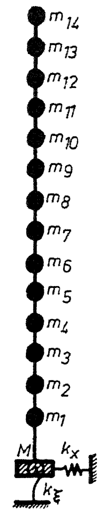

Fig. 6. Mathematical model idealization

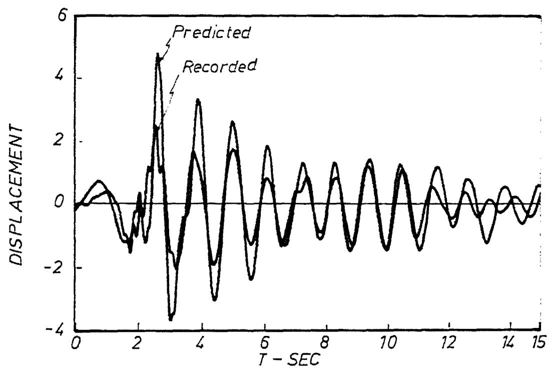

Fig. 8. Comparison of recorded and predicted displacement time histories at 7th floor

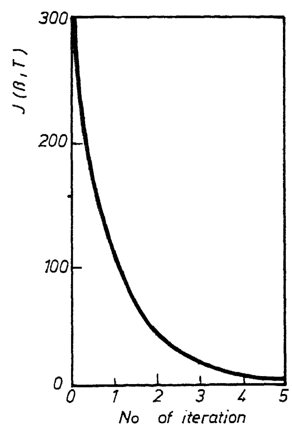

Fig. 7. Convergence of criterion function

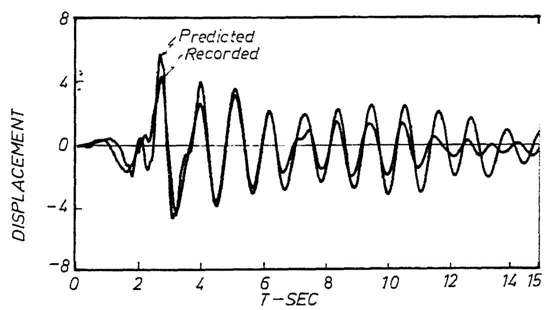

Fig. 9. Comparison of recorded and predicted displacement time histories at 13th floor