INFLUENCE OF DISCRETE DAMPERS ON SEISMIC RESPONSE OF A FREESTANDING TOWER

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SUMMARY

Dynamic analysis of large freestanding towers, which are among the most highly stressed civil engineering structures, challenges the capabilities of the structural analyst and the capacity of even modern day computers. At the same time, open-latticed steel towers usually do not have sufficient damping capacity to dissipate the energy delivered to the structures by wind and earthquake loadings. Therefore some mechanism is needed for introducing additional energy absorption capacity in order to limit dynamic response. One solution is to insert discrete damping devices into the structure at predetermined locations. These devices would be used to dissipate some of the excess energy and consequently lower the dynamic response.

To facilitate the dynamic analysis of large towers, a substructuring technique was developed which can be used for assemblage of dynamic models of self-supporting towers idealized as linearly elastic space truss structures. The substructure method was applied to the analysis of a 1047 ft (319 m) communication tower acted upon by moderate earthquake ground motion. The effect on tower response of incorporating damping devices into the tower model was investigated. The size, number and distribution of dampers required to significantly reduce tower response were determined. In general, the study showed that tower response was more dependent on the size and distribution of dampers than on the actual number of dampers used for the structure and loading considered.

INTRODUCTION

The development of advanced computer software [7][†] for structural analysis and design has resulted in tower structures which are slender, lightweight, and very lightly damped, usually less than 1% of critical [2]. The slender compression elements are susceptible to buckling failure, and the structure, as a whole, is sensitive to wind, wind combined with ice, and seismic loadings which could compromise the utility or structural integrity of the entire tower [3,4].

Dynamic analysis of large towers for wind and seismic loadings represents a particular challenge to the structural analyst at the present time. Procedures for dynamic analysis of large towers containing hundreds of members and joints are inevitably time-consuming and expensive, and may tax the capacity of even modern day computers. The method of substructures offers a possible approach for efficient analysis of large and complex tower structures for dynamic loadings.

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The sensitivity of towers to wind and earthquake loadings is at least in part due to the low damping capacity of open-latticed frameworks. The damping capacity of individual structures is difficult to predict and, in fact, may change significantly with time due to prior severe loadings which change the properties of the structure. Damping is typically assumed to depend upon some combination of internal elastic and inelastic deformation as well as friction between different structural elements. In lightweight structures, the deformations of massive structural elements cannot be relied upon to provide sufficient damping. Additional damping may be required to reduce dynamic response and control troublesome vibration problems which may ultimately result in the collapse of the entire structure. A better understanding of damping in structures would allow the structural designer to provide known levels of damping to control the transient response of structures. In particular, installing specific energy absorbing devices may be considered for lightly-damped structures such as towers. These devices would dissipate the energy caused by wind and earthquake excitations. The required number, distribution and type of damping devices depend upon the characteristics of the structure, but necessary features of an energy absorbing mechanism are low cost, large capacity for energy absorption, resistance to fatigue and other forms of structural deterioration, and replaceability. A number of dampers with these characteristics have been developed [8,9] and tested and may be suitable for use in slender frameworks.

In this study, a substructure analytical model was developed for the dynamic analysis of large, freestanding tower structures. The effect of incorporating damping devices into the condensed analytical model of a 1047 ft (319 m) television tower on structure response to earthquake ground motion was investigated. A dynamic model, vibration properties, and time-history response to base excitation are presented for the prototype structure with different levels of damping.

PROTOTYPE STRUCTURE

The prototype structure (Fig. 1) is a free standing, three-legged, latticed steel tower based on three 7 feet (2.1 m) diameter reinforced concrete caissons which extend approximately 50 feet (15.3 m) below ground surface. In plan the structure is an equilateral triangle and the side length of the triangle at the base is 94.167 feet (28.7 m). In elevation the structure has a constant slope of 2.55 degrees from the base to the first bend line at a height of 320.833 feet (97.9 m) above the base. The side length of the plan section at the first bend line is 44.594 feet (13.6 m). The tower tapers at a constant slope of 1.21 degrees from the first bend line to the second bend line at a height of 860.375 feet (262.4 m) above the base. The tower has a constant width of 5.25 feet (1.6 m) from the second bend line to the top of the supporting structure at a height of 980.375 feet (299.0 m) above the base. At this point a 66.625 feet (20.3 m) television antenna forms the uppermost segment of the tower.

Most of the tower members are angle shapes of A36 steel. However, the leg members consist of solid steel bars with yield strength varying from 95 ksi (6.55×10^5 kN/m^2) near the bottom to 36 ksi (2.48×10^5 kN/m^2) at the top of the tower. The diagonal members are predominately angle shapes and small steel rods. The minimum yield strength varies from 36 ksi (2.48×10^5 kN/m^2) to 50 ksi (3.45×10^5 kN/m^2) for these members. Of particular importance are several "tension-only" members located below the first bend line. These members are solid steel bars, 5/8 inches (1.6 cm) in diameter.

The structure contains 987 joints and 2850 members, and weighs approximately 609 kips (2707 kN). Both bolted and welded connections are used in the structure. Walkways are located at various levels in the structure. An access ladder and a number of transmission cables run up the southeastern leg of the tower, and the main television cable runs up the center of the tower. These walkways, ladders, and cables are assumed to add mass but no stiffness to the tower.

ANALYTICAL MODEL

Substructure Model.--A general space truss model, with three translational degrees of freedom per node and supports at the base of the structure only, was used to represent the overall freestanding tower structure. The nonlinear behavior of tension-only members was not accounted for in the model, and planar joints were stabilized by addition of supports or artificial members with small cross-sectional areas. Lumped masses were added to account for elements such as ladders, platforms, and communication equipment attached to the tower.

The tower model was subdivided into thirty-four substructures with no more than two substructures having a common boundary. Forty two dynamic degrees of freedom (2 translations at each of 21 different levels) were chosen to represent a condensed dynamic model of the tower. Assembly of structure stiffness and mass matrices for the reduced model follows the modified tridiagonal method except that master degrees of freedom may be chosen at any joint in the model. The method involves generation of condensed structure stiffness and mass arrays through a process of forward elimination working substructure-by-substructure from the top of the tower down to its base. Once the last substructure has been processed, these arrays remain and represent the elastic and inertial coupling, respectively, among the master degrees of freedom in the reduced tower model. The method is described in detail in Ref. 5.

Damper Model.--Resistive forces of a complicated nature are very often replaced for analysis purposes by equivalent viscous damping, expressed in the form of damping constant C_{eq}. The equivalent viscous damping constant is determined in such a manner as to approximate the dissipation of energy per cycle produced by the actual resistive forces. This methodology was adopted for use here and the effect of other forms of damping was ignored.

The internal damping present in the structure was represented by a damping matrix $\underset{\sim}{C}_I$, and the effects of add-on dampers by a matrix $\underset{\sim}{C}_D$. The total damping in the structure was then defined as the sum of $\underset{\sim}{C}_I$ and $\underset{\sim}{C}_D$. Simple modal damping with a damping ratio of 0.010 was used to form $\underset{\sim}{C}_I$ for the prototype structure. The discrete damper matrix $\underset{\sim}{C}_D$ for the entire tower structure was developed from individual member damping matrices for add-on damper elements connecting any two tower nodes. The damper device was assumed to be a part of an additional diagonal bracing element with the capability of two way action and to be able to generate a damping force proportional to the relative velocity between any two nodes i and j during either tension or compression of the element. The device was assumed to add no mass or stiffness to the structure. The damping element itself may, for example, be a conventional shock absorber with suitable valving to operate in the appropriate frequency range. Frequency and amplitude-dependent characteristics of the device were ignored for simplicity and its damping property characterized by an equivalent viscous damping constant C_{eq}. Based on calculations for several devices, a range of 0.1 to 0.5 k-sec/in (0.174 to 0.871 kN-sec/cm) for C_{eq} was selected for use in the parameter studies.

A wide variety of different damping cases was considered in the study. Results are presented only for the cases described in Table 1. Cases 0 and 2 are reference cases representing zero and 1% internal damping, respectively. In each of cases 5, 9, and 10, a series of discrete dampers of the size specified in Table 1 were uniformly distributed over the height of the tower.

Dynamic Response Analysis.--The time-history dynamic response of the prototype structure was computed for several moderate seismic loadings to evaluate the effectiveness of discrete damping devices in reducing structure response. In particular, the size, number and distribution of dampers required to attenuate structure dynamic response were sought in these studies. The eigenvalue problem was solved to determine frequencies and mode shapes for the condensed model. The direct linear extrapolation technique with the trapezoidal rule [1] was used to solve for total displacements at the dynamic degrees of freedom of the structure at each time step. The N00E component of the 1971 San Fernando earthquake (Fig. 2), recorded at Ft. Tejon, California, on February 9, at 6:00 a.m. P.S.T., was selected as the principal excitation to be used in the evaluation of damper effectiveness because of its moderate size and short duration.

RESULTS

The frequencies and mode shapes of the lower modes of the prototype structure are presented in Table 2 and Fig. 3, respectively. The displacement responses of the tower at degree of freedom 1 (see Fig. 1) to the selected seismic loading for the damping cases listed in Table 1 are superimposed in Fig. 4. In general, this figure shows that the effect of internal damping was small compared to the effect of the discrete dampers used in this study and that the effect of increasing the size of the add-on dampers was more noticeable during free vibration response than during forced-excitation response (0 to 10.4 seconds).

Two different response attenuation measures were used to compare the overall effectiveness of the different damping cases. In the first, the root-mean-square (RMS) values of the maximum responses at the master degrees of freedom were computed and are listed in Table 3. In general the RMS and maximum responses both decrease for increasing levels of overall damping. As a second measure of damper effectiveness, the logarithmic decrement, which indicates the level of decay of free vibration response, was also used to compare structure response for the different damping cases. The logarithmic decrement was taken to be a convenient measure of the overall damping effect of both internal and add-on damping, and was used because the structure was vibrating primarily in the first mode during free vibration response. The equivalent first mode damping ratios are listed in Table 3. Case 10 produced the highest damping ratio of 21.7%, a substantial increase over cases 0 and 2. Additional studies are described in Ref. 6.

CONCLUSIONS

A study of the effect of damping on the dynamic response of a free-standing communication tower of open lattice construction has been presented. Substructuring was found to be a flexible and economical means of dealing with the large number of degrees of freedom present in the structure. The effects of internal damping using the simple modal damping formulation produced results which were reasonable and expected. However, by adding discrete dampers between tower joints at various levels over the height of the tower, the overall level of damping, as indicated by several response attenuation measures, was increased, and the displacement response at the top of the structure was substantially reduced. In general, the degree of attenuation in the dynamic response of the tower depended on the size and distribution of dampers and, to a lesser extent, on the number of such add-on devices.

Future studies of the effects of add-on dampers should consider their frequency and amplitude-dependent characteristics, useful life, and effectiveness for actual loadings. Additional earthquake records and other forcing functions such as wind need to be considered in further studies.

ACKNOWLEDGMENTS

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Table 1 - Damping Cases

Case No.	Internal Damping Ratio	Add-on Dampers	
		Number	C_{eq} (k-sec/in)[a]
(1)	(2)	(3)	(4)
0	0	0	-
2	0.01	0	-
5	0.01	20	0.10
9	0.01	20	0.25
10	0.01	20	0.50

[a] 1 k-sec/in = 1.751 kN-sec/cm

Table 2 - Vibration Frequencies, In Hertz, for Prototype Structure

Mode Number	Mode Description	Frequency, in Hertz
(1)	(2)	(3)
1	1st Z translation	0.174
2	2nd Z translation	0.407
3	3rd Z translation	0.940
4	1st Y rotation	0.984
5	2nd Y rotation	1.380
6	4th Z translation	1.570
7	3rd Y rotation	1.849

Table 3 - Response Comparison

Case No.	RMS-Value (inches)[a]	Max-Response (inches)[a]	γ_{eq} (percent)
(1)	(2)	(3)	(4)
0	6.553	4.452	0.0
2	5.088	3.381	1.0
5	2.170	0.986	10.5
9	1.859	0.745	14.0
10	1.705	0.627	21.7

[a] 1 in. = 25.4 mm

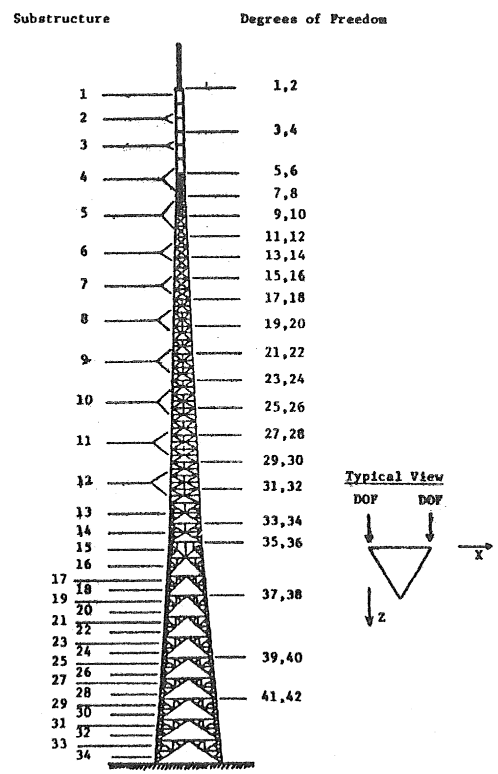

Fig. 1 42 DOF Substructure Model of Prototype Structure

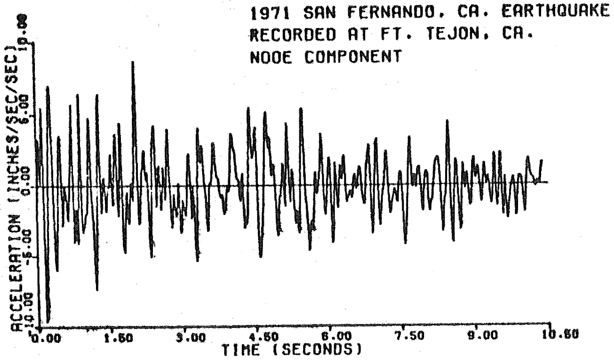

Fig. 2 Earthquake Excitation

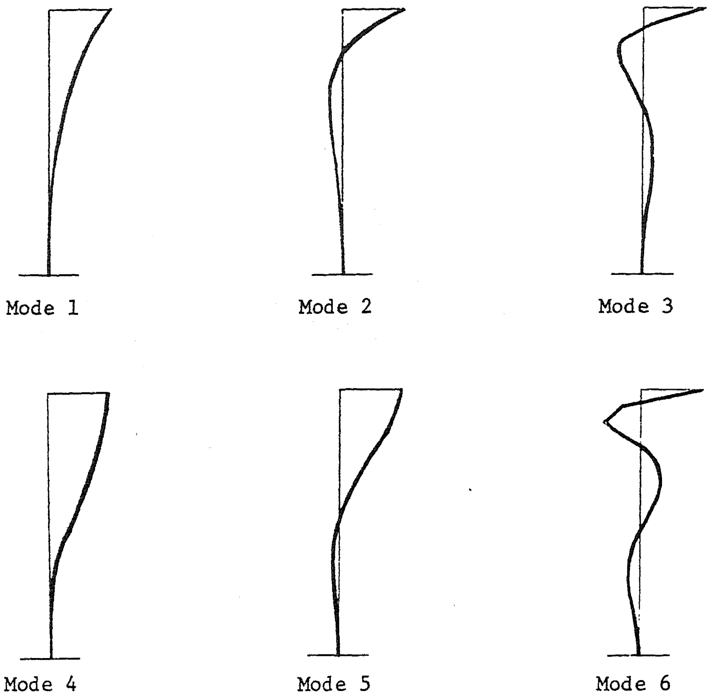

Fig. 3 Mode Shapes

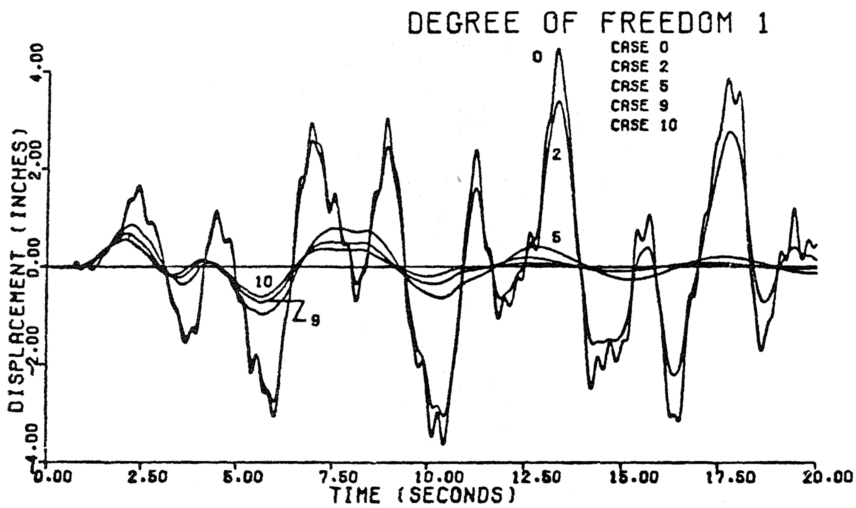

Fig. 4 Structure Response to the 10.4 second Ft. Tejon Record for Different Cases of Damping