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SYSTEM IDENTIFICATION OF A BRIDGE FROM OBSERVED STRONG MOTION RECORDS

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1. INTRODUCTION

Strong motion data recorded during earthquake are valuable means of understanding the true behavior of structure undergoing such excitations. In this paper a two-step method of identifying structural parameters of a bridge, which is idealized as a 2DOF lumped mass model, is presented and system identified results are compared with the theoretical predictions of physical parameter values.

2. STRUCTURAL IDENTIFICATION IN MODAL DOMAIN

Two common methods of analysis of recorded earthquake response data are the synthesized model approach with trial-and-error adjustment of the parameters and the transfer function approach in the frequency domain. Both methods suffer from serious limitations[3]. In the present study, an output-error method[3] which tries to identify the system modal parameters rather than the elements of the stiffness and mass matrices was used in the frequency domain for system identific ation.

The uncoupled modal equation for response u_{pr} for a linear, classically damped system can be written as

$$\ddot{u}_{pr} + a_r \dot{u}_{pr} + b_r u_{pr} = -c_{pr} \ddot{u}_g \qquad (1)$$

where u_{pr} is the displacement of the structure in the r-th mode at location p and u_g is the ground displacement. While a_r, b_r and c_{pr} are the modal parameters representing the modal damping, modal frequency and effective mode participation factors respectively and are defined as $a_r = 2\xi_r \omega_r$, $b_r = \omega_r^2$, $c_{pr} = \phi_{pr} \dfrac{\Phi_r^T [m]\{1\}}{\Phi_r^T [m] \Phi_r}$.

The modal equation of motion (1) can be transformed to the frequency domain by taking Fourier transforms on both sides. Let $A_{pT}(\omega)$ be the finite Fourier transform of the absolute acceleration (u_p+u_g), $Z_T(\omega)$ that of ground acceleration and $U_{pT}(\omega)$ that of $u_{pr}(t)$. Transforming equation (1) and combining all modal contrib utions gives

$$A_{pT}(\omega) = \left[1 + \sum_{r=1}^{N} \frac{\omega^2 (b_r - \omega^2) - i\omega^3 a_r}{(b_r - \omega^2)^2 + \omega^2 a_r^2} c_{pr} \right] Z_T(\omega) \qquad (2)$$

for zero displacement and velocity conditions at the start and end of the recorded time histories. The identification is performed by selecting the modal parameters to obtain a least square fit of the transform of the model response [Eq.2] to the transforms of the measured response a cceleration.

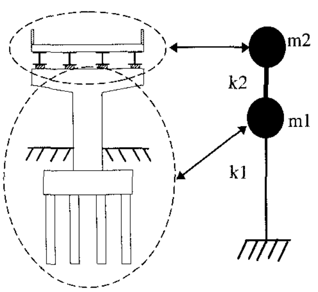

Fig.1: 2DOF idealization of Bridge

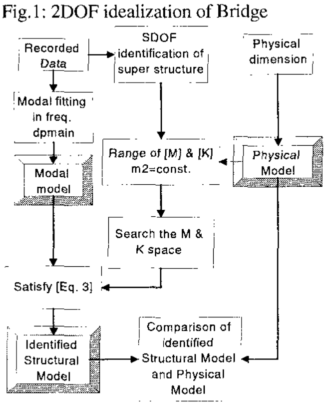

Fig.2: Flow chart for the proposed structural identification method

3. IDENTIFICATION OF STRUCTURAL PARAMETERS FROM THE MODAL PARAMETERS

After having identified a modal model, the next step is to transfer back to the structural domain as modal model does not provide any direct information to the designer. The major hindrance in the process is the non-availability of mass normalized mode shape vectors which are crucial in transferring from the modal domain to the structural domain. Therefore a method similar to the synthesized model method of system identification is adopted to obtain the elements of the mass and stiffness matrices. In this method, judicious estimates of the range of mass and stiffness elements are made and a thorough search of this multi-dimensional space is conducted such that

$$\left| (\omega)_{modal} - (\omega)_{structural} \right| = minimum \ \& \ \left| (c_{pr})_{modal} - (c_{pr})_{structural} \right| = minimum \qquad (3)$$

The outline of this method is shown in Fig.2. This method is superior to the synthesized model method as the objective functions against which to search are known [Eq.3].

4. APPLICATION OF THE PROPOSED METHOD TO MATSUNOHAMA VIADUCT BRIDGE

The base isolated bridge of the Matsunohama Viaduct of the Hanshin Expressway is a four span continuous bridge and is supported by laminated rubber bearings. During the 1995 Kobe earthquake, acceleration was measured at four locations viz. far field, pile cap, pier cap and girder in the longitudinal direction of this bridge. The far field data was measured only 20m away from the pile cap which resulted in far field and pile cap records having the same Fourier amplitude spectrum; thus

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reducing the number of measurement points to three. That is the reason why only a 2DOF model can be constructed. Abe et al[1] have shown that the super structure of base isolated bridges can be treated as a SDOF system. It is shown in this paper that the sub-structure of such bridges under moderate level of excitations can also be represented by one equivalent DOF.

Modal identification was done for the main shock and one after shock. Table 1 presents the results of the modal identification as well as the identified structural parameters. Search for the structural model from the modal model was facilitated as mass and stiffness of the super-structure were known from SDOF idealization. During structural identification, mass of the super-structure was kept constant while stiffness was varied to match the modal parameters. Fig. 2a shows the fourier amplitude spectrum for the recorded pier cap acceleration of the main shock while Fig 2b depicts the same for the 2-mode modal and the spectrum for the identified structural model is shown in Fig 2c.

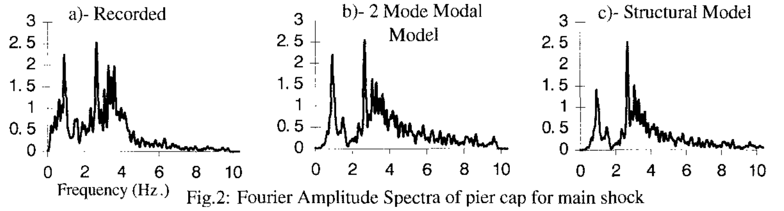

Fig.2: Fourier Amplitude Spectra of pier cap for main shock

Table 1: Parameters of Modal and Structural Models

	Modal Model				Structural Model					
	f_1 (Hz.)	f_2 (Hz.)	ξ_1	ξ_2	m_1 (Tons)	m_2 (Tons)	k_1 (kN/m)	k_2 (kN/m)	f_1 (Hz.)	f_2 (Hz.)
Main Shock	0.9	3.0	0.076	0.124	3400	6500	8.0E5	3.0E5	0.90	2.92
After Shock	1.6	3.5	0.03	0.03	8200	6500	2.5E6	1.05E6	1.59	3.54

5. COMPARISON OF PHYSICAL VALUES WITH THE IDENTIFIED VA LUES

(a) Mass

The values of the mass moment of inertia of the identified and the physical systems was compared. It was found that there is a large difference in the two values when mass of all five piers is taken while the difference reduces to less than 5% when the end piers are not co nsidered.

Table 2: Mass Moment of I nertia (Ton-m^2)

	Full Bridge		Middle Piers only	
	Identified	Physical	Identified	Physical
Main Shock	6.11E5	12.2E5	9.61E5	10.1E5
After Shock	1.11E6	1.11E6	7.89E5	7.77E5

(b) Sub-structure stiffness

Stiffness of columns and horizontal and rocking pile group stiffnesses were transformed to a single equivalent sub-structure stiffness and compared with the identified values. Pile group stiffnesses in the horizontal and rocking mode were found according to Dobry and Gazetas[2]. Pile group stiffness is strongly influenced by the effective shear modulus of soil and Tatsuoka et al[4] have suggested a reduction of upto 80% of the elastic value for dynamic loading condition. The soil properties were determined from the bore hole data and log of standard penetration test (SPT) values. Along the length of the bridge, average SPT N value was found to be 37 for the active pile length of 10m (8*pile dia.). This gave a maximum shear wave velocity of 266 m/sec. For soil damping ratio of 5%, the shear modulus of soil was found to be 100 MPa.

Table 3: Comparison of Sub-structure stiffness (kN/m)

	Identified	Physical	
		Full Bridge	Middle 3 Piers
Main Shock	800,000	1,500,400	785,388
After Shock	2,500,000	2,506,535	410,236

Comparison of identified and physical sub-structure stiffness is shown in Table 3. It should be noted that the physical stiffness values for the main shock were arrived at after reducing the elastic pile group stiffness and uncracked column stiffness by 40% each which represent the realistic values of these parameters for this acceleration level. No reduction to the elastic values was made for the after shock due to relatively good soil and lower excit ation level.

It can be concluded, for the class of bridges under discussion, that it is most likely that only a part of the sub-structure will participate during seismic excitations and pile group stiffness is likely to be fraction of the elastic value.

6. CONCLUSIONS

- It is possible to capture the dynamics of Matsunohama Viaduct bridge by a simple 2DOF lumped mass model.
- The two-step procedure of finding the structural parameters of the bridge system is relatively simple and quite effective.
- For medium level of excitation, only a part of the sub-structure participates in resisting the seismic excitation.
- Pile group stiffness has to be substantially reduced from its elastic value due to large dynamic soil strains.

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