do not introduce into the problem additional data. Therefore, they are mathematical devices for the solution of an undetermined set of equations. The belief in their ability to give the location and magnitude of the damage comes from the numerical examples. Hassiotis and Grigoriadis (1996) introduce a new set of criteria based on measurements of the system Markov parameters.

The numerical example is based on a 10-story two-bay frame. The frame was modeled by a 50-element, 90 degree-of-freedom (DOF) system using finite elements and not by 10 mass-spring units. Therefore, the use of 10 frequencies does not constitute a full set of data on natural frequencies. Damage on the 90 DOF structure was identified using as few as seven natural frequencies when the unknowns were any of 50 elements. The example chosen for publication is representative of results obtained when seven or more frequencies were assumed to constitute the data. In a similar example of a 10 DOF system that was modeled by spring-mass units, three natural frequencies were found to be enough for identification of damage. Precise identification of damage will be limited by the chosen model for the structure and by the data that can be obtained for a given structure.

In case of noise-polluted data, the identification is not as good as with exact data. A separate study using only the first criterion and noise-polluted data is published in Hassiotis and Jeong (1993). It was found that damage (in the order of 50% decrease in stiffness) can be identified when the measured natural frequencies are within ±1% of their actual values. However, in both the experimental example published within and the numerical examples published earlier, the identification of erroneous or "spurious" damage was observed mainly due to the noise in the data.

The damping ratios shown in Table 2 were used simply to obtain undamped natural frequencies. Although damage will affect the damping of a system, trying to concurrently identify both changes in the stiffness and damping matrices would be infeasible with such limited data.

The development of the theory depends on a model of a structure before damage and a model of the structure after damage. Since we are looking at a change between the two models, the original modeling of the boundary conditions will not affect the results or conclusions of the method presented. However, the writers agree with the discussers that the boundary conditions should be carefully modeled when dealing with real problems.

APPENDIX. REFERENCE

Hassiotis, S., and Grigoriadis, K. M. (1996). "Identification of structural damage." 1996 ASCE Struct. Congr., ASCE, New York, N.Y., 1107–1114.

Vibration Control of Tall Buildings Using Mega Subconfiguration[a]

Discussion by Fahim Sadek[3] and Bijan Mohraz,[4] Fellow, ASCE

The authors present an innovative system of vibration control to reduce the dynamic response of tall buildings. By considering the "mega-substructure configuration" where substructures consisting of several floors serve as vibration absorbers, the responses of the structure and its substructures are reduced significantly. This system is cost effective, since it requires no additional masses to control vibrations as tuned mass dampers and vibrations absorbers do.

To determine the substructure (absorber) parameters, the authors use a two-degree-of-freedom system and minimize the mean square response of the main mass to a white noise ground and wind excitation. For ground acceleration, they arrive at the following frequency and damping ratios (Table 2):

$$\beta = \frac{\sqrt{1 - \frac{\mu}{2}}}{1 + \mu} \tag{9}$$

and

$$h_2 = \frac{1}{2}\sqrt{(1 + \mu)\beta^4 + \beta^2 + \frac{1 - 3(1 + \mu)^2\beta^2}{(1 + \mu)^3}} \tag{10}$$

where β, μ, and h_2 = frequency, mass, and damping ratios of absorber, respectively. The damping ratio h_2 is defined in terms of the natural frequency of the structure instead of the absorber. In computing μ, the authors use the ratio of the substructure (absorber) mass to that of the mega-floor to which it is attached. For multi-degree-of-freedom structures, the floor masses do not represent the structural mass in a given mode; therefore, it would not be appropriate to use them to compute the mass ratio. Kaynia et al. (1981) and Villaverde (1985) have suggested that one should compute the mass ratio using the generalized mass for a given mode (usually the fundamental mode) corresponding to a unit modal participation factor. If one uses the generalized mass for the fundamental mode (M_1

[a]October 1995, Vol. 121, No. 10, by Maria Q. Feng and Akira Mita (Paper 9125).

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TABLE 4. Peak Response of Megastructure to NS Component of El Centro

	AUTHORS' DATA[a]				AUTHORS' METHOD USING μ = 0.28				VILLAVERDE'S METHOD USING μ = 0.28			
	Megastructure		Substructure		Megastructure		Substructure		Megastructure		Substructure	
Level (1)	x_{max} (mm) (2)	a_{max} (g) (3)	x_{max} (mm) (4)	a_{max} (g) (5)	x_{max} (mm) (6)	a_{max} (g) (7)	x_{max} (mm) (8)	a_{max} (g) (9)	x_{max} (mm) (10)	a_{max} (g) (11)	x_{max} (mm) (12)	a_{max} (g) (13)
Top	167	0.150	159	0.023	103	0.111	95	0.020	125	0.067	54	0.039
3	127	0.110	130	0.021	96	0.090	85	0.018	118	0.077	49	0.040
2	84	0.134	122	0.021	86	0.121	82	0.014	98	0.110	48	0.041
1	56	0.158	103	0.018	52	0.151	66	0.010	54	0.149	49	0.030

[a]The small discrepancy between the results presented in Table 3 of the paper and the results shown under (μ = 1.0) is due to the fact that the authors used approximate values in computing the substructure parameters, while exact parameters are used herein.

= 35,737 Mg) for their example, the mass ratio would be 0.28 instead of 1. Table 4 presents the response of the mega- and substructures to the same excitation used by the authors (NS component of El Centro, 1940, normalized to a peak velocity of 25 cm/s) for mass ratios $\mu = 1$ and $\mu = 0.28$. The results indicate that further reductions in response are achieved using the generalized mass.

Eqs. (9) and (10) are derived using a vibration absorber attached to a single-degree-of-freedom structure. For multi-degree-of-freedom structures, however, the validity of these equations is questionable. Villaverde (1985) considered the effect of the fundamental mode shape when determining the parameters of the absorbing systems. In such cases, the damping ratios for different substructures will no longer be equal, as assumed by the authors, since they would depend on the shape of the fundamental mode. The frequency and damping ratios suggested by Villaverde are

$$\beta = 1 \tag{11}$$

and

$$h_2 = \xi_s + \phi\sqrt{\mu} \tag{12}$$

where ξ_s and ϕ = structure's damping ratio and mode shape amplitude (normalized to have unit modal participation factor) at location of absorber, respectively. The responses of the same structure using (11) and (12) are also presented in Table 4. The table shows that by using (11) and (12), one can further reduce the displacements of the substructures. This reduction, however, is accompanied by some increase in accelerations when compared to those for $\mu = 0.28$.

It should be noted that when considering mega-substructure configurations, the connection between the megafloors and substructures underneath them should be accounted for in design by permitting the substructures to move independently of the megastructure.

The following is one minor correction: in the right column of page 1086, line 29, β should be 0.35 and h_2 should be 0.15.

APPENDIX. REFERENCES

Kaynia, A. M., Veneziano, D., and Biggs, J. M. (1981). "Seismic effectiveness of tuned mass dampers." *J. Struct. Div.*, ASCE, 107(8), 1465–1484.

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Closure by Maria Q. Feng,[5] Associate Member, ASCE, and Akira Mita,[6] Member, ASCE

For a multi-degree-of-freedom mega-sub building, the mass ratio in our study is defined with respect to a certain megastructure mode shape normalized with its top amplitude (Mita and Kaneko 1994). That is

$$\mu = (\phi_1^2 m_{s1} + \phi_2^2 m_{s2} + \cdots + \phi_n^2 m_{sn})/M$$

(= 1.0 in Feng and Mita's example for every mode) (13)

where ϕ_i represents normalized modal amplitude of ith megamass for selected mode; and m_{si} = ith submass. It is assumed that the dynamic characteristics of all substructures are the same. The generalized mass in (13), M, is defined as

$$M = \phi_1^2 m_{m1} + \phi_2^2 m_{m2} + \cdots + \phi_n^2 m_{mn} \tag{14}$$

where m_{mi} = ith megamass. It is clear that the mass ratio defined in this form is independent of participation factors, making it possible to define a unique mass ratio for various external loads.

The mass ratio of 0.28 presented by Sadek and Mohraz is a mass ratio between the top submass and the generalized mass for a unit participation factor. The submasses attached to the lower floors are not included. If they are included, the mass ratio should be close to 1.0.

Slight improvement in displacement response is observed in Sadek and Mohraz's example. However, this is mainly due to the fact that the spectral components of the El Centro (1940, NS) earthquake ground motion are not flat, while the optimum values obtained by Feng and Mita are for a white-noise input.

Sadek and Mohraz's interest in our paper is highly appreciated.

APPENDIX. REFERENCE

Mita, A., and Kaneko, M. (1994). "Vibration control of tall buildings utilizing energy transfer into sub-structural systems." *Proc., 1st Int. Conf. on Struct. Control, TA2-31-40.*

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