SEISMIC CONTROL OF STRUCTURES WITH DAMPED RESONANT APPENDAGES

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ABSTRACT
Throughout the years, dampers consisting of a relatively small mass, a spring, and a dashpot attached to a point of maximum vibration and in resonance with the structure to which they are attached have been used effectively to control the response of buildings, bridges, towers, chimneys, and other structures to wind forces, machine vibrations, and occupant activity. For the most part, however, these dampers, often called vibration absorbers or tuned mass dampers, are considered to be ineffective to reduce the response of structures to earthquake loads. Thus, they have not commanded the attention of the civil engineering profession for this purpose. It is the objective of this paper to demonstrate that, in spite all previous claims to the contrary, such devices may be used effectively to control the seismic response of structures. For this, the paper presents: (a) a basic mechanism that explains under what conditions such dampers may work effectively under earthquake loads, (b) some recommendations derived on the basis of this mechanism for the selection of their masses, stiffness constants, and damping factors, and (c) the results of a series of numerical and experimental tests which amply verify that dampers designed according to these recommendations effectively and consistently reduce the response of different types of structural systems to different types of earthquake excitations. The presented numerical tests involve a two-dimensional ten-story shear building; two three-dimensional frame buildings, one with a single story and the other with ten stories; and a three-dimensional cable-stayed bridge. In the experimental studies, the structural systems considered are small-scale models of a three-story building and a cable-stayed bridge. Based on these test results, conclusions are drawn about the advantages and disadvantages of the devices and their potential to become a practical mechanism to protect structures against the devastating effects of earthquakes.

1. INTRODUCTION

Dampers consisting of a relatively small mass, a spring, and a dashpot in resonance and installed at a point of maximum vibration have been implemented effectively to reduce wind-induced vibrations in high-rise buildings [3-6] and to reduce floor vibrations induced by occupant activity [12, 14, 19]. From the practical point of view, these dampers, first suggested by Frahm in 1909 [2, 7] and often called vibration absorbers or tuned mass dampers, represent an attractive means to protect structures against the detrimental effects of dynamic loads. In comparison with other vibration control techniques, they offer two major advantages. One is that their impact on the design of the structure is only minimal since a structure with this type of device does not require special design procedures. The other is that they are easy to construct. Its construction only requires putting together a mass, a spring, and a dashpot at localized points of the structure, without the need for sophisticated hardware. Hence, its construction introduces only localized disruption and may be performed by non-specialized contractors. Additional advantages are: (a) they do not depend on an external power source for their operation; (b) they do not interfere with the principal vertical and horizontal load paths of the structure; (c) they can respond to small levels of excitation; (d) their properties can be adjusted in the field (e) they can be considered in new designs as well as in upgrading work; (f) a single unit can be effective in reducing vibrations induced by different types of dynamic loadings; (g) they require low maintenance; and (f) they can be cost effective. Up to now, however, their effectiveness under earthquake loads has been in question. Some researchers [1, 9, 11, 20, 21] have found that such damping devices may indeed reduce the response of structures to earthquake loads, and some other [8, 10, 13] altogether dismiss their effectiveness.

Such contradictory results seem to point towards a single conclusion: the effectiveness of these dampers in reducing the earthquake response of structures is highly dependent on their characteristics. That is, they work well under earthquake loads only when they possess the adequate values for their masses, stiffness constants, and damping coefficients. This conclusion in turn leads one to infer that the reason appendages have been investigated with the wrong parameters is because the mechanism that makes them work is still not well understood.

For several years now, the author and his co-workers have suggested [15-17] a basic mechanism that explains the conditions under which this type of damper may work effectively under earthquake loads, established on the basis of this mechanism recommendations for the selection of their parameters, and demonstrated through a series of numerical and experimental tests that the suggested working mechanism is indeed accurate and that dampers designed according to the established recommendations consistently reduce the earthquake response of structures. With the purpose of providing a convincing evidence that with the rightful choice of parameters appendages may reduce the response of different types of structures to different types of earthquake ground motions, this paper presents a comprehensive summary of this work. To this end, the aforementioned recommendations for the selection of effective appendages are introduced first. Then, the results from all the numerical and experimental tests that up to now have been conducted to support these recommendations and the claim that with the rightful choice of parameters appendages work effectively under earthquake loads are presented and discussed in an overall context.

2. SELECTION OF DAMPER CHARACTERISTICS

It is shown in Reference 18 that if (a) a multidegree-of-freedom appendage with a generalized mass m_a, natural frequency ω_0, and damping ratio ξ_a in one of its modes is attached to a multidegree-of-freedom structure for which the natural frequency, damping ratio, and generalized mass in one of its modes are respectively equal to $\omega_{si} = \omega_0$, ξ_{si}, and M_{si}; and (b) the parameters of the appendage and the structure are such that $|\xi_{si} - \xi_a| \leq |\Phi_{ki} \sqrt{m_a/M_{si}}|$, where Φ_{ki} denotes the value corresponding to the mass that supports the appendage in the structural mode shape with frequency ω_0, times the participation factor in this mode; then the combined structure-appendage system results in a system with two modes of vibration with a natural frequency that is very close to ω_0, the natural frequency that is common to both the structure and the appendage, and a damping ratio approximately equal to $(\xi_{si} + \xi_a)/2$; that is, the average of the damping ratios of the two independent components.

Since a damper of the type herein being discussed is nothing else but a small single-degree-of-freedom appendage in resonance with the structure in which it is installed, the possible reduction in structural response induced by such a damper occurs, according to this theory, as a result of the increase in the damping of the structure from its original value ξ_{si} to a higher value approximately equal to $(\xi_{si} + \xi_a)/2$, where ξ_a is the damping ratio of the damper. From this theory, it may also be seen that an effective damper is one that is attached to the point of the structure which undergoes the largest amplitude when the structure vibrates in its dominant mode, its fundamental natural frequency is tuned to the natural frequency of the structure in such a dominant mode, its damping ratio in its fundamental mode of vibration is high in comparison with that of the structure, and its mass, stiffness, damping ratio and location within the structure are such so that the relationship $|\Phi_{ki} \sqrt{m_a/M_{si}}|$ is equal to $|\xi_{si} - \xi_a|$. Furthermore, it may be seen that if the damping ratio for the damper is chosen to be a given value ξ_a, then the mass m_a, stiffness constant k_a, and damping coefficient c_a that make it work effectively are given by the following formulas:

$$m_a = \frac{(\xi_{si} - \xi_a)^2}{\Phi_{ki}^{\,2}} M_{si} \; ; \qquad k_a = \omega_{si}^2 m_a \; ; \qquad c_a = 2\xi_a \omega_{si} m_a \qquad (1)$$

In the studies herein reported, it is postulated that the dampers under consideration behave as resonant appendages according to the aforementioned theory. Therefore, the characteristics of the dampers used in the conducted analytical and experimental tests are selected according to these formulas.

3. NUMERICAL STUDY

3.1 Two-dimensional ten-story shear building. In this test, the 10-story shear building shown in Figure 1 is analyzed under two different earthquake records, for the cases in which (a) the building has no damper; (b) the building has a damper with 20 per cent damping attached to its roof; and (c) the building also has a damper attached to its roof, but the damping ratio of the damper is 30 per cent. This test is conducted to assess the sensitivity of the building response to small increments in the damping ratio of the damper. The building is assumed to behave elastically at all times, with a damping ratio of 2 per cent in its fundamental mode, and with a damping matrix proportional to its own stiffness matrix. The damper is modeled as an elastic single-degree-of freedom system. The earthquake records used are: (a) E-W accelerogram recorded at Foster City during the October 17, 1989, Loma Prieta, California, earthquake; and (b) S60E accelerogram obtained by combining the two components of horizontal motion recorded at SCT during the September 19, 1985, Mexico City earthquake. In each case, the natural frequency of the damper is tuned to the fundamental frequency of the building, which in this particular case equals 0.5 Hz. The mass of the damper with 20 per cent damping represents 1.4 per cent of the total mass of the building and 19.4 per cent of the roof mass, whereas for the damper with 30 per cent those percentages are 3.4 and 48.0, respectively. The results of this numerical study are summarized in Tables 1 and 2, where in each case the given reduction factors are defined as the ratio of the structural response with a damper to that with no damper.

3.2 Three-dimensional frame buildings. This test is conducted to demonstrate that resonant appendages may also work effectively for three-dimensional frame buildings, and to investigate their effectiveness when they are considered with high damping ratios. The analyzed buildings are shown in Figures 2 and 3. Their properties, dynamic characteristics, and the parameters of the dampers are given in Reference 15. Dampers with damping ratios of 40 and 80 per cent are used. In all cases, the dampers are tuned to the fundamental frequencies of the buildings, which turn out to be equal to 3.67 Hz for the one-story building and 0.43 Hz for the ten-story one. The damper is attached to the top center of the building and represented by a steel shear beam with a concentrated mass at its free end. The ground accelerations used are the first ten seconds of the following accelerograms: (1) El Centro, May 18, 1940, component N-S; (2) Taft, July 21, 1952, component N21E; and (3) Pacoima Dam, February 9, 1971, component S16E. For the one-story building, the mass of the damper equals 4.7 and 19.7 per cent of the total mass of the building, respectively for the dampers with damping ratios of 40 and 80 per cent, while for the ten-story building these values are 6.8 and 28.5 per cent. In the comparative analysis, the buildings are analyzed first with no damper, and then with one attached to their roofs. Tables 3 and 4 display the maximum floor displacements and the corresponding reduction factors obtained in each case.

3.3 Three-dimensional cable-stayed bridge. In this test, numerical simulations are conducted with a finite element model of a full-scale cable-stayed bridge. The test is carried out to show that appendages can also be effective for bridge structures and that the formulas presented above can also be employed to determine the parameters of effective dampers for bridges. The cable-stayed bridge considered is schematically described in Figure 4, and its properties given in Reference 17. The bridge has a main span of 250 m, two side spans of 121 meters, a width of 10.9 m, and a pier height of 103.6 m. The total mass of the bridge is approximately 28,802 Mg. The damping matrix of the bridge is considered to be orthogonal, with a damping ratio of 1 per cent in all its modes. The towers are assumed fixed at the foundation level. Spring, mass, and dashpot elements are used to model the components of the damper. Four different damping ratios are considered for the damper: 10, 15, 20, and 30 per cent. The ground acceleration time histories used are: (a) first ten seconds of the N-S accelerogram recorded during the May 18, 1940, El Centro earthquake; (b) E-W accelerogram recorded at Foster City during the October 17, 1989, Loma Prieta earthquake; and (c) E-W accelerogram recorded at UC Santa Cruz during this same earthquake. Because of the limitations of the computer program, the analysis is performed with no phase lag between the input motions at the bridge supports.

From a free-vibration analysis, it is found that the bridge undergoes a significant longitudinal motion in its fourth mode, that this mode exhibits a natural frequency of 0.313 Hz, and that the deck is the bridge component that undergoes the largest longitudinal displacement in this mode . The damper is thus designed to reduce the response of the bridge in such a mode. The masses of the used dampers represent about 0.67, 1.51, 2.68, and 6.04 per cent of the bridge total mass, respectively for the cases of 10, 15, 20, and 30 per cent damping. In the performance test, the bridge is analyzed first by itself and then with one of the dampers described above attached to the left end of the deck. The maximum displacements obtained in each case and the corresponding reduction factors are listed in Table 5.

4. DISCUSSION OF NUMERICAL RESULTS

The results from the numerical simulations presented in the previous section indicate that the addition of a resonant appendage to a structure in general induces a reduction in the earthquake response of the structure. It is observed that a significant reduction is obtained independently of the type of structure, and that the extent of the reduction increases with an increase in the damping ratio of the damper. This to some extent confirms that the postulated working mechanism is indeed accurate. It also confirms that the proposed formulas for the selection of the parameters of dampers lead to effective dampers, independently of the type of structure for which they are used. From the results of the shear building under the Mexico City accelerogram, it is also observed that a damper reduces not only the maximum displacements of a structure but also its interstory shears and its base shear. Likewise, it is observed that in the case of the cable-stayed bridge the damper reduces not only the longitudinal displacement of the deck, but also that of the towers.

The exception to the above observations are the shear building under the Foster City accelerogram, for in this case both the displacements and the interstory shears are not significantly reduced with the addition of any of the studied dampers. In fact, there is even a slight increase in the value of these response parameters for some of the floors. It should be noted, however, that the absence of a reduction in the response of the building under this particular ground motion does not mean that dampers may work well for some excitations but not so well for some others. It should be recalled, instead, that what the addition of a damper does to a building is to increase the damping ratio in its fundamental mode, and that damping is effective in reducing the response of a structure only when there is a significant ground motion amplification. This can indeed be verified by taking a close look at any set of response spectrum curves, and observing that the difference between the spectral ordinates for low and high damping ratios is significant only in the frequency region for which there is an important ground motion amplification. Furthermore, the fact that damping does not reduce the response of a structure equally for all possible ground excitations should not lead one to the false conclusion that a significant increase in the damping of the structure is not an effective alternative to design against earthquake forces. Damping works effectively for resonant ground motions and thus it provides protection against those ground motions that can induce serious structural damage. Since structures are usually designed using a response spectrum envelope that includes such possible resonant ground motions, an increase in the damping of the structure will permit a reduction in the ordinates of such a response spectrum envelope and, hence, significant savings in the cost of the structure.

To prove that dampers under study indeed work well to reduce the response of structures whenever they are subjected to ground motions that induce a significant response, the shear building is analyzed again under the Foster City accelerogram, but this time the fundamental natural frequency of the building is shifted to a value of 0.7 Hz, which corresponds to a frequency in the response spectrum for that accelerogram for which there is a noticeable ground motion amplification. As expected and as in the case with the Mexico City accelerogram, this time an important reduction in response is attained with the addition of the dampers. Reduction factors of the order of 0.83 are obtained with the damper with 20 per cent damping and of the order of 0.75 with the damper with 30 per cent damping.

There is another feature of the results from the numerical study that requires an explanation. That is the consistently smaller reduction in the interstory shear for the tenth story of the shear building. An explanation for this is that in addition to the reduction in response caused by the damper, there is also an increase in the interstory shears caused by the additional shear force on the damper mass. Since the interstory shear in the top story is always the smallest, the increase in interstory shear for this story will represent a large percentage over its original value. As a result, the top story will also be the one that will experience the smallest reduction in interstory shear.

5. EXPERIMENTAL STUDY

5.1 Building model. To verify experimentally the conclusions from the numerical study, a small wooden structural model is built and tested in a shaking table under random and sinusoidal excitations with and without a damper. The structural model built is 915 mm in height and represents a three-story building structure. Plywood, 13-mm thick, is used for the floors and dowels, 6 mm in diameter, for the columns. Figure 5 shows the configuration and dimensions of the model. To add mass to the floors, steel weights are attached to each story. The total floor masses are: 2.16 kg for the top story, 1.74 kg for the middle story, and 1.74 kg for the bottom one. The experimentally obtained values for the natural frequency and damping ratio of the model in its fundamental mode are 2.18 Hz and 3.95 %, respectively. The damper is designed as a single-degree-of-freedom system consisting of a mass attached to a spring and a dashpot and for a target damping ratio of 53.5 per cent. The total mass of the damper is 0.77 kg, which represents 14 per cent of the total structural model mass.

The structure is placed on a shaking table and tested first under random excitations generated by a HP3562A dynamic signal analyzer. The structure is tested over a period of time after which averages are taken and frequency response curves are obtained. The testing is repeated with the damper attached to the top story and new frequency response curves are obtained. The frequency response curves obtained in each case are shown in Figure 6.

To corroborate experimentally the finding from the numerical study that the reduction in response induced by a damper is the largest when the dominant frequency of the exciting ground motion is close to the natural frequency of the structure, the shaking table experiment is extended to include tests under different sinusoidal excitations. Sinusoidal excitations with a frequency of 1.5, 2.0, and 3.0 Hz are considered. The frequency of 2.0 Hz represents a frequency close to the fundamental natural frequency of the structural model. On the other hand, the frequencies of 1.5 and 3.0 Hz are relatively far from the fundamental frequency of system, with the frequency of 3.0 Hz being the one that is the farthest apart. The reductions in the acceleration response of the model's top floor obtained in these tests are 45.2, 5.9, and 2.9 percent respectively for the excitations with frequencies of 2.0, 1.5 and 3.0 Hz.

5.2 Bridge model. In this test, a small-scale cable-stayed bridge and a damper are built and tested on a pair of shaking tables under various base acceleration time histories. The bridge model is 3.7-m long, made of aluminum, and designed as a simple span bridge with its cables supported by two towers 0.79 m in height. The cables are arranged over two parallel planes, using a harp configuration. Its geometry and dimensions are given in Figure 7, and the characteristics of its components in Reference 17. To add mass to the bridge, 0.45 kg steel weights are attached along the bottom of the deck and along the sides of the towers. Tension is applied to the cables to keep the deck straight under its own weight and the added weights. The abutments rest on ball bearings and are attached to the towers by means of relatively rigid braces to make them undergo the same base motion as the towers. The total mass of the bridge model, without the abutments and the braces that join these to the towers, is approximately 18.7 kg.

From a finite element analysis, it is observed that the mode with the most significant longitudinal motion is the first one and that the deck is the bridge component that displaces the most along such a longitudinal direction. Therefore, the damper is designed to damp the bridge's first mode with the purpose of minimizing this longitudinal motion. The natural frequency and damping ratio of the bridge model in this mode turn out to be equal to 6.23 Hz and 7 per cent, respectively.

The damper is designed for a target damping ratio increment of 25 per cent. On the basis of the results from the numerical study, this value is selected so that, on the one hand, it is high enough to secure a significant damping augmentation, but, on the other hand, it is low enough to keep the damper mass within practical limits. The damper is built with a weight attached to a spring and an air cylinder in parallel. The mass of the damper represents about 8 per cent of the total mass of the bridge.

For the performance test, the bridge model is mounted on the shaking tables as depicted in Figure 7. The model is tested first by itself and then with the damper attached to the middle of the deck. In both tests, the two supports of the model are subjected to the same base motions, these being the El Centro, Foster City, and UC Santa Cruz ground acceleration records described in the previous section, but with the accelerations scaled by factors of 0.5, 0.7, and 0.4, respectively, During each test, the accelerations at a point on the deck and the top of one of the towers are measured and recorded. Table 6 lists the measured maximum accelerations in each case and the corresponding reduction factors.

6. DISCUSSION OF EXPERIMENTAL RESULTS

The results from the experimental tests validate the conclusions from the numerical study. A comparison between the two frequency response curves in Figure 6 shows that the addition of the damper to the building model induces a reduction of 38.3 percent in the ordinate that corresponds to the fundamental frequency of the model. A reduction is observed too in the value for the third natural frequency. In contrast, a slight increase is observed for the second mode, which, given that this mode is a torsional one, could have been the result of an increase in the moment of inertia of the system when the damper is added to its top story. In the test with the sinusoidal excitations, the building model also responded as expected. At 2.0 Hz, the input frequency that is the closest to the fundamental frequency of the model, the damper performed at its best. When the input frequency is reduced to 1.5 Hz, the damper also induces a reduction in that response, but not to the same extent as at 2.0 Hz. At 3.0 Hz, a reduction is still attained, but it is an insignificant one. In the test with the cable-stayed bridge, it is found that a damper with a weight equal to 8 per cent of the total weight of the bridge model and a damping ratio of 32 per cent reduces the peak longitudinal response of the model's deck by 41 per cent and that of the top of its towers by 12 per cent. Hence, this test corroborates the conclusions from the numerical study in that dampers may also be effective for bridge structures and that Equations 1 can also be employed to determine the characteristics of effective dampers for bridges.

7. CONCLUSIONS

The results of the numerical and experimental studies show that resonant appendages may indeed be suitable to reduce the earthquake response of structures. It also discloses three drawbacks associated with their use. The first is the size of the damper mass that is needed to attain a substantial response reduction. It is observed that the reduction attained with a appendage increases with its damping ratio, but at the same time its mass also increases with this damping ratio. The second is the uncertainty in the tuning of the damper to the desired structural frequency. The effectiveness of a appendage diminishes when it is not perfectly tuned to the structure. The third is that there exists a dependence of the attained reduction in response on the characteristics of the ground motion that excites the structure. This reduction in response is large for resonant ground motions and is progressively less as the dominant frequency of the ground motion gets further apart from the structure's natural frequency to which the damper is tuned.

At first sight, these three drawbacks may appear to be an obstacle to the practical application of the studied dampers; that is, they can mislead one to believe that an excessively large mass is needed to attain a large reduction factor, that the system might not work well because in a practical implementation it is difficult to predict with certainty the natural frequencies of the structure, and that the device might be effective only for some but not all possible earthquake excitations. It is believed, however, that these drawbacks are only so in appearance, for they can be overcome in the design process. For instance, in regard to the size of the damper mass, the designer should borne in mind that what appendage does to a structure is simply an augmentation of its damping. As such, since beyond a certain limit additional damping will not significantly reduce a structure's response any further, a high damping ratio, and hence a large damper mass, will not be necessary in most cases. Moreover, for a given structure a designer can always find ways to minimize the mass needed for the construction of the damper by using the mass of some parts of the structure or some of its appurtenances. Similarly, in weighing the use of appendage as a possible design solution, the designer can consider the uncertainty in the values of the calculated natural frequencies of the structure and reduce accordingly the effectiveness of the damper. In regard to the ground motion characteristics, it important to keep in mind that the damper that is needed is one that will be effective under those ground motions that in the absence of damping would induce a large structural response and, hence, under those critical ground motions that govern the design of the structure.

The major conclusion from this study seems to be thus that, after all, resonant appendages may also work effectively to control the earthquake response of structures and have the potential to become a practical mechanism to protect structures against the devastating effects of earthquakes.

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Table 1. Maximum floor displacements and interstory shears of 10-story shear building and corresponding reduction factors (R.F.) under Mexico City accelerogram

Floor	Maximum displacements			Maximum interstory shears		
	Displ. without damper (m)	R. F. with 20% damping damper	R. F. with 30% damping damper	Interstory shear without damper (MN)	R. F. with 20% damping damper	R. F. with 30% damping damper
1	0.024	0.67	0.58	1.490	0.69	0.59
2	0.048	0.69	0.58	1.430	0.69	0.58
3	0.072	0.69	0.58	1.330	0.69	0.59
4	0.094	0.69	0.59	1.210	0.69	0.59
5	0.116	0.69	0.59	1.060	0.70	0.60
6	0.135	0.69	0.59	0.887	0.70	0.62
7	0.151	0.70	0.60	0.707	0.71	0.65
8	0.164	0.70	0.60	0.523	0.73	0.69
9	0.173	0.70	0.61	0.341	0.78	0.77
10	0.178	0.70	0.61	0.165	0.92	1.03

Table 2. Maximum floor displacements and interstory shears of 10-story building and corresponding reduction factors (R.F.) under Foster City accelerogram

Floor	Maximum displacements			Maximum interstory shears		
	Displ. without damper (m)	R. F. with 20% damping damper	R. F. with 30% damping damper	Interstory shear without damper (MN)	R. F. with 20% damping damper	R. F. with 30% damping damper
1	0.042	0.95	0.93	2.620	0.97	0.94
2	0.080	0.96	0.95	2.250	0.97	0.99
3	0.114	0.96	0.97	1.900	1.04	1.06
4	0.143	0.98	1.00	1.760	1.04	1.06
5	0.167	1.01	1.04	1.690	1.02	1.02
6	0.191	1.03	1.05	1.570	1.01	1.01
7	0.215	1.03	1.06	1.450	0.95	0.95
8	0.235	1.04	1.06	1.240	0.94	0.90
9	0.253	1.03	1.06	0.891	0.96	0.91
10	0.263	1.03	1.05	0.458	0.98	1.00

Table 3. Maximum roof displacements of 1-story frame building and corresponding reduction factors (R.F.)

El Centro			Taft			Pacoima Dam		
Displ. with no damper (m)	R. F. with 40% damping damper	R. F. with 80% damping damper	Displ. with no damper (m)	R. F. with 40% damping damper	R. F. with 80% damping damper	Displ. with no damper (m)	R. F. with 40% damping damper	R. F. with 80% damping damper
1.50	0.83	0.81	0.89	0.79	0.62	6.00	0.69	0.38

Table 4. Maximum floor displacements of 10-story frame building and corresponding reduction factors (R.F.)

Floor	El Centro			Taft			Pacoima Dam		
	Displ. without damper (cm)	R. F. with 40% damping damper	R. F. with 80% damping damper	Displ. without damper (cm)	R. F. with 40% damping damper	R. F. with 80% damping damper	Displ. without damper (cm)	R. F. with 40% damping damper	R. F. with 80% damping damper
1	8.46	0.87	0.55	2.04	0.91	0.61	13.03	0.89	0.77
2	14.02	0.87	0.55	3.35	0.91	0.61	21.23	0.89	0.79
3	18.45	0.87	0.55	4.31	0.91	0.64	27.32	0.89	0.80
4	22.51	0.86	0.55	5.11	0.91	0.65	32.47	0.88	0.83
5	26.00	0.85	0.55	5.75	0.90	0.66	36.99	0.89	0.85
6	29.09	0.86	0.55	6.36	0.89	0.65	42.48	0.90	0.86
7	32.44	0.85	0.54	7.38	0.86	0.66	50.57	0.90	0.84
8	35.20	0.82	0.53	8.28	0.86	0.67	57.70	0.90	0.82
9	38.53	0.84	0.53	9.36	0.84	0.67	65.55	0.89	0.82
10	40.72	0.84	0.51	10.23	0.83	0.67	70.58	0.88	0.82

Table 5. Maximum longitudinal displacements of bridge deck end and tower top and corresponding reduction factors (R.F.)

Accelerogram	Bridge deck end					Tower top				
	Displ. without damper (m)	R. F. with 10% damping damper	R. F. with 15% damping damper	R. F. with 20% damping damper	R. F. with 30% damping damper	Displ. without damper (m)	R. F. with 10% damping damper	R. F. with 15% damping damper	R. F. with 20% damping damper	R. F. with 30% damping damper
El Centro	0.333	0.76	0.71	0.65	0.53	0.455	0.67	0.64	0.59	0.50
Foster City	0.445	0.85	0.73	0.70	0.69	0.453	0.88	0.80	0.79	0.78
U.C. Santa Cruz	0.462	0.12	0.12	0.12	0.13	0.453	0.14	0.15	0.15	0.16

Table 6. Maximum accelerations in deck and left tower top without and with damper in experimental test with cable-stayed bridge and corresponding reduction factors (R.F.)

Accelerogram	Horizontal acceleration in g of deck left end with no damper	R.F. with damper	Horizontal acceleration in g of left tower top with no damper	R.F. with damper	Vertical acceleration in g of deck center with no damper	R.F. with damper
El Centro	0.077	0.59	0.103	0.94	0.107	0.91
Foster City	0.060	0.68	0.059	0.88	0.071	0.74
U.C. Santa Cruz	0.095	0.65	0.074	0.97	0.112	0.69

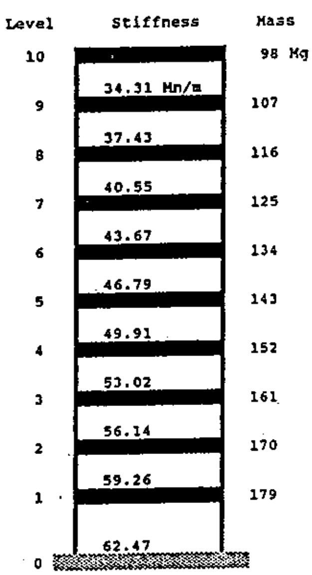

Figure 1. Ten-story shear building in numerical study

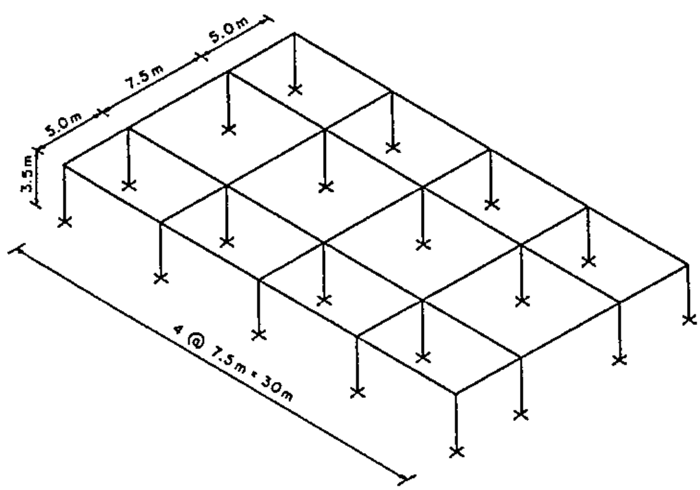

Figure 2. One-story three-dimensional frame building

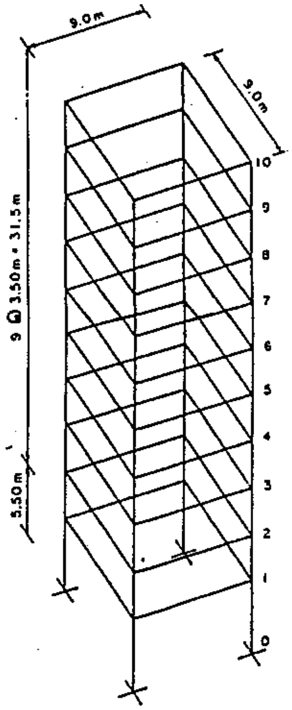

Figure 3. Ten-story three-dimensional frame building

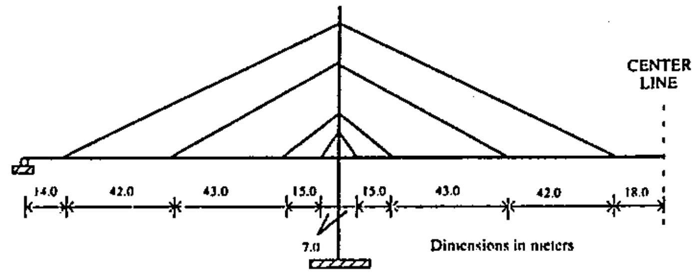

Figure 4. Elevation of bridge model in simulation study

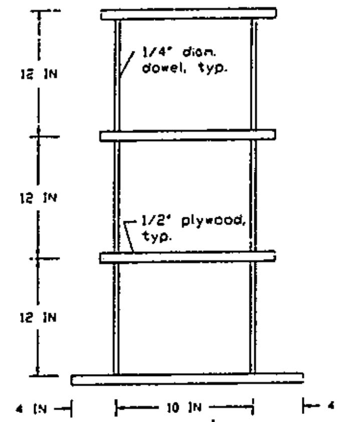

Figure 5. Building model in experimental study

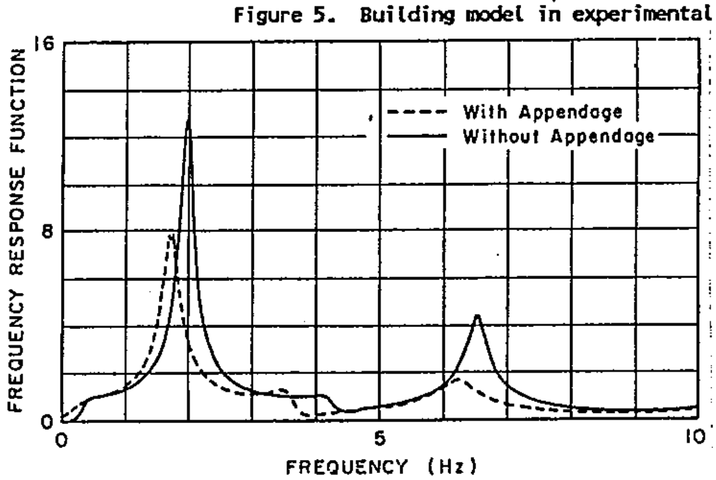

Figure 6. Frequency response functions of building model without and with damper

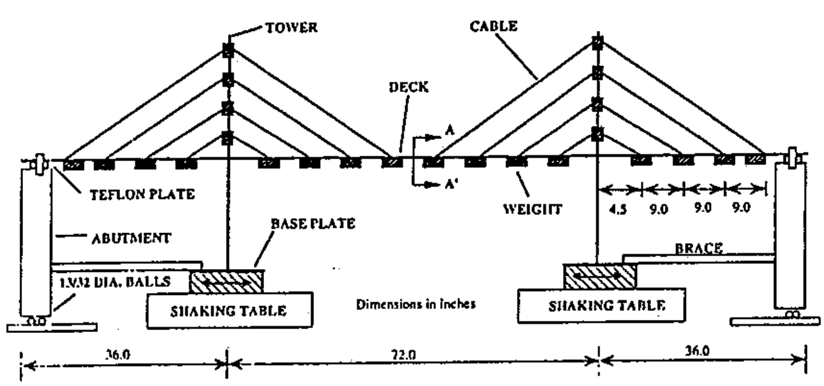

Figure 7. Configuration and dimensions of bridge model in experimental study