USING TUNED-MASS DAMPERS TO REDUCE SEISMIC RESPONSE

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SUMMARY

A realistic prototype high-rise building was designed and modeled using linear elastic and also nonlinear, degrading stiffness idealizations. Using an effective damper mass ratio of 0.026, three different techniques were used to design an optimum tuned-mass damper (TMD) for the prototype, and all were found to give essentially the same design. The response of the idealized prototype building to strong ground motion was computed, with and without a TMD. The TMD did not reduce the prototype's maximum response. Two modifications of the usual TMD were studied and also found to be ineffective in reducing maximum response. Vibration absorbers are not recommended as a means of reducing the maximum seismic response of tall buildings.

INTRODUCTION

Recent articles in various engineering publications have described the installation of so-called "tuned-mass dampers" to reduce the wind response of high-rise buildings such as the John Hancock Building in Boston or the Citicorp Center in New York City [1,2,3,4]. Although TMD's were installed in those two structures solely to reduce occupant discomfort [4,5], they have been found to decrease wind-induced accelerations by as much as 40% [4]. The purpose of the study described herein was to investigate whether or not TMD's or similar devices could be used to reduce the response of structures to earthquake ground motions.

BACKGROUND

The term "tuned-mass damper" is commonly applied to a wide variety of vibration absorbers. As shown in Fig. 1, the typical TMD consists of a mass M_T which can move freely relative to the primary structure, and which is attached to the structure by a linear elastic spring of stiffness K_T, in parallel with a dashpot C_T. The optimum TMD frequency ω_T is that which minimizes the primary structure's response, while optimum damping is that value of C_T which maximizes the energy dissipated per cycle by the TMD.

The above problem has been solved by Den Hartog for the case of a single degree of freedom system excited by a sinusoidally varying force [6,7]. The results can be conveniently summarized graphically, as in Figs. 2 and 3. For a given effective mass ratio μ (TMD mass to primary structure generalized mass in the mode under consideration), Fig. 2 gives

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the ratio of (ω_T/ω) which minimizes the primary structure's steady state response. Figure 3 gives the fraction of critical damping for the TMD

$$\xi_T = C_T/2M_T\omega_T$$

which minimizes the TMD's displacement relative to the primary structure. Den Hartog's solution has been extended by Wiesner [5] for a single degree of freedom system subjected to white noise wind excitation, and by Wirsching et al. [8] for similar systems subjected to white noise ground excitation. The solutions are compared in Figs. 2 and 3. Note that for typical effective mass ratios, about 0.03 or less, all three approaches give practically identical results.

PROTOTYPE BUILDING

It was decided to study a prototype building which might require a TMD because of wind serviceability requirements, yet have a fundamental period short enough to make earthquake-induced forces the controlling design lateral load. The prototype building was therefore configured to have a large lateral surface area, a flexible lateral load-resisting system, and a low average building density. As shown in Fig. 4, the 25-story building has a total floor area of 640,000 sq. ft. Gravity loads are resisted by a steel frame, and all lateral loads are assumed to be resisted by the two reinforced concrete elevator/service cores. Details of the building are given in Ref. 9.

In the linear elastic range, the structure was idealized as a vertical cantilever beam with a fixed base. The flexural stiffness of the core wall was assumed to decrease from the base to the tip of the structure, as shown in Fig. 5. The building's mass was assumed to be distributed uniformly over its height, and the building was idealized using an 8 node lumped-mass model with two degrees of freedom per node. Based on this model, and considering only the flexural stiffness of the two cores, the fundamental period of the building in the N-S direction was calculated to be 3.75 sec. However, the fundamental period was calculated to be only 1.90 sec. by an empirical formula [10] which would reflect the stiffening effects of nonstructural elements as well as the steel frame. The fundamental period was assumed to be 1.90 sec., and the flexural stiffness of the model was therefore factored by the ratio $(3.75/1.90)^2$ to adjust its fundamental period to that value.

The nonlinear structure was modeled using the nonlinear analysis program DRAIN 2D [11]. The nonlinear, degrading stiffness behavior described in the previous section was approximated by modeling each beam section as a linear elastic beam-column with zero-length hinges at each end. The hinges were assigned yield moments as shown in Fig. 5, and a degrading moment-rotation behavior described by the Takeda model with a strain-hardening ratio of 0.3. A damping coefficient of 5% of critical was used for the first and second vibrational modes, and the damping matrix was computed assuming Rayleigh damping, and using the initial elastic stiffness matrix.

EFFECT OF TMD ON SEISMIC RESPONSE OF PROTOTYPE BUILDING

A TMD was designed using a mass ratio of 0.65%, corresponding to an effective mass ratio of 0.026 between the TMD mass and the building's first-mode generalized mass. Using $\mu = 0.026$, the optimum TMD constants are shown in Table 1. It is clear that there is little difference among optimum TMD's designed by any available method.

TABLE 1

	K_T (k/in.)	C_T (k-sec/in.)
Den Hartog	13.84	0.84
Wiesner	13.69	0.69
Wirsching	13.40	0.68

The prototype building, with and without a TMD, was subjected to the first 30 sec. of the N-S component of El Centro 1940. Figure 6 compares the tip displacement response for the linear elastic multi degree of freedom model, with and without an optimum Den Hartog TMD. This figure shows the TMD to have no effect on the building's maximum response.

Figure 7 compares the displacement response of the nonlinear degrading stiffness model, with and without the optimum Den Hartog TMD. It is clear that a TMD with a practical value of μ (0.026 in this case) does not significantly alter the response of the prototype building to the given earthquake record. Yielding occurs at the base of the core walls in spite of the presence of the TMD.

ALTERNATIVE TYPES OF VIBRATION ABSORBERS

Variable Frequency Tuned-Mass Damper

When a building sustains earthquake damage, its modal frequencies decrease, causing the TMD to get further and further out of tune and thus be less effective. Since its natural frequency can be altered substantially, the variable tuned-mass damper (VTMD) can be maintained at the optimum tuning ratio as the building degrades. Referring to Fig. 8, as C_3 is varied from zero to large values, the VTMD's natural frequency varies from $\omega_{VT} = \sqrt{K_T/M_T}$ to $\omega_{VT} = \sqrt{(K_T + K_3)/M_T}$.

Despite the fact that it can be optimally tuned over a wide range of building frequencies, the VTMD is no more useful than the TMD in preventing the primary structure from yielding. Like the TMD, the VTMD is a passive vibration absorber needing primary structure motion to make it effective, and therefore could not prevent the repeated yielding of the prototype building as observed in Fig. 7.

Anticipatory Vibration Absorber

The anticipatory vibration absorber (AVA), shown in Fig. 9, was proposed in an attempt to circumvent the principal disadvantage of all passive systems: their reliance on motion of the primary structure to make them effective as response reduction devices. Controlled by an accelerometer at the base of the primary structure, the AVA actuator would push against the AVA mass, producing a force on the primary structure opposite in sense to those produced by the ground accelerations, and thereby reducing the total response.

However, the AVA was found to have an uncorrectable shortcoming. In order to counteract the effects of ground motion, the AVA would have to apply generalized impulses approximately as large as those from the earthquake. Since the AVA's mass is at most a few percent of the building's, the relative accelerations of the AVA must be an order of magnitude higher than the ground accelerations, resulting in very large displacements of the AVA relative to the building. When practical displacement limits were applied, the AVA was found to be ineffective and considerably more difficult to control than a passive device having a comparable mass ratio.

CONCLUSIONS

For realistic effective mass ratios, all available TMD design procedures gave optimum TMD's which were essentially identical. Using a TMD mass ratio of 0.65% (i.e., a first-mode effective mass ratio of 0.026), it was found that the optimum TMD made no contribution towards reducing the maximum lateral forces at the base of the building. Vibration absorbers are not recommended for reducing the maximum seismic response of tall buildings.

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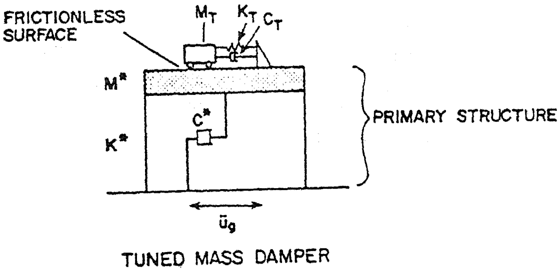

Fig. 1

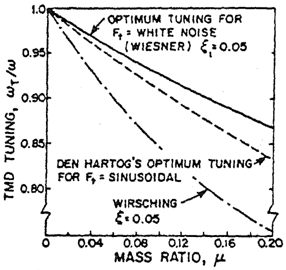

Fig. 2

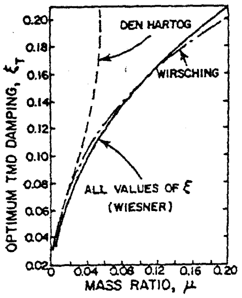

Fig. 3

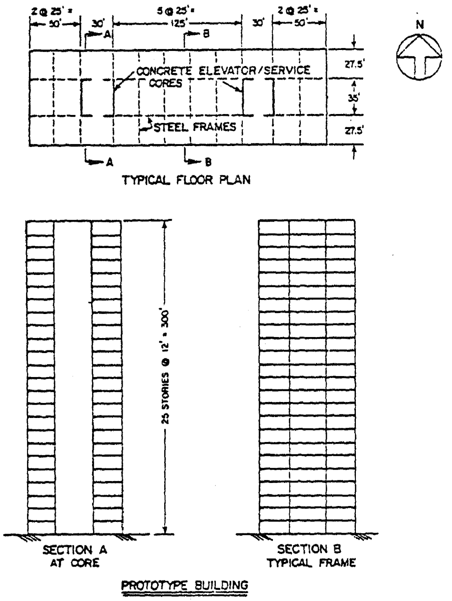

Fig. 4

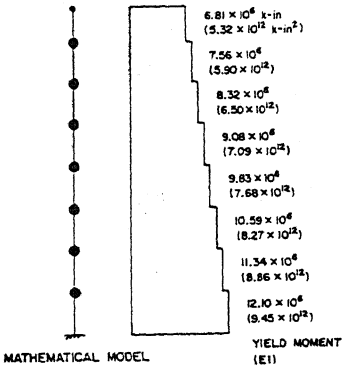

Fig. 5

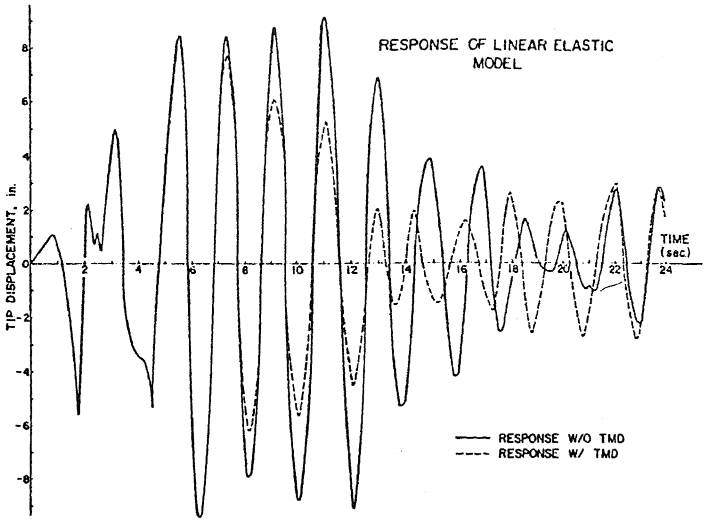

Fig. 6

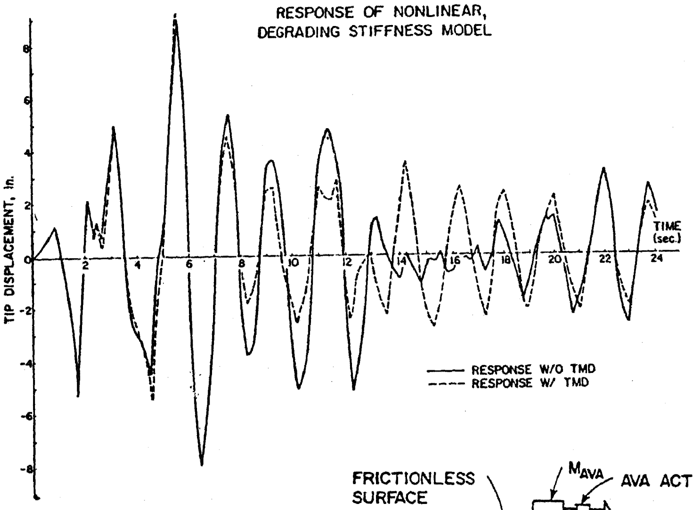

Fig. 7

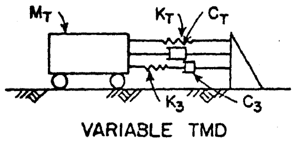

Fig. 8

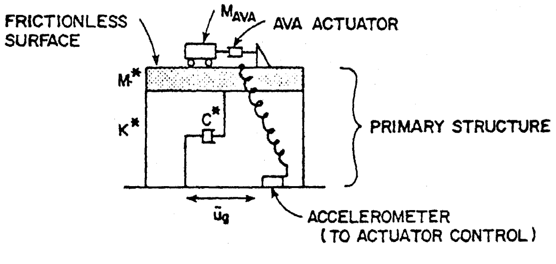

Fig. 9