Wind and Structures, Vol. 2, No. 3 (1999) 201-251

Mitigation of motions of tall buildings with specific examples of recent applications

Ahsan Kareem[†] and Tracy Kijewski[‡]

NatHaz Modeling Laboratory, Department of Civil Engineering and Geological Sciences, University of Notre Dame, Notre Dame, IN 46556-0767, U.S.A.

Yukio Tamura[†]

Tokyo Institute of Polytechnics, Atsugi 243-02, Japan

Abstract. Flexible structures may experience excessive levels of vibration under the action of wind, adversely affecting serviceability and occupant comfort. To ensure the functional performance of a structure, various design modifications are possible, ranging from alternative structural systems to the utilization of passive and active control devices. This paper presents an overview of state-of-the-art measures that reduce the structural response of buildings, including a summary of recent work in aerodynamic tailoring and a discussion of auxiliary damping devices for mitigating the wind-induced motion of structures. In addition, some discussion of the application of such devices to improve structural resistance to seismic events is also presented, concluding with detailed examples of the application of auxiliary damping devices in Australia, Canada, China, Japan, and the United States.

Key words: damping; auxiliary damping devices; tuned mass damper; tuned liquid damper; hybrid mass damper; active mass damper, aerodynamic modifications; structural systems; wind-induced motion; structural control; earthquakes; turbulence; dynamics; buildings; towers.

1. Introduction

The race toward new heights has not been without its challenges. With the invention of E. G. Otis' elevator and the introduction of structural steel, towers and skyscrapers have continued to soar skyward, where they are buffeted in the wind's complex environment. Unfortunately, these advances in height are often accompanied by increased flexibility and a lack of sufficient inherent damping, further increasing the structure's susceptibility to the actions of wind. Major innovations in structural systems have permitted these increased lateral loads to be efficiently carried; however, the dynamic nature of wind is still a factor, causing discomfort to building occupants and posing serious serviceability issues. The next generation of tall building research has been devoted in part to the mitigation of such wind-induced motions via global design modifications to the structural system or building aerodynamics and

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† Professor & Chair

‡ Graduate Student

Table 1 Means to suppress wind-induced responses of buildings

Means	Type	Method and Aim	Remarks
Aerodynamic Design	Passive	Improving aerodynamic properties to reduce wind force coefficient	Chamfered corners, openings
Structural Design	Passive	Increasing building mass to reduce air/building mass ratio	Increased Material Costs
		Increasing stiffness or natural frequency to reduce non-dimensional windspeed	Bracing Walls, Thick Members
Auxiliary Damping Device	Passive	Addition of materials with energy dissipative properties, increasing building damping ratio	SD, SJD, LD, FD, VED, VD, OD
		Adding auxiliary mass system to increase level of damping	TMD, TLD
	Active	Generating control force using inertia effects to minimize response	AMD, HMD, AGS
		Generating aerodynamic control force to reduce wind force coefficient or minimize response	Rotor, Jet, Aerodynamic Appendages
		Changing stiffness to avoid resonance	AVS

SD: Steel Damper; SJD: Steel Joint Damper; LD: Lead Damper; FD: Friction Damper; VED: Visco-Elastic Damper; VD: Viscous Damper; OD: Oil Damper; TMD: Tuned Mass Damper; TLD: Tuned Liquid Damper; AMD: Active Mass Damper; HMD: Hybrid Mass Damper; AGS: Active Gyro Stablilizer; AVS: Active Variable Stiffness

through the incorporation of auxiliary damping systems, as summarized in Table 1. The following study encompasses the entire spectrum of techniques from aerodynamic tailoring to auxiliary damping devices, geared specifically toward reducing the toll of winds on structures, particularly those which affect occupant comfort.

In addition to their applications in Australia, Canada, China, Japan, and the United States for the mitigation of wind-induced motions, auxiliary damping devices have also gained much recognition for their performance in seismic regions. Thus, while treatment will be given primarily to wind-sensitive structures which utilize these technologies, seismic applications are also presented.

2. Perception criteria

The design of typical structures requires the engineering of a system that efficiently and effectively carries the anticipated lifetime loads. In this sense, a structure may be designed to meet some functional purpose without any regard for the human element; however, this element is a critical consideration in high-rise construction. With increasing height, often accompanied by increased flexibility and low damping, structures become even more susceptible to the action of wind, which governs the design of the lateral system. While a given design may adequately carry all loads, the structure may still suffer from levels of motion causing significant discomfort to its occupants. Therefore, many aerodynamic and structural design modifications such as the addition of an auxiliary damping system are explicitly incorporated to improve the structure's ability to meet serviceability or perception criteria. Before discussing the techniques to mitigate these wind-induced motions, a review of the various perception criteria is provided.

The response of high rise buildings to wind is typically comprised of two lateral components, i.e., alongwind and acrosswind, and a torsional component (Kareem 1985). The alongwind motions result from the pressure fluctuations on the windward and leeward faces of the structure and generally follow the fluctuations in the approach flow in the low frequency range. The acrosswind motion results from pressure fluctuations on the side faces, which are influenced by fluctuations in separated shear layers and wake dynamics. The torsional response, which may be further amplified in geometrically and structurally asymmetric buildings, results from differences in the instantaneous pressure distribution on the building faces.

Understandably, any of these motions may cause discomfort to the structure's occupants and may trigger responses analogous to those associated with motion sickness. While the response of each person varies, symptoms may range from concern, anxiety, fear, and vertigo to extreme responses of dizziness, headaches, and nausea. As a result, numerous studies have been devoted to determining the thresholds marking the onset of these sensations, which vary with each individual.

Perception limits have traditionally been determined based on the response of individuals to tests using motion simulators (Chen and Robertson 1973, Irwin 1981, Goto 1983, AIJ 1991). In most cases, such experiments rely on sinusoidal excitations; however, there appear to be some discrepancies between these testing environments and those of actual structures (Isyumov 1993). Since the motion of the structure is a narrowband random excitation inducing bi-axial and torsional responses, the use of uni-axial sinusoidal motions is questionable. In addition, the absence of visual and audio cues in the test environment neglects critical stimuli, particularly for torsional motions which are known for triggering visual stimulus.

From such studies of the population's thresholds for perception, criteria are defined as levels of motion which may be exceeded in a particular recurrence interval. Typically, in North America, a ten-year interval is used; however, in regions with frequent typhoons and hurricanes, the use of a shorter recurrence interval, e.g., one year, may be necessary. Fig. 1 illustrates some of the

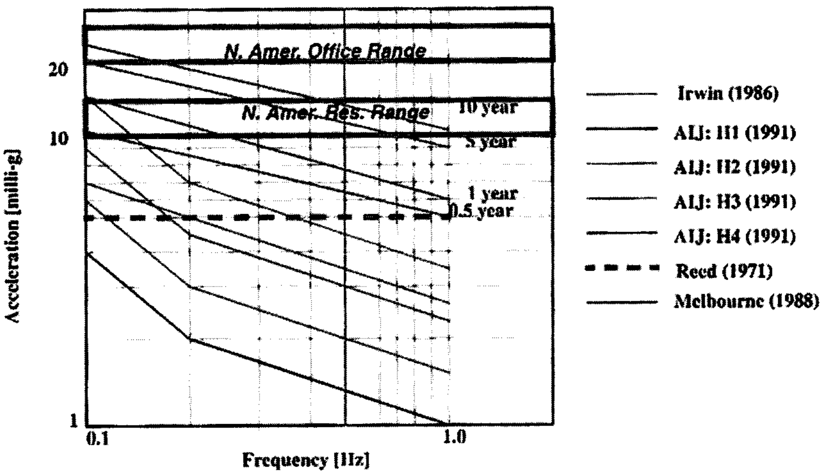


Fig. 1 Various perception criteria for occupant comfort

perception criteria which are currently in use. Note that in North America, the typical practice is to use 10-15 milli-g peak horizontal accelerations at the top floor for residential buildings, and 20-25 milli-g for office buildings, based upon a ten-year recurrence interval (Isyumov 1993). Kareem (1988a) proposed an rms acceleration threshold of 8 to 10 milli-g's for a ten-year recurrence interval. The lines labeled H1-H4 are taken from the Japanese AIJ standards (AIJ 1991) and represent various levels of peak acceleration perception, with H-2 typically used for residential applications and H-3 for office buildings. The light blue lines represent an equation for peak acceleration proposed by Melbourne and Cheung (1988) based in part upon the previous findings of several researchers. The expression is derived from the maximum response observed during a ten minute interval for various recurrence intervals. Also shown is Reed's (1971) constant perception limit of 5 milli-g's for a six-year recurrence interval and Irwin's (1986) E2 curve for rms accelerations, which is also given in ISO6897 (ISO 1984), illustrating the difference between the use of rms and peak accelerations.

Criteria based on rms accelerations, as opposed to peak accelerations, offer a more accurate means of combining the response in different directions based on their respective correlations (Kareem 1992). Using the peak acceleration criterion, the first peaks in each direction are determined and subsequently combined by an empirical combination rule; however, care must be exercised since different response components may have a different probability structure, requiring different peak factors. Further discussions have revealed that the jerkiness of the structural response may be primarily responsible for the perception of motion. Quite simply, while humans are capable of adjusting to accelerations, any change in the acceleration will require additional adjustments for equilibrium. As a result, basing perception criteria on a measure of rms jerk, or the rate of change of acceleration, would better capture the stimulus which defines our perception thresholds under random motion.

In addition, frequency-dependent motion perception threshold criteria (Melbourne and Palmer 1992) and criteria which take into account the probabilistic distribution of human perception limits are also being considered (Kareem 1987). In particular, the frequency dependence of perception thresholds becomes critical, since there is evidence that perception levels increase with decreasing frequency.

3. Structural systems

In light of human perception and serviceability concerns, a host of techniques have been developed to mitigate the discomforting motions induced by wind. In addition to the rudimentary design of structural systems to efficiently carry lateral loads, certain features can be engineered to improve the structure's performance under the action of wind. If seismic effects are not a concern, increasing the building's mass will reduce the natural frequency; however, this modification increases the non-dimensional wind speed (V/nB). Therefore, this trade-off relation can occasionally increase the input wind force energy and increase the response. However, it is very difficult and unrealistic to increase the building's mass, considering the resulting amplification of the seismic inertia force and the costs associated with supporting the additional mass.

On the other hand, fundamental dynamics proves that increases in stiffness will provide reductions in the amplitude of motion, but will not affect accelerations which comprise the stimulus for motion perception. Furthermore, by stiffening the structure, the jerk component, another contributing factor to motion stimulus, may increase. Therefore, the selection of an efficient structural system must include an evaluation of its ability to resist lateral wind loads with minimum jerk and acceleration levels for the upper floors.

Despite all of these considerations, selecting an efficient structural system can provide the most effective means of controlling a structure's response to wind in the lateral and torsional directions. This may be accomplished through any number of systems including space frames and mega frame systems, or through the addition of vierendeel frames, belt trusses, super columns, vierendeel-type bandages and outrigger trusses. A structural system can also benefit from concrete or composite steel/concrete construction with higher inherent damping. For example, the Petronas Towers in Kuala Lumpur utilized a concrete structural system which helped to improve the serviceability performance of the towers. The application of some of these strategies are highlighted in the following sections.

3.1. Outrigger systems

The use of outrigger systems, which are illustrated Fig. 2a, has become a popular approach for improving the efficiency of the core system by simply engaging the exterior columns to aid in resisting part of the overturning moment resulting from lateral loads. While 35~40 story buildings can typically rely solely on shear wall and steel-braced core systems, which are very effective in resisting the forces and deformations due to shear racking, the resistance of these systems to the overturning component of drift decreases approximately with the cube of building height (CTBUH 1995). As a result, core systems become highly inefficient for taller skyscrapers. The incorporation of outrigger walls or trusses, often two to three stories deep, can overcome such restrictions by transferring some of the loads to the exterior frame.

The incorporation of such systems has proven successful for a host of the world's tallest buildings, including the proposed 560 m Melbourne Tower. The project, shown in Fig. 2b, features two-story deep outrigger trusses every 20 stories to aid in carrying lateral loads (Civil Engineering 1999).

3.2. Belt/bandage systems

The outrigger concept has been modified by using belt walls/trusses as "virtual outriggers," as shown schematically in Fig. 3a, accomplishing the same load transfer without requiring the complicated direct connection between the outrigger system and the core (Nair 1998). The concept relies upon stiff floor diaphragms to transfer the moment in the form of a horizontal couple from the core to the belt wall/truss, which connects the exterior columns of the structure. The wall/truss then converts the horizontal couple into a vertical couple in the exterior columns. This "virtual outrigger" system, utilizing belt walls, has been applied to the world's tallest reinforced concrete building, the 77-story Plaza Rakyat (Fig. 3b) office tower in Kuala Lumpur, Malaysia (Baker *et al.* 1998). The structure relies on a concrete shear core and two-story exterior concrete belt walls connected to the concrete perimeter frame at two levels to carry the lateral loads without the restriction of mechanical space, a consequence of using the conventional outrigger system.

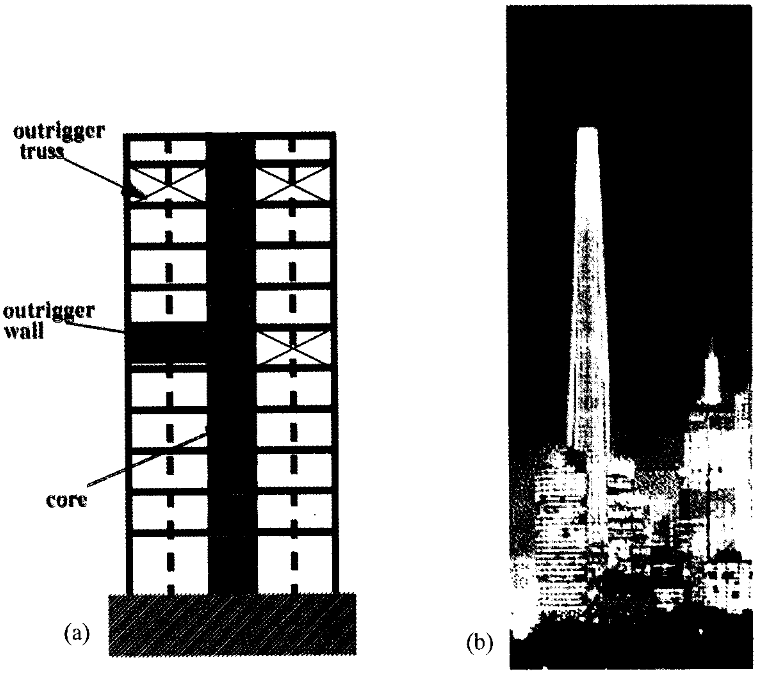


Fig. 2 (a) Schematic representation of outrigger system; (b) Composite sketch of Melbourne Tower (taken from Denton, Corker, Marshall)

A similar concept, the Vierendeel bandage, shown in Fig. 4, has been implemented in the 236 m tall First Bank Place in Minneapolis (Dorris 1991). The tower, which is supported by a cruciform spine with steel columns and four massive composite supercolumns, required diagonal bracing due to its lack of sufficient torsional stiffness. However, to permit unobstructed views, a series of three-story tall, 36-inch deep Vierendeel bandages were implemented. The addition of the bandages triples the tower's torsional stiffness while improving the lateral stiffness by 36%. In

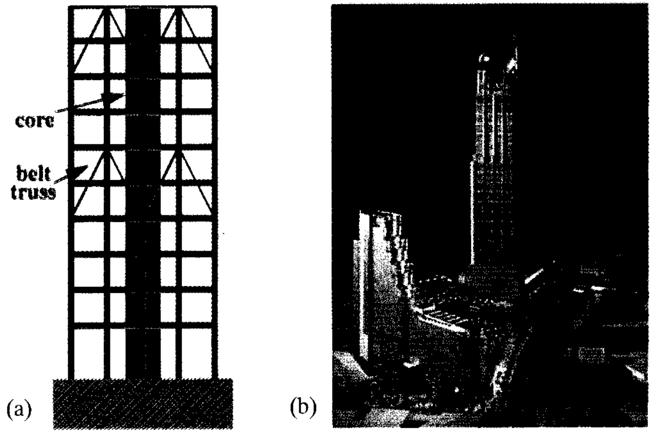


Fig. 3 (a) Illustration of "virtual outrigger" system using belt trusses; (b) Model of Plaza Rakyat (taken from Skidmore, Owings and Merrill, LLP)

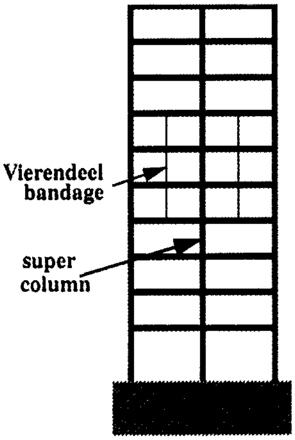


Fig. 4 Schematic of Vierendeel bandage

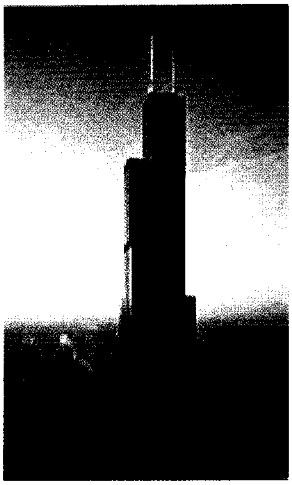


Fig. 5 Sears Tower (taken from Skidmore, Owings, and Merrill, LLP)

addition, the bandages carry the load from the upper floors and transfer it to the four major columns at the corners.

3.3. Tube systems

One trademark of high rise construction in the late twentieth century has been the use of tube systems. From the innovative designs of Fazlur Khan, who developed both the bundled and braced tube concepts, tube systems have served as a successful lateral load resisting system, comprised of a series of closely spaced exterior columns and deep spandrel beams held rigidly together (CTBUH 1995). The use of such systems became quite popular following their introduction in landmark structures such as the Sears Tower (shown in Fig. 5), World Trade Center Towers and John Hancock Center.

The concept is continually being extended in the construction of modern skyscrapers such as the Shanghai World Financial Center, (shown later in Fig. 9) which is scheduled for completion in 2001. This design features the tube-in-tube or double tube system, which consists of both an exterior composite tube of structural steel frame with reinforced concrete and an interior tube provided by a reinforced concrete core. Under the action of wind, 15 to 40% of the shear force is resisted by the interior tube, which justifies the use of the double tube system for reducing the wind loading (Hori and Nakashima 1998), a major concern in light of the frequent typhoons in Shanghai. Further discussion of the incorporation of aerodynamic modifications and auxiliary damping devices in this structure is provided in subsequent sections.

3.4. Increasing modal mass

Other options for improving building performance in high winds may include shifting the major frequency axes from the main axes of the building shape and altering mode shapes to create an increased modal mass in the structure's upper floors (Banavalkar 1990). The latter technique can markedly improve occupant comfort since wind-induced accelerations are inversely proportional to the effective mass. For example, this approach was applied to the Washington National Airport Control Tower. By eliminating transfer girders at the base and mounting the tower on a 3 m deep pyramidal truss, base rotation of the tower was eliminated and the effective mass of the tower was increased, thereby reducing the dynamic response of the tower (Banavalkar and Isyumov 1998).

4. Aerodynamic modifications

The specific concern for wind-induced effects has prompted much investigation into the relationship between the aerodynamic characteristics of a structure and the resulting wind-induced excitation level. Often aerodynamic modifications of a building's cross-sectional shape, the variation of its cross-section with height, or even its size, can reduce the building's motion (Kwok and Isyumov 1998). Such aerodynamic modifications include slotted and chamfered corners, fins, setbacks, buttresses, horizontal and vertical through-building openings, sculptured building tops, tapering, and drop-off corners (Kareem and Tamura 1996). These modifications are discussed below.

4.1. Modifications to corner geometry and building shape

Initiatives to explore the effects of building shape on aerodynamic forces have confirmed the benefits of adjusting building configurations and corners, as illustrated in Fig. 6 (Hayashida and Iwasa 1990, Hayashida *et al.* 1992, Miyashita *et al.* 1993, Shimada *et al.* 1989). Investigations have established that corner modifications such as chamfered corners, horizontal slots, and slotted corners can significantly reduce the alongwind and acrosswind responses as compared to a basic building shape (Kwok 1995). Significant rounding of the structure's corners has been shown to significantly improve the response of the structure. Such modifications were applied to the 150 m Mitsubishi Heavy Industries Yokohama Building (Fig. 7a) which was erected in a water front area in the wake of peripheral tall buildings. To reduce the response, each of the four corners were chamfered, which consequently reduced the wind forces (Miyashita *et al.* 1995).

Still, there is no definitive consensus regarding the benefits of corner geometry modifications, since studies have also shown that modifications to building corners were ineffective and even had adverse effects (Miyashita *et al.* 1993, Kwok and Isyumov 1998).

Improved acrosswind responses have also been observed in tall buildings whose cross-sectional shape varies with height or whose upper level plan areas are reduced through tapering effects, cutting corners, or progressively dropping off corners as height increases. As illustrated in Fig. 7b, changing the cross sectional shape along the vertical axis, coupled with effective tapering, can be especially effective in reducing the acrosswind forces (Shimada and Hibi 1995). These results have been confirmed in other works and imply that the more sculptured a building's top is, the better it can minimize the alongwind and acrosswind responses. Fig. 8 illustrates the use of such geometries in two recent projects: the Jin Mao Building (Fig. 8a) in China and the Petronas Towers (Fig. 8b) in Malaysia. The Jin Mao Building uses setbacks and tapering of its 421 m facade. In addition, the building is crowned by ornate tiers shifted from the major axis of the structure, creating an effect reminiscent of the ancient pagoda. Similarly, the

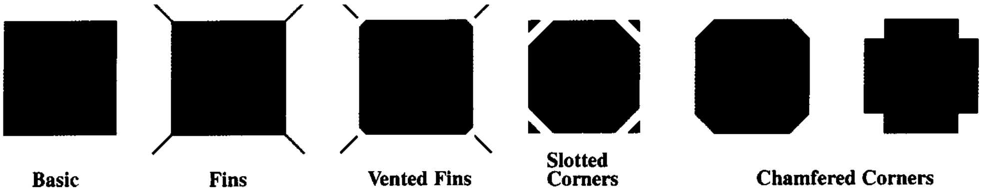


Fig. 6 Aerodynamic Modifications to Square Building Shape

benefits of tapering were integrated into the design of the 450 m twin Petronas Towers.

4.2. Addition of openings

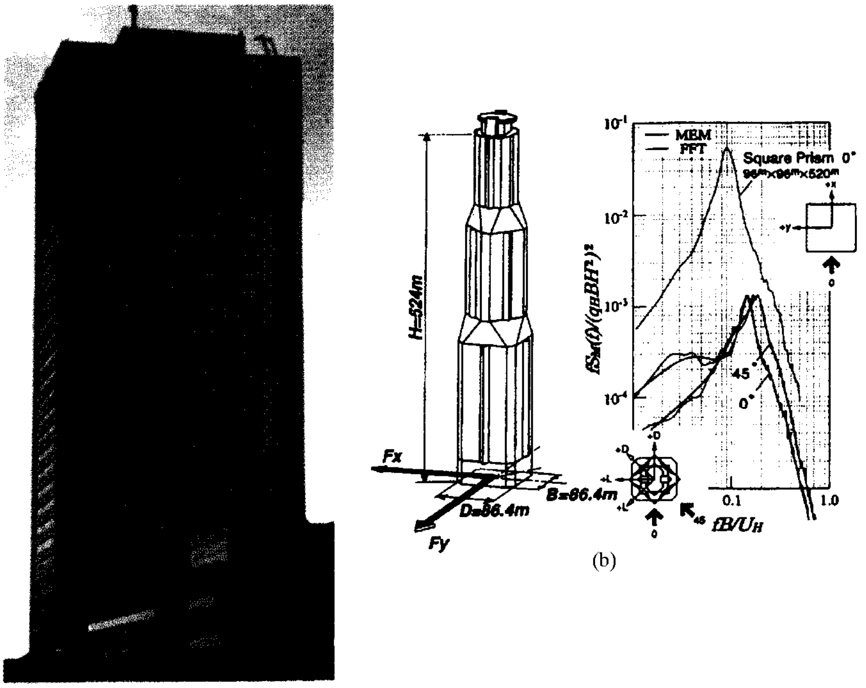
The addition of openings (Miyashita *et al.* 1993, Irwin *et al.* 1998) in a building provides yet another means for improving the aerodynamic response of that structure. However, this approach, like any aerodynamic modification, must be used with care to avoid adverse effects. Openings that extend completely through the building, particularly near the top, have been found to significantly reduce vortex shedding-induced forces, and hence the acrosswind dynamic response, shifting the critical reduced wind velocity to a slightly higher value (Dutton and Isyumov 1990, Kareem 1988b). However, the effectiveness of this modification diminishes if the openings are placed at lower levels of the building. In addition, the inclusion of openings and other such modifications may adversely affect habitability if they reduce the resonant vortex frequency (Tamura 1997).

Through-building openings have been used in several buildings in Japan and are being currently applied to the proposed new world's tallest building, the Shanghai World Financial Center. The building features a 54 m square shaft and diagonal face that is shaved back with the aperture cut off to relieve pressure at this location. The opening, shown atop the tower in Fig. 9, measures 51 meters in diameter. The design utilizes the benefits of both through-building openings and those provided by shifting and decreasing the cross section with increasing height, essentially tapering the 460 m tower.

However, care must always be taken in order to engineer modifications that will produce the desired effect, constantly consulting wind tunnel tests to verify the effects of altering the plan shape or employing other forms of aerodynamic modifications. By using modifications which do not increase the projected area or the effective breadth of a building, engineers may achieve significant response reductions (Kwok 1995).

5. Damping sources

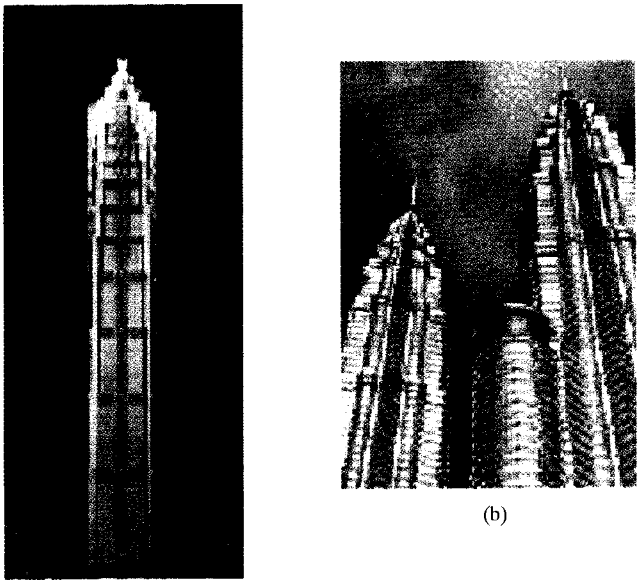
Increasing the effective damping of a structure will also lead to decreased structural motion. Such an increase may be accomplished through any of the four major sources of damping: structural, aerodynamic, soil, and auxiliary. Structural damping is limited to the damping that is inherent in the materials: steel, concrete, or their composite. At times, aerodynamic damping may also contribute in the alongwind direction, depending on the wind velocity, structural shape, and building dynamic characteristics. However, the contribution in the acrosswind direction is negligible and may even become adverse at higher wind speeds,



(b)

(a)

Fig. 7 (a) MHI Yokohama Building (taken from Mitsubishi Heavy Industries, Ltd.); (b) Efficiency of changing sectional shape along vertical axis (taken from Shimada & Hibi 1995)



(b)

(a)

Fig. 8 (a) Sketch of Jin Mao Building (taken from Skidmore, Owings and Merrill, LLP); (b) Photo of upper plan of Petronas Towers (taken from kiat.net)

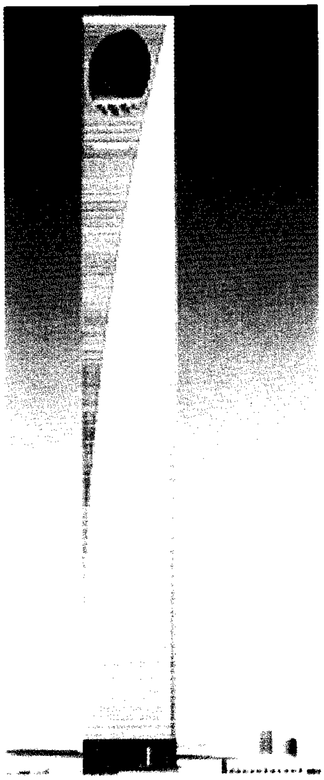


Fig. 9 Shanghai World Financial Center (taken from Mori Building Co., Ltd.)

though the presence of adjacent structures may introduce different effects. Although not marked for high rise buildings, damping contributions may also be obtained from the soil-foundation interaction, i.e., soil damping. However, these three forms of damping make only limited contributions. In addition, the damping in the structure cannot be engineered like the mass and stiffness properties of the structure, nor can it be accurately estimated until the structure is completed, resulting in a certain level of uncertainty (Kareem and Gurley 1996). In cases where the inherent damping is not sufficient, auxiliary damping devices may be introduced, offering a somewhat more predictable, adaptable, and reliable method of imparting additional damping to a system.

6. Auxiliary damping sources

Unlike the mass and stiffness characteristics of the structural system, damping does not relate to a unique physical phenomenon, and it is often difficult to engineer without the addition of external damping systems. Furthermore, the amount of inherent damping cannot be estimated with certainty; however, a known level of damping may be introduced through an auxiliary source (Housner *et al.* 1997, Kijewski *et al.* 1998). Such sources come in the form of both active and passive systems, illustrated schematically in Fig. 10. These devices may be further subcategorized based on their mechanism of energy dissipation and system

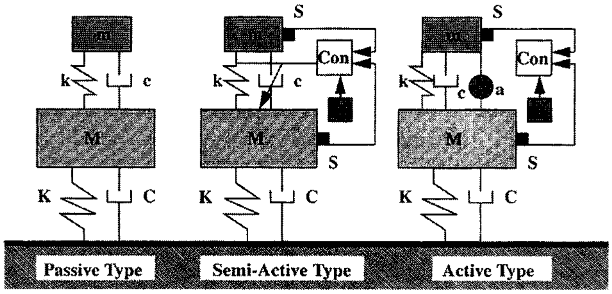


Fig. 10 Schematic of various auxiliary damping devices utilizing inertial effects (Con: controller, a: actuator, Ex: excitation, S: sensor)

Table 2 Auxiliary damping devices and number of installations in Japan, including buildings planned to be constructed after 1997

Building Height	Passive									Active				Total
	SD	SJD	LD	FD	VED	VD	OD	TLD	TMD	HMD	AMD	AVS	AGS	
H<45 m	4	2	1	0	1	2	2	5	1	3	2	0	0	23
H≥45 m	20	1	2	3	2	5	4	7	10	15	3	1	1	74
Total	24	3	3	3	3	7	6	12	11	18	5	1	1	97

See abbreviations in Appendix Table 1.

requirements.

As Table 2 illustrates, such systems have become increasingly popular, especially in Japan, for the mitigation of motions that result from wind, and in some cases, for wind and seismic considerations, as demonstrated by Table 3. Accordingly, each of these auxiliary damping systems will be discussed herein, with specific attention to notable applications of these devices to actual structures in Australia, China, Canada, Japan, and the United States, for the purpose of controlling wind induced vibrations. While this discussion of applications is not exhaustive, Appendix-Table 2 contains information on other applications utilizing inertial systems.

6.1. Passive dampers (with indirect energy dissipation)

Auxiliary damping is commonly supplied through the incorporation of some secondary system capable of passive energy dissipation. One example is the addition of a secondary mass attached to the structure by a spring and damping element which counteracts the building motion. Such passive systems (Soong and Dargush 1997) were embraced for their simplicity and ability to reduce the structural response. Of the passive devices that impart indirect damping through modification of the system characteristics, the damped secondary inertial system is most popular. These systems, which will be discussed below, impart indirect damping to the structure by modifying its frequency response (Kareem 1983).

6.1.1. Tuned Mass Dampers (TMDs)

A TMD typically consists of an inertial mass attached to the building at the location where

Table 3 Target excitations for response control in Japan (47 Buildings)

Target Excitation		Wind Force Only	Wind and Seismic Forces	Target Excitation		Wind Force Only	Wind and Seismic Forces
Passive	TLD	9	1	Active	HMD	13	6
	TMD	7	5		AMD	2	4
	Total	16	6		Total	15	10

the response is maximum, generally near the top, through a spring and damping mechanism, typically viscous and viscoelastic dampers, shown previously in Fig. 10. TMDs transmit inertial force to the building's frame to reduce its motion, with their effectiveness determined by their dynamic characteristics, stroke and the amount of added mass they employ. The amount of additional damping introduced by the system is also dependent on the ratio of the damper mass to the effective mass of the building in the mode of interest, resulting in TMDs which weigh 0.25%-1.0% of the building's weight in the fundamental mode (typically around one third). Often, spacing restrictions will not permit traditional TMD configurations, requiring the installation of alternative configurations, including multi-stage pendulums, inverted pendulums, and systems with mechanically-guided slide tables, hydrostatic bearings, and laminated rubber bearings. Coil springs or variable stiffness pneumatic springs typically provide the stiffness for the tuning of TMDs. Although TMDs are often effective, better performance has been noted through the use of multiple-damper configurations (MDCs), which consist of several dampers placed in parallel with natural frequencies distributed around the optimal frequency (Kareem and Kline 1995). For the same total mass, a multiple mass damper can significantly increase the equivalent damping introduced to the system. Presently, there are several types of TMDs in use in Japan, typically employing oil dampers, though a few viscous and viscoelastic dampers are being used, (Tamura 1997) as shown by Table 4. In addition, several other structures in the United States, Australia and Canada employ TMDs.

6.1.2 Applications of Tuned Mass Dampers

Tuned Mass Dampers and their variations comprise the greatest percentage of secondary damping systems currently in use, as Appendix-Table 2 reflects. Not only have they been applied to buildings, but also to chimneys, bridges and other industrial facilities in Australia, Belgium, Canada, Germany, Japan, Pakistan, Saudi Arabia, and the United Kingdom. Recent applications of TMDs include the 67.5 m Washington National Airport Control Tower (Banavalkar and Isyumov 1998), shown in Fig. 11a. The incorporation of this device added an estimated 3% in damping to the 0.5% inherently present. Another example is the legs of the Petronas Towers 54.8 m Skybridge (Breukelman *et al.* 1998). The lightweight cylindrical legs of the Skybridge were highly sensitive to vortex excitations. The application of additional damping through tuned mass dampers resulted in a total damping of 0.5%, which was sufficient to prevent vortex shedding and the ensuing fatigue damage.

One of the earliest TMD installations dates back to June 1977 in the 244 m Hancock Tower (ENR 1977) in Boston, shown in Fig. 11b. Two TMDs were installed at opposite ends of the 58th floor in order to counteract the torsional motion. Each unit measured about 5.2×

Table 4 Mass support mechanisms and dampers for TMDs in Japan (11 buildings) (Kitamura *et al.* 1995)

Mass Supporting Mechanism			Damper Attached to TMD		
Pendulum Including Multiple Type	5	46%	Oil Dampers	8	73%
Laminated Rubber Bearings	4	36%	Visco-Elastic Dampers	2	18%
Roller Bearings and Coil Springs	2	18%	Viscous Dampers	1	9%

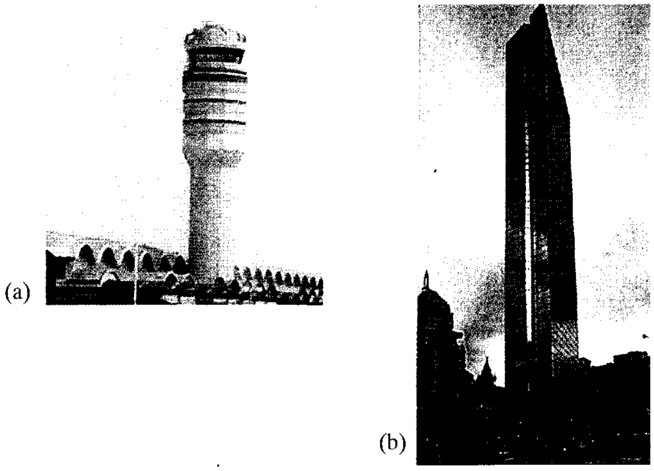


Fig. 11 (a) Washington National Airport Control Tower (taken from Civil Engineering 1996); (b) Boston's Hancock Tower (taken from Boston Society of Architects)

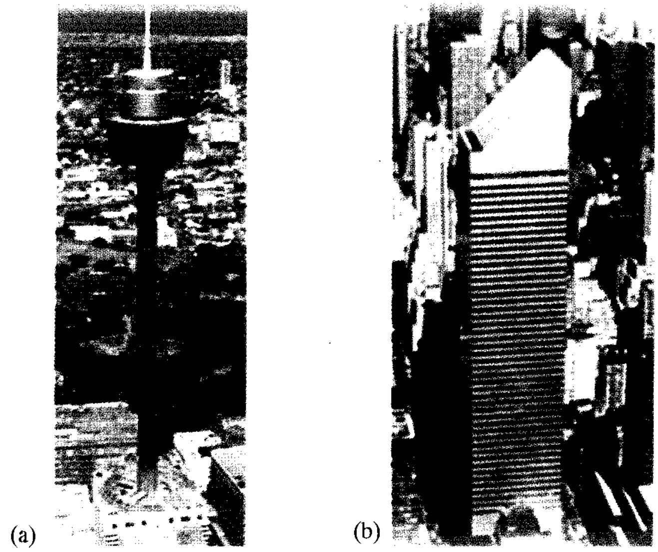


Fig. 12 (a) Sydney Tower (taken from Bartel Ltd.); (b) Citicorp Center (taken from Flour City Architectural Metals Ltd.)

5.2×1 m and was essentially a steel box filled with lead, weighing 300 tons, attached to the frame of the building by shock absorbers. The system is activated at 3 milli-g's of motion at

Table 5 Other configurations of TMDs currently in use

Host Structure	Location	Description	Installation Date	Results
CN Tower	Toronto	20 ton doughnut-shaped lead pendulums	1975	
Sydney Tower (Fig. 12a) (Kwok and Samali 1995)	Sydney	doughnut-shaped water tanks and energy dissipating shock absorbers	1981	Response Reduced 40-50%
Chiba Port Tower (Kitamura *et al.* 1995)	Chiba	slide-platform type	1986	Response Reduced 40%-50%
Fukuoka Tower (Kihara 1989)	Fukuoka	slide-platform type	1989	
Higashimyama Sky Tower (Konno and Yoshida 1989)	Nagoya	inverted pendulum type w/coil springs	1989	Response Reduced 30-50%
Huis Ten Bosch Domtoren (Kawamura *et al.* 1993)	Nagasaki	TMD w/VE material made of asphalt between steel plates of laminated rubber bearings	1992	Response Reduced to 1/2-1/3
Chifley Tower (Kwok and Samali 1995)	Sydney	single pendulum w/hydraulic cylinders	1994	ζ= +2-4%
Washington National Airport Tower (Banavalkar and Isyumov 1998)	Washington, D.C.	TMD	1997	ζ=+3%
Sendai AERU	Sendai	TMD w/Laminated Rubber Bearings+Coil Spring	1998	Response Reduced 1/2

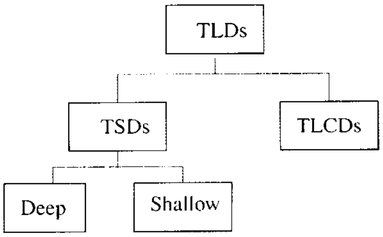


Fig. 13 Schematic of the TLD Family

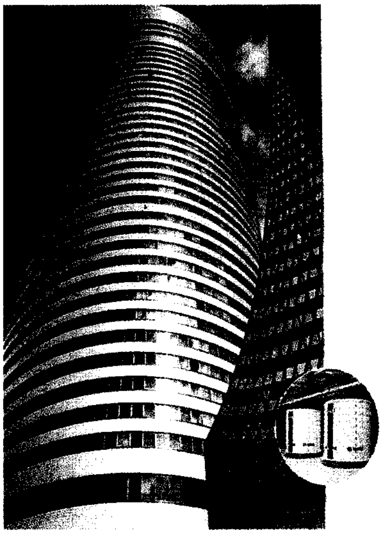


Fig. 14 Shin Yokohama Prince Hotel and TSD units installed (taken from Shimizu Corp.)

which time the steel plates, upon which the devices rest, are lubricated with oil so that the weights are free to slide (Campbell 1995).

Another early application of TMDs has been in use in New York's 278 m Citicorp Building (Petersen 1980), shown in Fig. 12b, since 1978. The system measures 9.14×9.14×3.05 m and consists of a 410 ton concrete block with two spring damping mechanisms, one for the north-south motion and one for the east-west motion, installed in the 63rd floor. The system was included in the overall design due to the building aspect ratio and dynamic features. Activation of the system at the critical acceleration threshold of 3 milli-g's is accomplished by hydraulically raising the concrete mass, allowing full motion of the block as it is regulated by two computer-controlled hydraulic actuators which push and pull the block in the east-west and north-south directions simultaneously to insure that the system behaves as an "ideal" passive bi-axial TMD (Wiesner 1979). The block, which rests on a series of twelve hydraulic pressure-balanced bearings, has its motion inhibited by two pneumatic springs which are tuned to the natural period of the building. The system reduces the wind-induced response of the Citicorp Building 40% in both the north-south and east-west directions simultaneously (Wiesner 1979).

Often, tuned mass dampers can be engineered without introducing additional mass to the structure. Three structures in Japan utilize such an approach: the Rokko-Island P & G Building in Kobe, the Crystal Tower (Nagase and Hisatoku 1992) in Osaka, and the Sea Hawk Hotel and Resort in Fukuoka (Nagase 1998). All three structures have successfully utilized existing ice thermal or water tanks for the suppression of wind-induced vibrations. Other notable applications of TMDs are provided in Table 5, and a more complete catalogue is given in Appendix-Table 2.

6.1.3. Tuned Liquid Dampers (TLDs)

Tuned Liquid Dampers, encompassing both Tuned Sloshing Dampers (TSDs) and Tuned Liquid Column Dampers (TLCDs), shown schematically in Fig. 13, have become a popular inertial damping device (Fujino *et al.* 1992, Kareem and Sun 1987, Kareem 1990, Kareem 1993, Kareem and Tognarelli 1994, Modi and Welt 1987, Sakai *et al.* 1989, Tamura *et al.* 1988) since their first applications to ground structures in the 1980's. In particular, the TSDs are extremely practical and are currently being proposed for existing water tanks on buildings. By simply configuring internal partitions into multiple dampers, the tanks may be utilized as auxiliary damping devices without adversely affecting the functional use of the water supply tanks. Considering only a small additional mass, if any, is added to the building, these systems and their counterpart TMDs can reduce acceleration responses to 1/2 to 1/3 of the original response, depending on the amount of liquid mass (Tamura *et al.* 1995). This, coupled with their low maintenance requirements, has been responsible for their wide use.

Currently, both deep and shallow water configurations of TSDs, which exploit the amplitude of fluid motion and wave-breaking patterns to provide additional damping, are being used worldwide. The shallow water configurations dissipate energy through viscous action and wave breaking. However, Yalla and Kareem (1999) have recently noted and modeled the high amplitude liquid impacts or "slamming" phenomena. The addition of PVC floater beads may also add to the dissipation of sloshing energy. Deep water TSDs, on the other hand, require baffles or screens to increase the energy dissipation of the sloshing fluid. However, the entire water mass often does not participate in providing the secondary mass in these configurations (Kareem and Sun 1987).

While the natural frequency of a TLD may be simply adjusted by the depth of water and the dimension of the container, there are practical limitations on the water depth and thus the frequency which may be obtained by a given container design. One possible solution is a device which adjusts the sloshing frequency of the damper using a spring mechanism so that the same device can be effective should the building experience a change in its dynamic characteristics (Shimizu and Teramura 1994). With this device, the TLD can be made into one large tank instead of multiple containers.

Innovations in TLCD technologies include their extension to active control applications. A current study (Honda *et al.* 1992) on a nine-story steel building involves a pressurized U-shaped oscillator installed at the structure's top floor. The natural frequency of the device may be continuously adjusted through the modulation of the pressure in the air chamber. In addition, other configurations such as liquid column vibration absorbers or LCVAs (Hitchcock and Kwok 1993), adaptive TLCDs (Kareem 1994), inertia pump dampers, and amplitude-dependent orifice and multiple-orifice systems have been explored as effective sources of secondary damping for structures.

6.1.4. Applications of Tuned Liquid Dampers

While the use of TLDs has not been particularly popular in the United States, they have been incorporated in structures elsewhere. In Australia, the 105 m Hobart Tower in Tasmainia was equipped with 80 TSD units after the tower was cloaked in a protective cylindrical shell. The shell, while shielding the transmission antenna from the harsh conditions, unfortunately increased the wind-induced response, necessitating the installation of the TSD units. In addition, Japanese installations of TLDs include six shallow TSDs, one deep TSD, and five TLCDs as of 1997. The TSDs primarily utilize circular containers for shallow configurations and rectangular ones for deep water TSDs, while the TLCDs rely on the traditional U-shaped vessel. Such applications work best for buildings with small vibrations and have been observed to reduce the structural response to 1/2 to 1/3 the original response in strong winds.

One Japanese TSD application located in the top floor of the 158 m Gold Tower in Kagawa features sixteen units. The installation of ten tons of TSDs was found to reduce the response to 1/2 to 1/3 of the original response. The cube-shaped tank is filled with water and equipped with steel wire nets to dissipate the motion of the liquid. By adjusting these damping nets, the length of the tank, and the depth of water, the device may be appropriately tuned. There are many advantages to such applications, including: (1) there is no mechanical friction in the system, so it is effective for even the slightest vibrations, (2) failure of the system is virtually impossible, (3) it is effective against the strong motion of earthquakes and winds, (4) the period is easy to adjust, and (5) the system is inexpensive and easy to maintain (Noji *et al.* 1991). However, there are drawbacks as well in that all the water mass does not participate in counteracting the structural motion. This results in added weight to the structure without the benefit of commensurate response control. The low density of water as a secondary mass adds to the large volume of the damper and the space required to house it.

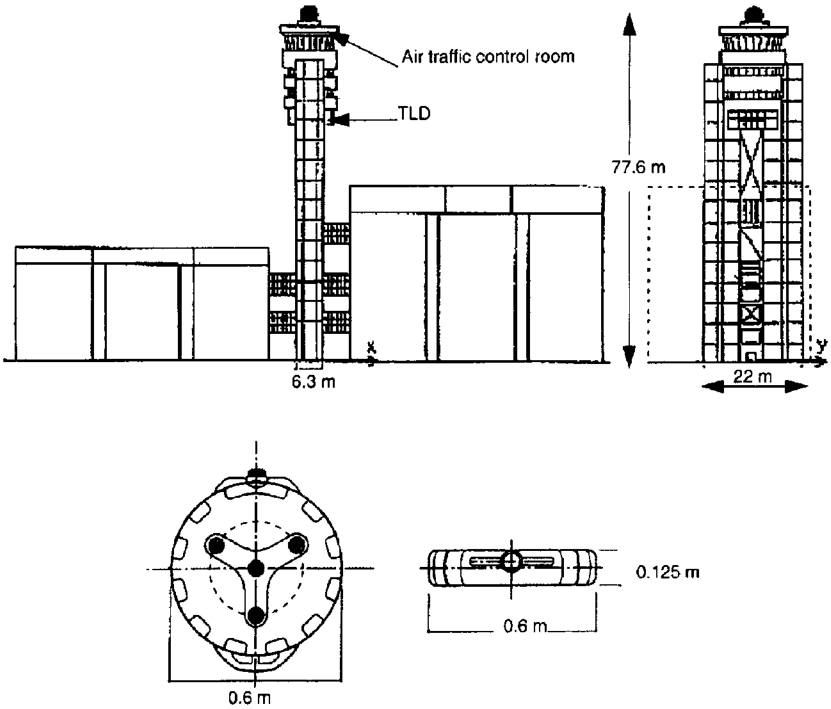


Fig. 15 Tokyo International Airport Tower (TIAT) at Haneda and views of TLD units installed (taken from Tamura *et al.* 1996)

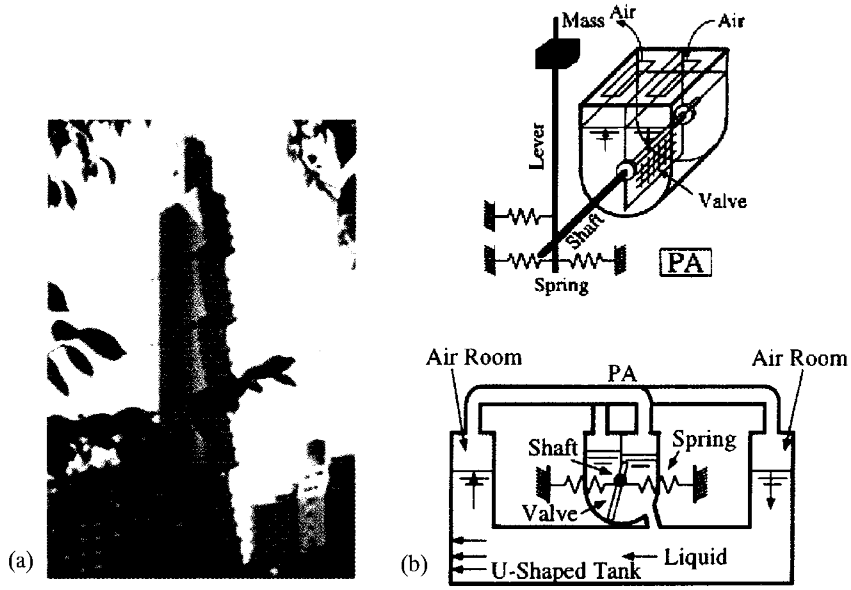


Fig. 16 Cosima Hotel and sectional view of the LCD-PA concept with detail of period adjusting mechanism (taken from Shimizu & Teramura 1994)

An alternative TSD configuration consisting of multi-layer stacks of nine circular (2 m dia.) fiber-reinforced plastic containers, each 22 cm high, was installed in 1991 in the 149 m Shin Yokohama Prince Hotel (SYP) in Yokohama, Japan (Fig. 14). Each layer of the TSD was equipped with twelve protrusions installed in a symmetric radial pattern to preclude the swirling motion of the liquid and to get adequate additional damping. From performance observations of this installation, the hotel has been shown to successfully meet minimum perception levels prescribed in ISO 6897 Standards (max rms acceleration of 0.6 cm/s^2) with a maximum rms acceleration of 0.5 cm/s^2 (Wakahara *et al.* 1994), and rms response reductions of 30~50% in winds exceeding 20 m/s.

Similarly, another multi-layer configuration of 25 units was installed in the 42 m Nagasaki Airport Tower in 1987. Twelve cylindrical, multi-layered vessels constructed of vinyl chloride measuring 50 cm high and 38 cm in diameter were installed on the air-traffic control room floor with the remaining thirteen distributed on each stair landing. Each vessel is divided into seven, 7 cm high layers, each containing 4.8 cm of water and weighing 38 kg. Therefore, 950 kg of TSD units were installed in the tower. Run down tests conducted to calculate the frequency and damping ratio of the tower revealed that there was more displacement due to the acrosswind component than the alongwind. These tests also uncovered the presence of beat phenomena which was eliminated through the use of floating particles that helped to dampen the liquid motion in the containers. An examination of the tower response has once again shown that the performance of the TSD appears to improve at even higher velocities with the response in reduced 35% in winds of 20 m/s (Tamura *et al.* 1995).

Another airport tower has also been equipped with a TSD system. Consisting of approximately 1400 vessels containing water, floating particles, and preservatives, the device was installed in the 77.6 m Tokyo International Airport Tower at Haneda in 1993, as shown in Fig. 15. The 12.5 cm-tall shallow circular cylindrical vessels measure 60 cm in diameter and were filled with water to a height of 5.3 cm. The vessels feature injection taps and handles to serve as projections, with four conical dents on the upside and base. These projections and dents provide additional stiffness for stacking the polyethylene vessels. During an actual storm, data revealed that the 22.7×10^3 kg TSD application raised the damping ratio to 1%, peaking at 7.6% as the rms acceleration grew (Tamura *et al.* 1995).

In addition to the various installations of TSDs, there are also some applications of TLCD technologies, including those with period adjustment mechanisms. By equipping a Tuned Liquid Column Damper with Period Adjustment Equipment (LCD-PA), the behavior of the liquid motion in the damper column may be regulated. Such a system has been installed in the top floor of the 26-story Hotel Cosima, now called Hotel Sofitel (Fig. 16) in Tokyo.

The LCD-PA consists of a rectangular, U-shaped tank, a pair of air rooms, and period adjustable equipment, as shown in Fig. 16. When the tank is moved in the horizontal direction, fluid travels in both the vertical and horizontal directions. Thus, in one side, the air is compressed, while in the other chamber, the air pressure is reduced. The sinusoidal pressure fluctuations induce fluid movement in the subsidiary U-shaped tank, resulting in the movement of the valve, shaft and springs. The device installed in the hotel is a rectangular-based, bi-directional LCD with four PA's and an effective total weight of 51 tons with an effective liquid weight of 36 tons. The tank consists of a portion where liquid is free to move in any horizontal direction, four vertical reservoirs (VR) at each corner above the horizontal partition,

Table 6 Other Japanese liquid damper applications

TSD applications	Atsugi TYG Building, Narita Airport Tower, Yokohama Marine Tower (Wakahara *et al.* 1994)
TLCD applications	Hotel Cosima, Hyatt Hotel in Osaka, Ichida Building in Osaka (Shimizu and Teramura 1994)

See Appendix-Table 2 for more details and applications.

and four air chambers separated by partitions. The PA is arranged between the two vertical reservoirs (Shimizu and Teramura 1994). The system has been observed to reduce the maximum acceleration to 50-70% of its original value and the rms acceleration to 50% as well.

The Shanghai World Financial Center, shown earlier in Fig. 9, is also to be equipped with eight TSD units on its 91st floor upon its completion sometime in 2001 (Wakahara *et al.* 1998). Each tank will be 7.5 m in diameter, separated into six layers. The installation of the 800-ton TSD system (1% mass ratio) is anticipated to successfully reduce story drift and peak and rms accelerations to acceptable limits, when compared to ISO standards (Hori and Nakashima 1998). Other notable installations of TSDs and TLCDs in Japan are listed in Table 6.

6.1.5. Impact dampers

Impact Dampers (Masri and Caughey 1966, Reed 1967) serve as a practical and unique form of inertial system. The devices are typically in the form of small rigid masses suspended from the top of a container mounted at its side to the structure. In Fig. 17, an equivalent lumped mass model is shown. The container is designed to a specified dimension so that an optimal spacing is left between the suspended mass and the container, allowing collisions to occur between the two as the structure vibrates. While gap distance serves as a major parameter in the design of such systems, the suspension length and mass size are also of extreme importance, as they dictate the frequency of the system. This type of damper is particularly effective for masts and tower-like structures with oscillations in one plane. Optimal tuning of the device is achieved when two impacts take place in each cycle of motion. This type of device is being used widely, particularly for rooftop masts (Koss and Melbourne 1995).

6.1.6. Applications of impact dampers

While impact dampers have been used extensively to control the vibrations of turbine blades, printed circuit boards, and machine tools, their application for the vibration of large structures is still relatively limited (Ying and Semercigil 1991). Early applications of impact dampers in the form of chains encased in plastic were utilized by the Navy in their communications antennas. These pioneering applications proved that displacements could be significantly reduced through the impact of the coated chains (Reed 1967). This form of impact damper, termed the Hanging Chain Damper (HCD), consists of rubber coated chains housed in cylinders. The device combines the benefits of the inelastic impacts with the added internal friction of the chain links rubbing against each other. These technologies have been repeatedly used in towers, masts, and light poles in Australia and Japan to control vibrations

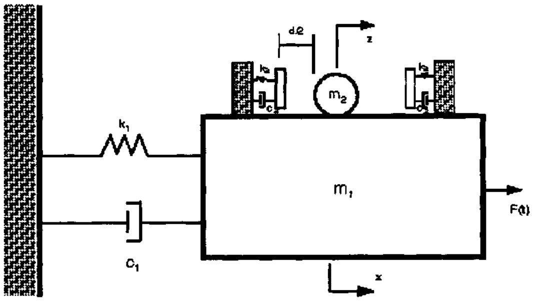


Fig. 17 Schematic of an impact damper

Table 7 Applications of impact dampers in Australia and Japan

Structure	Height [m]	Device
Australian Applications		
Tower	30	3 HCDs
Mast	24.7	4 HCDs to control 1st mode, 1 HCD for 2nd mode
Mast	25	1 HCD with 6 m chain
Mast	17	4 HCDs for 1st mode, mast itself used as cylinder for HCD for 2nd mode
Japanese Applications		
Light Poles of Oonaruto Bridge (1986) and in Yokohama (1988); Bridge Pylons in Fuchuo-ku (1992)		

due to wind, as summarized in Table 7.

6.2. Passive dampers (with direct energy dissipation)

Passive systems may also raise the level of damping in a structure through a direct energy dissipation mechanism, such as the flow of a highly viscous fluid through an orifice or by the shearing action of a polymeric/rubber-like (viscoelastic) material. Other classes of passive systems with direct energy dissipation include Viscous Damping Devices (VDDs), Friction Systems, and Metallic Dissipators. The use of such mechanisms in structures has grown in popularity both in the United States and Japan, since they require very little space and can be easily retrofitted into existing frames. Their efficiency under large amplitude events such as earthquakes has made them an especially popular choice in seismic areas, as discussed in the following sections.

6.2.1. Viscoelastic dampers (VEDs)

Viscoelastic dampers are one of the earliest types of passive dampers to be successfully applied to structures (Mahmoodi *et al.* 1987). VEDs commonly use polymeric or rubberlike materials which are deformed in shear to provide both energy dissipation and a restoring force. These devices are particularly effective in the high frequency range and at low vibration levels against strong winds and moderate earthquakes. This form of damper, which usually consists of steel plates which sandwich the viscoelastic (VE) material, is readily installed as part of a diagonal brace, where it can dissipate vibrational energy by the shearing action of the VE material. The force generated by this system is dependent on the velocity and is out of phase with the displacement, making these devices particularly efficient in a building's diagonal bracing system, such as rod and piston dampers (Chang *et al.* 1992).

Currently, work is ongoing to explore the performance of such VED systems under various excitation records. Preliminary studies indicate that these devices not only add damping to the system, but also stiffness, which raises the natural frequency of the test structure. In addition, these devices perform satisfactorily for both steel and concrete structures (3M 1995). However, since the VE damper's properties (storage and loss moduli analogous to spring and dashpot constants, respectively) are dependent on vibrational frequency and environmental temperature, the system may manifest varied performance based on the particular situation. However, research indicates that the damper properties remain somewhat constant with strains below 20% for a given temperature and frequency (Chang *et al.* 1992, Oh *et al.* 1992).

6.2.2. Applications of viscoelastic dampers

To date, VEDs have been installed in four buildings in the United States for the minimization of wind-induced vibrations, with the earliest installation being the World Trade Center Towers in New York. These applications are summarized in Table 8 (3M 1995).

In Japan, VEDs have been used to reduce the wind-induced response of several buildings: the Seavans South Tower in Tokyo (1991), the Old Wooden Temple, Konohanaku Symbol Tower (1999), the ENIX Headquarter Building, the Sogo Gymnasium in Chiba (1993), the Goushoku Hyogo Port Distribution Center (1998) with viscoelastic joint dampers which reduce the seismic response by one half, and the Torishima Riverside Hill Symbol Tower, whose 1999 installation features eight VEDs per story for the 1st to 19th floors and reduces to four VEDs per story for the 20th to 38th stories. In addition, the Chientan Railroad Station in Taipei, Taiwan has also been equipped with eight viscoelastic units to control the wind-

Table 8 US Applications of VEDs to reduce excitation due to wind

Building(Location)	Location and Installation Date	Number of Units	Location in Structure	Performance
World Trade Center Towers (Mahmoodi *et al.* 1987)	New York 1969	10,000/ tower	installed in lower chord of trusses that support the floors	ξ=2.5-3% in Hurricane Gloria
Columbia SeaFirst Building (Mahmoodi and Keel 1986)	Seattle 1982	260	parallel to main diagonal braces of building	ξ=3.2% at design wind and upto 6.4% in storms
Two Union Square Building	Seattle 1988	16	parallel to four columns on one floor of bldg	
Torishima Riverside Hill Symbol Tower	Japan 1999	224	8 VED/floor on first 19 floors 4 VED/floor on 20-38 floors	Wind acceleration response: 80%

induced vibrations of its unique suspended dragon boat roof (Cermak *et al.* 1998).

Although the use of VEDs to control excitations due to wind has been common for over twenty years, their use in seismic applications has just begun to flourish (Samali and Kwok 1995). Installing these devices in the form of rubber-asphalt attached to the walls in one direction of every floor of a 24-story building was found to improve the structural responses under earthquake conditions by 30% (Yokota *et al.* 1992). There have been numerous other seismic applications in the United States, particularly in the area of retrofitting, including the Santa Clara Civic Center Office Building.

6.2.3. Friction systems

Facilitating direct damping through friction systems enables plastic behavior by providing non-linearity while allowing the structure itself to remain elastic. The systems, which are carefully controlled by a sliding surface, feature a very large initial stiffness and the possibility of nearly perfect rectangular hysteretic behavior (Aiken and Clark 1994). There are two main types of friction dampers used in steel-framed buildings: rigid frame friction dampers, which provide real plastic hinges that can easily be replaced after an earthquake, and braced frame friction dampers, which utilize diagonal bracing that slips at a predetermined stress value.

Since the aforementioned systems have a predictable slip load and uniform hysteretic behavior, they are excellent for damping seismic vibrations and may also be applied to reduce wind-induced vibrations (Taylor and Constantinou 1996).

6.2.4. Applications of friction systems

Friction systems are currently in use in several buildings in Canada that feature friction braces and some in Japan which use piston-type friction dampers (Aiken and Clark 1994). There have been several other applications of friction systems, as exemplified by Table 9.

6.2.5. Viscous Damping Devices (VDDs)

Viscous Damping Devices (Oil Dampers: Viscous Fluid Dampers or Oil Pressure Dampers) have become quite common in the construction of new structures and for retrofitting in

Table 9 Some applications of friction systems

Building	Structure/ Use	Year	Height (m)	Fundamental Natural Frequency(Hz)	Equipment/Mechanism
Sonic City Office Tower, Ohmiya	Steel/ Office	1988	140	w/o Dampers: 0.32 (x), 0.33 (y) w/Damper:0.35 (x), 0.36 (y)	x-dir: 4 dampers/floor y-dir: 4 dampers/foor friction force/damper: 10 t
Asahi Beer Tower, Tokyo	Steel/ Office	1989	94.9	w/o Dampers: 0.32 (x & y) w/Damper: 0.35 (x & y)	x-dir: 2/floor (1st-20th floors) y-dir: 2/floor (1st-20th floors)

seismic zones, following their development and early applications in military operations. This form of damper dissipates energy by applying a resisting force over a finite displacement through the action of a piston forced through a fluid-filled chamber for a completely viscous, linear behavior, or in damping walls which use a full-story steel plate traveling in a wall filled with viscous material to provide added damping. Through careful design, the devices are capable of providing viscous damping to the fundamental mode and additional damping and stiffness to higher modes, and may, in effect, completely suppress their contributions, raising the structural damping to 20~50% of critical. By incorporating fluid viscous dampers to control wind induced vibrations, structures may be built with reduced lateral stiffness, as the fluid dampers alone reduce the wind deflection by a factor of 2 to 3, which greatly improves occupant comfort without creating localized stiff sections (Taylor and Constantinou 1996).

Though operating on the same premise as many of the other forms of energy dampers, the fluid damper holds several advantages. For example, these are attractive for incorporation in diagonal bracing systems (Aiken and Clark 1994). By requiring no external power source and little maintenance, they have become very attractive options for civilian applications, having proven their durability and effectiveness in over 100 years of large scale military use (Taylor and Constantinou 1996).

6.2.6. Applications of viscous damping devices

Other passive systems also exist and are rapidly gaining popularity, especially in the design of seismically vulnerable structures. In this area, the application of Viscous Damping Devices (fluid inertial dampers) has been notable. The first use of VDDs for seismic zones was in 1993 in the earthquake-resistant design of the San Bernadino County Medical Center in California. The addition of VDDs to the system helped to keep displacements under 22 inches and lengthened the effective period to 3.0 seconds (Asher *et al.* 1994).

Since that installation, there have been numerous other seismic applications, including the Pacific Bell Emergency Communications Building (Sacramento, CA), the Woodland Hotel (Woodland, CA), the CSUS Science II Building (Sacramento, CA) and recently for the seismic retrofit of bridges. In addition, they were installed in 1984 in the North American Air Defense Command in Wyoming for the possible loads caused by a nuclear attack and have been proposed for use in residential structures (Taylor and Constantinou 1996).

While such devices have witnessed widespread application in seismic zones, they have also been installed in several structures for the explicit purpose of controlling wind-induced vibrations, as Table 10 reflects (Taylor and Constantinou 1996).

In addition to these applications, viscous dampers were also installed in the Sato Building in Tokyo (1992), the Shimura Dormitory in Tokyo (1993) and the Structural Planning Headquarters (1999). In addition, a viscous damping wall was installed in the TV Shizuoka Media City Building, an office building in Shizuoka, Japan, in 1993. For this application, a total of 170 walls were implemented with the device in the x and y directions on each of the building's fourteen floors. Other viscous damping wall installations in Japan include the Daikanyama Apartment House, the Postal Service Administration (Kanto Area) Government Office (Kihara *et al.* 1998) and the Academic Information Center.

Table 10 Applications of viscous damping device to reduce wind-induced excitation

Structure	Location	Installation Date	Type and Number of Dampers	Additional Information
Rich Stadium	Buffalo, NY	1993	12 Fluid Dampers 50 kN, mm stroke	Dampers connect light poles to stadium wall to eliminate base plate anchor bolt fatigue
28 State Street	Boston, MA	1996	40 Fluid Dampers 670 kN, ±25 mm stroke	Used in diagonal bracing for serviceability issues
Petronas Twin Towers	Kuala Lumpur City	1995	12 Fluid Dampers 10 kN, ±50 mm stroke	Part of mass damping system in skybridge legs
Building A		1995	80 Oil Dampers, ±60 mm stroke	Increased Damping by 2.1% of critical

6.2.7. Metallic dissipators

Another passive device, the metallic dissipator, uses the plastic deformation of mild steel, lead, or special alloys to achieve predictable hysteretic behavior, as was achieved by the ancient architects of the Parthenon for improved resistance to earthquakes. Around 400 BC, the Greek builders recognized the importance of lateral resistance in their famous temples, incorporating socketed dowels which linked the drum-like layers comprising their columns (National Geographic Society 1992). Greek temples such as the Parthenon, whose columns are shown in Fig. 18a, relied on iron dowels embedded in lead for this purpose. The marble disks of the columns could then slide horizontally in an earthquake while maintaining the gravity loads on the structure. During this action, shearing of this lead core, shown in Fig. 18b, and the frictional resistance generated between the two disks of marble, provided an additional mechanism for energy dissipation. Over 2300 years later, in 1993, Japanese engineers followed in the Greek's initiative when they installed twelve steel dampers in the Chiba Ski-Dome, a modern indoor ski stadium.

One type of metallic dissipator, the Added Damping and Stiffness (ADAS) device, utilizes a series of steel plates which undergo distributed flexural yielding when the assemblage is sheared (Aiken and Clark 1994). Most plastic deformations during an event will then be in the ADAS devices, and therefore, damage to the primary building components is limited (Perry and Fierro 1994). Other examples of metallic dissipators include lead extrusion dampers featuring a piston to extrude lead through a constricted orifice within a confined cylinder to give very stable hysteretic behavior over repeated yield cycles. These systems are currently in use in Japan and New Zealand. Other recent developments include shape memory alloys such as the nickel-titanium alloy, Nitinol, which have the ability to undergo a reversible phase transformation under stress, dissipating energy similar to yielding steel without permanent damage (Aiken and Clark 1994).

6.2.8. Applications of metallic dampers

In recent years, there has been a considerable increase in the number of installations of

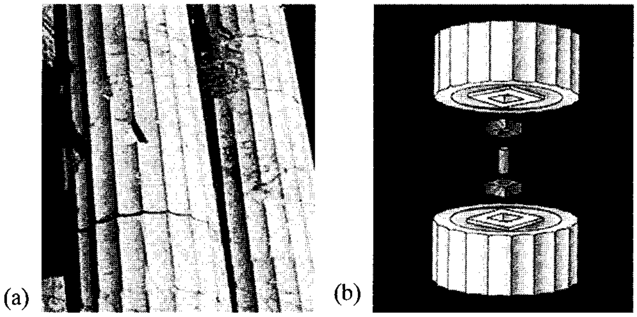


Fig. 18 (a) Photograph of columns in Greek Parthenon; (b) Schematic of lead dowel action in columns (taken from National Geographic 1992)

metallic damping devices in seismic areas. One example is an ADAS installed in the Wells Fargo Bank in San Francisco, along with bracing and additional upgrading to improve its ability to resist earthquakes. The ADAS system consists of 50 ksi steel plates cut in an hourglass shape that bends in double-curvature flexure when subjected to lateral loading (Perry and Fierro 1994). Several other applications of metallic dampers are provided in Table 11.

6.2.9. Application of alternative passive system

Another form of passive damper that is also being developed for use in seismic applications is comprised of an inverted T-shaped lever, which amplifies the damping force and is accompanied by a pair of oil dampers (Kani *et al.* 1992). A similar system with an oil damper and an I-shaped lever (instead of the T-shaped lever) has been implemented on a full-scale level to a twelve story residential structure in 1993 and was found to achieve an effective damping of 10%.

6.3. Active dampers

Originally, passive control was favored over active control because of its simplicity and reliability. The devices remained functional without an external power source and posed no significant risk of generating an unstable situation. However, without the use of control mechanisms, passive devices could not adjust to any variation in the parameters of the system or the loading characteristics. More efficient and swifter control could be obtained by using a system that has ability to respond to such changes which led to the emergence of active control. This involves smaller devices that are capable of controlling the vibration of structural systems. This aim is accomplished using hydraulic or electro-mechanical actuator systems driven by an appropriate control algorithm. Examples of such algorithms include closed loop or feedback, in which the control forces are determined by the feedback response of the structure; open loop or feedforward, in which the control forces are determined by measured external excitations; and closed-open loop or feedforward-feedback, in which the control forces are determined by both the measured response of the structure and the measured external excitation. Active systems (Soong 1990) include active mass drivers, active

Table 11 Applications of metallic dampers in Japan

Building	Structure/Use	Installation Date	Height	Mechanism
Fujita Corp. Main Office (Tokyo)	Steel/Office	1990	19 story	20 Lead Dampers x 2 directions
KI Building (Tokyo)	Steel/RC/Office	1989	5 story bldg. and 9 story bldg.	12 Steel Dampers
Hitachi Main Office (Tokyo)	Steel/Office	1984	72.6 m	Steel Damper
Ohjiseishi Building (Tokyo)	Steel/Office	1991	81.4 m	Steel Damper
Sea Fort Square	Steel and Reinforced Concrete / Hotel, Residence		93.65 m	120 Honeycomb Steel Dampers
ART Hotels Sapporo	Steel/Hotel	1996	90.4 m	x-dir: 952 Steel Dampers y-dir: 1068 Steel Dampers(slits)
Two Apartment Houses	Reinforced Concrete/ Residential		5 stories	Steel Joint Damper Bell Shape
Garden City School Complex	Steel/School		75.5 m	Honeycomb Steel Damper for torsional vibration
New Central Government Office Building No. 2	Steel/Office		99.5 m	Low-Yield Steel (and Viscous Damper)
Taisho Medicine Headquarter	Steel and Reinforced Concrete / Office		38.75 m	Honeycomb Steel Damper
Kobe Fashion Plaza (Kobe)	Steel/Store, Hotel	1997	81.6 m	Steel Dampers on 12th-18th Floors
Nissei Sannomiya Building	Steel/Office	1997	61.7 m	16 Steel Dampers (Double Column)/story
Miyagi Prefectural Office East Building	Steel and Reinforced Concrete / Office	1998	64.5 m	Hypermild Steel Bracing (164 Total)
Keio Department Store	Steel/Department Store	1998 (retrofit)	9 stories	31 Honeycomb Steel Dampers/story
Kobe Distribution Center	Steel/Warehouse	1998	4 stories	40 Lead core beams+K brace
Art Hotels Sapporo	Steel/Hotel	1998	90 m	Total 2020 Slit Steel Dampers

variable stiffness systems (AVS), active tendon control systems, active gyro stabilizers (AGS), active aerodynamic appendages, and active pulse control systems.

Table 12 Mass supporting mechanisms and actuators for AMDs and HMDs in for 19 buildings in Japan (Kitamura *et al.* 1995)

Mass Supporting Mechanism			Actuator		
Pendulums Including Multiple Type	8	42%	AC Servo-Motors and Ball Screws	13	68%
Laminated Rubber Bearings	7	37%			
Linear Bearings	3	16%	Hydraulic Actuators	6	32%
V-Shaped Rail on Rollers	1	5%			

6.3.1. Active Mass Dampers (AMDs)

For inertial systems, such as the Active Mass Damper (AMD) shown earlier in Fig. 10, a control computer analyzes measured response signals and introduces a control force based on the feedback of the motions of the structure. The actuator operates on the secondary mass, in either sliding or pendulum form, to counteract the building motion. Though these systems require smaller damper masses and have efficiency levels superior to those of their passive counterparts, they require higher operation and maintenance costs and reliability concerns. AMDs have been found to reduce actual structural responses in wind by 1/3 to 1/2 of their uncontrolled values. Currently in Japan, multi- and single-pendulum AMDs and active systems utilizing standard, hollow, and linear rubber bearing systems are being used (Tamura 1997, Sakamoto 1993, Sakamoto and Kobori 1996), as illustrated by Table 12. A list of buildings in Japan that utilize AMDs is then shown in Table 13.

6.3.2. Applications of Active Mass Dampers (AMDs)

The world was first introduced to AMDs in 1989 when two units were installed in the 33 meter tall flexible steel Kyobashi Siewa Building shown in Fig. 19a (Koshika *et al.* 1992). The system, (Fig. 19b) installed to protect the building from earthquakes and strong winds, is capable of responding in 1/100 of a second to vibrations. The system utilizes sensors to detect motions and tremors in the basement and on the sixth and eleventh floors. The two AMDs were installed by positioning one large unit (4 ton) in the middle to control large oscillations and tremors for the entire building and one smaller unit (1 ton) to the side to counteract torsion. The two damper masses are suspended by a wire rope and driven by servo hydraulic actuators. Two pumps and an accumulator act as the hydraulic pressure source for the actuator, providing rapid pressurization at a low energy cost. The system, while only about 1.5% of the building's weight, can reduce the response 1/2 to 2/3 of the uncontrolled response.

Table 13 Japanese applications of AMDs in actual buildings

Name	Location	Date	Height (m)
Kyobashi Siewa Building	Tokyo	1989	33
Sendagaya INTES Building	Tokyo	1991	44
Hanku Chayamachi Building (Applause Tower)	Osaka	1992	161
Riverside Sumida Building	Tokyo	1994	134
Herbis Osaka	Osaka	1997	189

A time history of the acceleration of the building's top floor, shown in Fig. 19c, illustrates the reduction of the response under the action of wind, limiting the accelerations below perception thresholds.

Several other flexible buildings in Japan have employed AMDs, as shown in detail by Table 14. Among these applications, the 58 m Sendagaya INTES Building in Tokyo is especially notable. The building was fitted with two AMD units to control the torsional and translational motions. The AMD units were designed to move in only one direction, since only north-south winds were of interest. In this case, the designers avoided the addition of extra dead weight by using the ice thermal storage tank from the building's air conditioning system as the mass for the AMD (two @ 36 tons), under the action of a hydraulic actuator with ±5 cm stroke. The masses are supported by multi-stage rubber bearings which reduce the control energy consumed in the AMD and make smooth movements. After the system's installation, full-scale data that reflected its performance in strong winds was recorded. Studies have shown the added damping to be approximately 2~4% of critical (Yamamoto *et al.* 1998). During strong winds up to 30.6 m/s, the response of the primary mode over a 30 second interval was reduced by 18% in translation and 28% in torsion. In addition, data on the performance of the system under several earthquakes confirms a response reduction of 57% (Higashino and Aizawa 1993).

Another instance in which the AMD mass was provided by elements already existing in the structure is the Hanku Chayamachi Building, also known as the Applause Tower (Higashino and Aizawa 1993) in Osaka, shown in Fig. 20. The 480-ton heliport, resting on multi-stage rubber bearings at the top of the building was chosen as the AMD mass, thus saving money while not adding any additional weight to the structure. A digital controller, servo mechanism and hydraulic design were implemented along with two 5 ton thrust actuators for both the x and y directions. Free vibration tests have revealed the success of this endeavor, which increased the damping ratio from 1.4% to 10.6%.

The application of active mass systems has not been limited exclusively to Japan. An active mass damper system was designed for incorporation in the 340 m Nanjing TV Tower in China (Reinhorn *et al.* 1998, Kareem *et al.* 1998). Due to space limitations, passive systems, which were initially considered, could not be incorporated. The resulting active system consists of a 590 kN ring-shaped mass, approximately 1% of the tower mass, which slides on arc-shaped teflon bearings. The ring mass has an outer radius of 4.75 m with inner radius of 3.9 m and is controlled by three servo-hydraulic actuators with a stroke of +1.5 m.

6.3.3. Active Variable Stiffness (AVS) system

The AVS system is a new form of active control device that actually changes the stiffness of a structure (Sakamoto and Kobori 1996). The active variable stiffness system is an anti-resonant seismic control system designed to control the vibrations of a structure, even in strong earthquakes. Its installation requires placing large inverted V-shaped braces on each story at both ends of the structure to inhibit transverse motion. Each installation is then attached to the variable stiffness device, which is activated by opening the internal valve. When this valve is closed, the system is locked in place. By analyzing the seismic ground motions, the controller optimally alters the frequency of the structure by selecting the

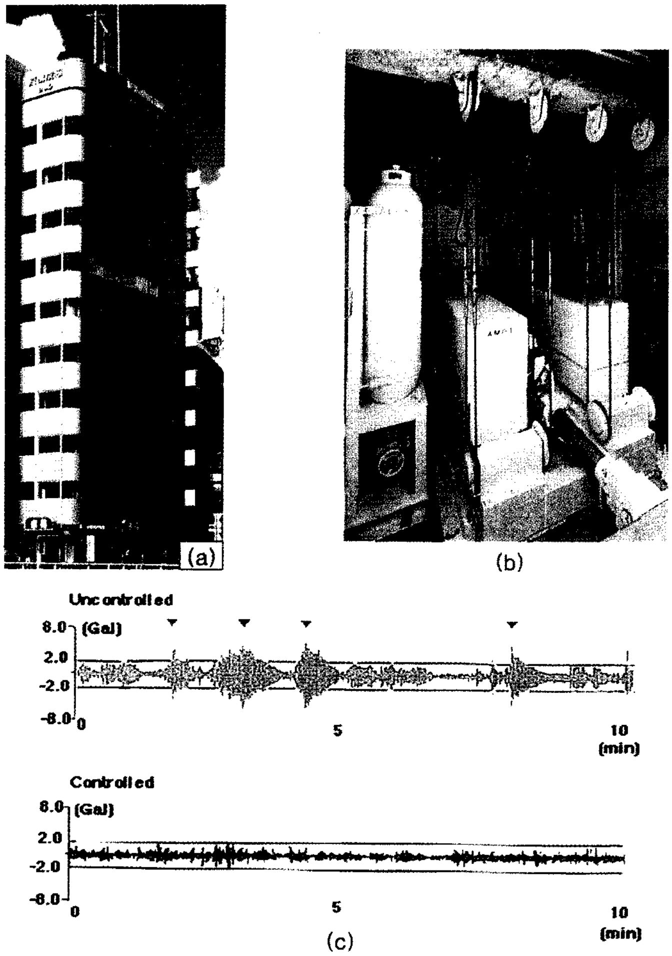


Fig. 19 (a) Kyobashi Siewa Building and (b) its AMD unit: (c) performance of structure under wind (taken from Kajima Corporation)

appropriate stiffness for the building from those available and locking or unlocking different braces to achieve it. Thus, resonant behavior can be eliminated through the successful adjustment of structural stiffness (Sakamoto 1993).

6.3.4. Application of active variable stiffness system

In 1990, the AVS prototype was installed to control the shaking table test facility building at Kajima Technical Research Institute, in Tokyo, Japan. Sensors at the base of the structure analyze the seismic ground motions of the first floor with an earthquake motion analyzer. This information is forwarded to the AVS controller, which engages the system if the ground floor acceleration exceeds 10 cm/s^2 and alters the rigidity of the structure by selecting the optimum rigidity to attain the lowest level of response. The inverted V-shaped braces installed on both short sides of the three-story (12 m) building, with the peak of the "V" attached to the beam, are

Table 14 Other applications of AMDs in Japan

Building	Device	Damper Weight	Performance	Additional Information
Riverside Sumida Building (Suzuki *et al.* 1994)	2 masses, servo motors and ball screws, uni-directional, linear bearings	2@15 t	damping ratio increased from 0.85% to 8.0% and reduced response 20-30% in earthquake	±100 cm stroke; capable of controlling multiple modes: 1st-3rd transverse modes and 1st torsional mode
Herbis Osaka (Takenaka 1997)	2 masses, restoring force by suspended pendulum	2@160 t	observed under 1997 Typhoon	utilizes 2 ice thermal storage tanks masses; also employs rubber dampers at lower levels

See Appendix-Table 2 for more applications and details.

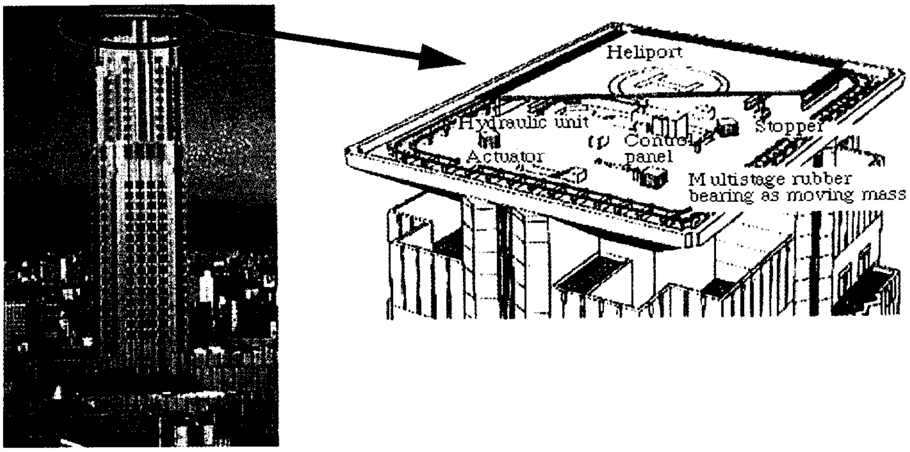


Fig. 20 Hankyu Chayamachi Building (Applause Tower) and heliport used as AMD mass (taken from Takenaka Corporation)

adjusted by the cylinder lock device. The lock is the opening or closing valve within the device, dictating a state of "free" or "lock". The required electricity is only 20 W per device, and thus, in case of blackout, a small emergency generator is capable of booting up the system. The system has been shown to effectively reduce the buildings response in a real earthquake observed on November 11, 1991. Its performance is still being monitored to date (Sakamoto 1993).

6.3.5. Additional applications of active control

Active Gyrostabilizers, which have been observed to perform best in tower-like structures, are now available commercially (Kazao *et al.* 1992). The system is composed of a high-speed rotating flywheel called a "rotor" and a supporting frame for the rotor called a "gimbal". The system has two servo motors: one rotating the flywheel at high speed levels, and the other controlling the angle of the gimbal to actively generate the gyroscopic moment. The moment along the y-axis stabilizes the bending response of the structure on which the gyro is placed. A full-scale demonstration was conducted on a 60 m tower-like structure equipped with two gyrostabilizers and a 408 kg flywheel rotating at 1260 rpm. The system is supported by a gimbal driven by a servo motor with reduction gear. The inherent damping coefficient was found from free vibration tests to be 0.96%. Following the addition of the device, the damping coefficient was found to increase to 8.1%. Under actual wind loads, the peak response acceleration with control was reduced to 30% to 80% of that without control, and the rms response acceleration with control was reduced to 25% to 60% of uncontrolled values.

6.4. Hybrid/Tuned Active Dampers

Another genre of control systems, known as hybrid systems, were also devised to overcome the shortcomings of a passive system, e.g., its inability to respond to suddenly applied loads like earthquakes and weather fronts. In the case of a TMD, the building may be equipped with a passive auxiliary mass damper system and a tertiary smaller mass connected to the secondary mass with a spring, damper, and an actuator. The secondary system is set in motion by the active tertiary mass, and it is driven in the direction opposite to the TMD, magnifying its motion, and hence, making it more effective (Sakamoto 1993, Sakamoto and Kobori 1996).

Hybrid Mass Dampers (HMDs), behave as either a TMD, by utilizing moving mass-supported mechanisms with the same natural period as the building, or as an AMD according to the wind conditions and building and damper mass vibration characteristics (Tamura 1997). The active portion of the system is only used when there is high building excitation; otherwise, it behaves passively. In such systems, the device will typically maintain active control. However, in the event of a power failure or extreme excitations which exceed the actuator capabilities, the system will automatically switch into passive mode until it can safely resume normal operations. In Japan, this combination of passive and active systems has been found to reduce structural responses by more than 50%. While these systems are expensive to install, the reduced operation of the AMD implies low maintenance and operation costs.

A special class of hybrid systems, the Tuned Active Damper (TAD), have become popular in Japan. In fact, most applications involving active control technologies are of the hybrid or tuned active type, as Table 15 reflects. These TAD systems permit the tuning of a traditional active device to the fundamental frequency of the building; however, this definition has also been extended to include bi-directional devices which feature passive control in one direction and active control in another. The following section will discuss some of these applications in more detail.

6.5. Applications of hybrid/Tuned Active Dampers

One notable application of TAD technology is the Landmark Tower (Fig. 21) in Yokohama, a 296 m, mega steel-framed structure, weighing 260,000 tons. In June of 1993, a TAD system

Table 15 Japanese applications of HMD/TADs (18 Buildings)

Name	Location	Date	Height (m)
Osaka ORC200	Osaka	1992	200
Ando Nishikicho Building	Tokyo	1993	68
Dowa Kasai Phoenix Tower	Osaka	1994	145
Hamamatsu ACT City	Hamamatsu	1994	212
Hirobe Miyake Building	Tokyo	1994	30
Hotel Ocean 45	Miyazaki	1994	154
Kansai Airport Control Tower	Osaka	1994	86
Long Term Credit (LTC) Bank	Tokyo	1993	130
Mitsubishi Heavy Industries Building	Yokohama	1994	152
MKD8 Hikarigaoka Building	Tokyo	1993	100
NTT CRED (RIHGA Royal Hotel) Building	Hiroshima	1994	150
Osaka World Trade Center	Osaka	1994	252
Plaza Ichihara	Chiba	1995	61
Porte Kanazawa	Kanazawa	1993	131
Rinku Gate Tower Building	Osaka	1995	255
Shinjuku Park Tower	Tokyo	1993	227
Yokohama Landmark Tower	Yokohama	1993	296
Yoyogi 3-Chrome Kyodo Building	Tokyo	1998	89

was installed on the penthouse first floor (282 m above ground), consisting of two units, each comprised of a three-stage pendulum active in two directions with a tuned spring system and a control system with an AC servomotor (Yamazaki *et al.* 1992). The multi-stepped pendulum (Fig. 21) has a period of 6.0 s. However, this value may be adjusted to values as low as 4.3 s through the use of a natural period regulator which can alter the effective length of the pendulum, in order to correspond to various fundamental periods including that of the tower. Each unit measures 9 meters square, standing 5.0 meters tall and weighing 250 tons, including the pendulum itself which weighs 170 tons. The additional mass was installed in the center of a three-nested structure with the three frames connected by triplicated ropes of element wire. Oil dampers with variable damping coefficients were installed between each frame to ensure stability and safety.

The TAD control system regulates the additional mass through a state-vector feedback system using as state variables the displacement and velocity of the mass and the floor on which the device is installed (Yamazaki *et al.* 1992). Free-vibration tests concluded that the wind-induced response was diminished by 50%, consistent with theoretical estimates. Using a maximum pendulum stroke of only 1.70 meters, the system ensures the habitability requirement of 5.8 cm/s^2 for a 5-year wind (approximately a 43 m/s wind at the top of building) and has reduced the building sway by 50% (Yamazaki *et al.* 1992). A similar device with two TADs is installed in the ACT Tower (Miyashita *et al.* 1998) in Hamamatsu City, Japan, and the control tower of the Kansai Airport (Morita *et al.* 1998), serving the Osaka/Kyoto area. Several other buildings, shown in Table 16, employ similar pendulum systems.

Another hybrid device has been installed in the Ando Nishikicho Building, (Fig. 22a) which is a fourteen-story building that is highly susceptible to strong winds. The system was installed near the top, at the building's center of gravity and consists of bi-directional simultaneous control with oil dampers and laminated rubber bearings as vibration isolators to prevent vibration and noise. The AMD driving system is comprised of an AC servo motor

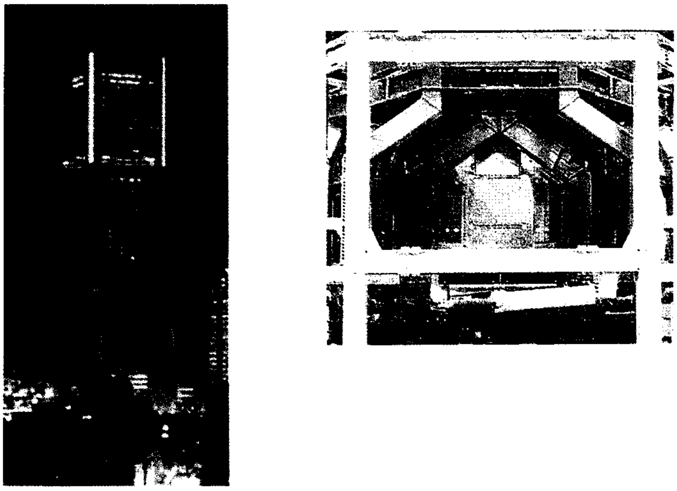


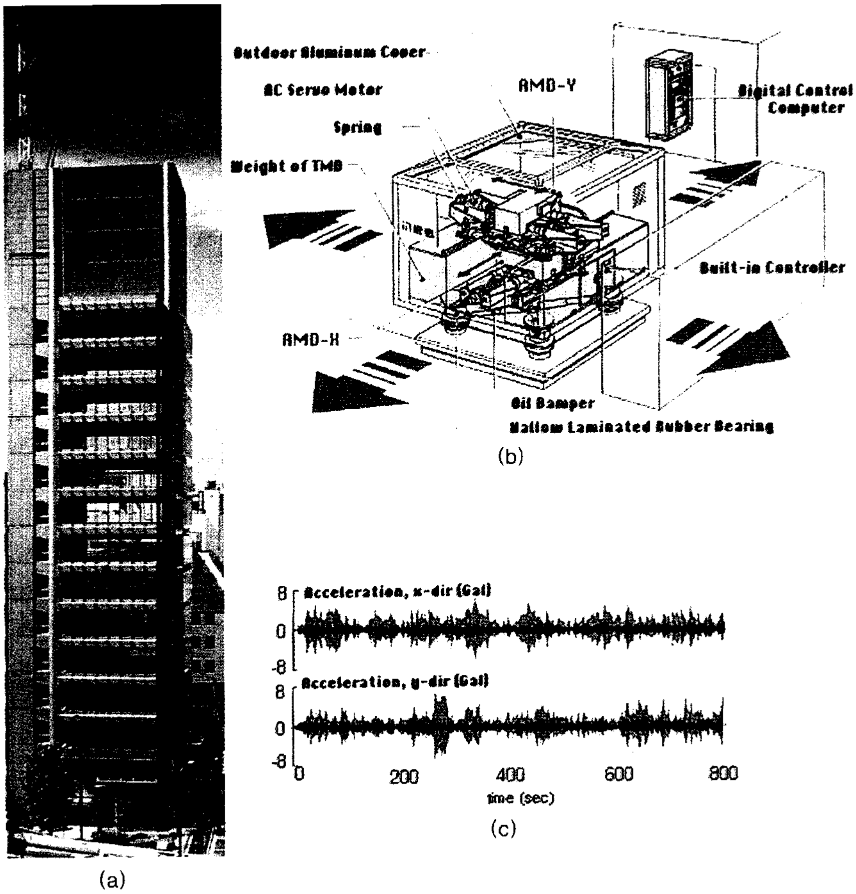
Fig. 21 Landmark Tower and TAD unit installed within (taken from Mitsubishi Heavy Industries, Ltd.)

and ball screws mounted one on top of the other in a criss-cross manner, as shown in Fig. 22b. The TMD weighs 18 tons, approximately 0.3%-0.8% of the building weight, while the AMD units each weigh 2 tons, or 10%-15% of the TMD weight (Sakamoto and Kobori 1993). The system is capable of handling excitations from earthquakes of Japanese Intensity 5 and strong winds with a recurrence interval of 5-20 years. Beyond these levels, the TMD passive control runs until normal excitation levels resume.

Performance tests have shown that the system was successful, increasing damping by 6.4% in the x-direction and 8.5% in the y-direction, reducing the displacements and accelerations in the x-direction 58% and 69%, respectively, while reducing displacements 30% and accelerations 52% in the y-direction, as illustrated by the time histories in Fig. 22c. This system and a similar system in the Dowa Kasai Phoenix Building are capable of performing in large earthquakes. Other similar systems are also shown in Table 16.

The Osaka Resort City (ORC) 200 Symbol Tower (Fig. 23) has also benefited from the installation of two HMD units on the top floor, following its sensitivity to torsional and transverse vibrations. The units (Fig. 23) behave as HMDs in the transverse direction and TMDs in the other. Each unit weighs approximately 100 tons with a ±100 cm stroke and a maximum control force of 7.0 tonf. For safety purposes, the system features air brakes to lock the device in the event of large amplitude structural motion. The HMD's effectiveness was confirmed under winds of 17 m/s, with the structural response suppressed about 1/2 to 1/3 (Maebayashi *et al.* 1993).

Another hybrid system features a weight sliding on rollers which acts like a pendulum, resulting in a smaller system than the equivalent suspended pendulum device, measuring 7.6 m × 4.4 m × 3.5 m high. Therefore, this system overcomes the space requirements that a lengthy pendulum may require. The active forcing of the system is provided by an electric motor (Nishimura *et al.* 1988). The vibration period of the weight can be precisely adjusted because the apparent length of the pendulum can be modified simply by altering the rail angle, permitting a range of tuning frequencies between 3.7 and 5.8 seconds (Tanida *et al.* 1994).



(b)

(c)

(a)

Fig. 22 (a) Ando Nishikicho Building; (b) Schematic representation of its hybrid system; (c) performance of system, controlled and uncontrolled (taken from Kajima Corporation)

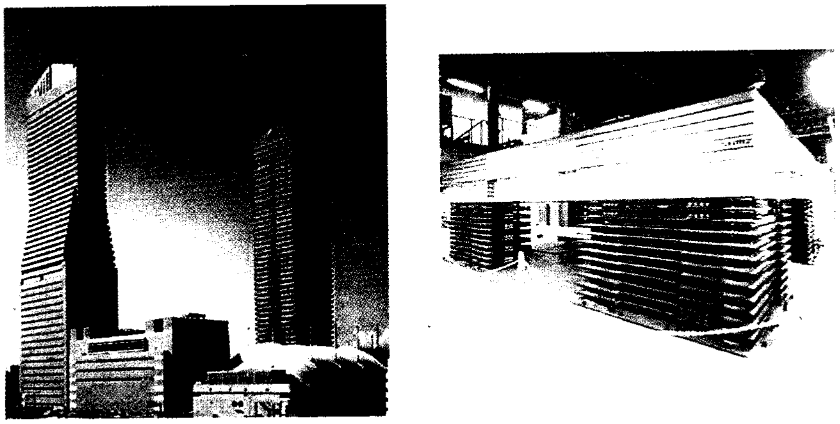


Fig. 23 ORC 200 Symbol Tower and HMD unit installed inside (taken from Yasui Architects & Engineers, Inc.)

Table 16 Details of additional HMD/TAD applications in Japan

Building	System Type	System Dimensions	Additional Information
MHI Yokohama Bldg. (see Figs. 7a and 27) (Miyashita *et al.* 1995)	TAD: 2 stage pendulum, active in 2 directions	5.4 m×5.4 m, ×4.2 m	0.8 m stroke, 80 t (60 t pendulum)
Dowa Kasai Phoenix Bldg. (Sakamoto and Kobori 1993)	2 AMDs+TMD	30 ton TMD+ 2×6 ton AMDs (total wt=42 t)	ball bearings, laminated rubber bearings for TMD; TMD: ±50 cm; AMD: ±100 cm
Kansai Intl Airport Control Tower (see Fig. 28) (Hirai *et al.* 1994)	2 TADs: pendulum, active in two directions	2.2 m×2.2 m ×2.2 m, TAD: 5 t each	control sway and torsion in wind, AC servo motors and ball screws for driving, approximately 50% reduction of wind response
Hotel Ocean 45 (Tomoo and Keiji 1998)	HMD (x-dir)+TMD (y-dir)	100 t mass ±100 cm stroke	multistage rubber bearings, AC servomotors and ball screws, optimal state feedback, VE damping
LTC Bank of Japan (Teramoto *et al.* 1998)	HMD utilizing heat storage tanks	2×30 t mass ±100 cm stroke	reduced max acceleration in wind by 50% and RMS 30%
Yoyogi 3-Chrome Kyodo Building	TMD (x-dir)+HMD (y-dir)	40 ton, bi-directional x 2	analysis results: 50% response reduction

Other notable applications of HMD/TADs in Japan: (18 total Japanese applications)

ACT City Building (multi-stepped pendulum in one direction and passive damper in the other); NTT CRED Motomachi Building (also known as the RIHGA Royal Hotel) (2 stage pendulum, active in one direction-see Figs. 25 and 26); Porte Kanazawa (Aizawa *et al.* 1997); Experimental Elevator Building (Watakabe *et al.* 1998) See Appendix-Table 2 for more applications and details.

This adjustment is accomplished by changing the thickness of the spacers between the rail and the weight. The system has been observed to be particularly effective against long-period vibrations and in reducing the vibrations of the top stories of high-rise buildings. This, coupled with its effectiveness against moderate and small earthquakes and its ability to quickly suppress residual free-vibrations, made it the perfect system to be installed in Tokyo's Shinjuku Park Tower, (Fig. 24a) which houses the Park Hyatt Hotel in its upper floors (Koike *et al.* 1998).

Wind tunnel tests and analytical studies of the 52-story structure indicated strong levels of first mode oscillation in the transverse direction (Kobori *et al.* 1991). For this reason, three of these units (Fig. 24b) were installed on the 38th floor of the South Tower. The auxiliary masses of the system weigh only about 0.25% of the above-ground building weight. Each unit had an auxiliary mass of 110 tons, with a maximum stroke of ±100 cm. Free vibration tests revealed that the inherent damping of 1.1% was increased to 4.9% by the HMD units. Since then, the structure has been monitored under the action of typhoons and earthquakes and was found to reduce the response by about 50% during a 1996 typhoon, as illustrated by

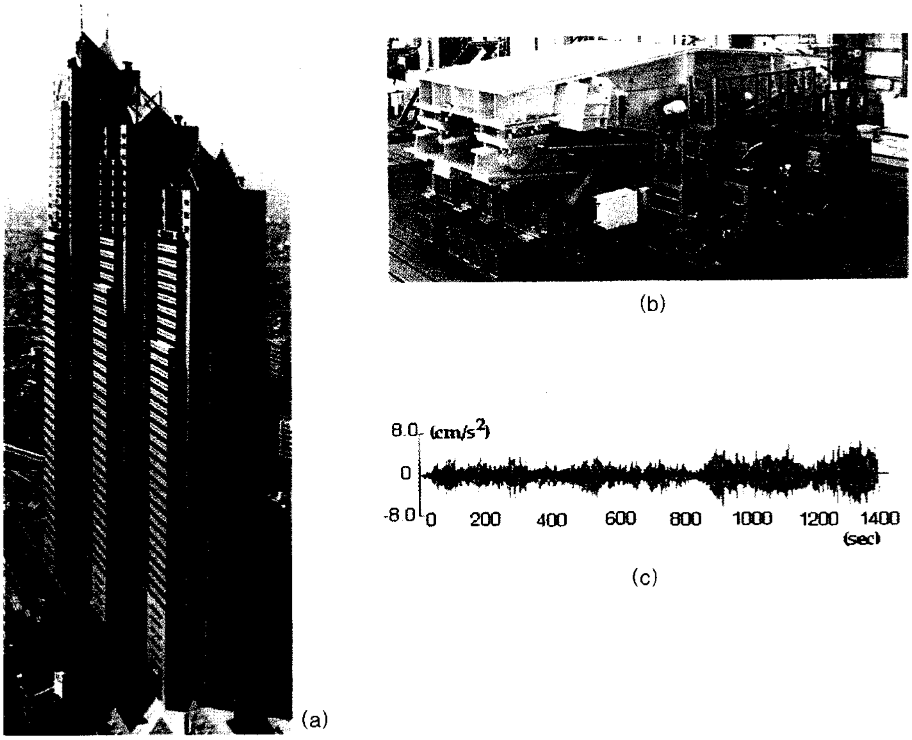


Fig. 24 (a) Shinjuku Park Tower, (b) hybrid system installed and (c) its performance (taken from Kajuma Corporation)

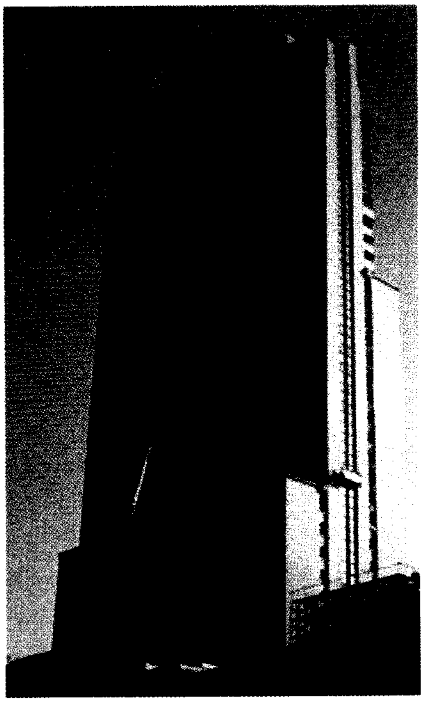


Fig. 25 NTT CRED Motomashi Building (taken from Mitsubishi Heavy Industries, Ltd.)

the acceleration response time history in Fig. 24c. The pink and purple lines denote the uncontrolled and controlled response, respectively (Koike *et al.* 1998).

6.6 Semi-active dampers

Following extensive work in both active and passive control, researchers have developed a new generation of control devices based on semi-active control, which combine the best features of their parent devices. This includes, for example, control of the secondary system's damping or stiffness characteristics, illustrated in Fig. 10. Possessing the adaptability of active control without the potential for instability and demand for external power sources, semi-active systems can quickly respond to a sudden gust front or earthquake and provide auxiliary damping dependent on the excitation type and level. Typical examples of these technologies include variable orifice control and dampers with controllable fluids such as electrorheological (ER) and magnetorheological (MR) devices. Preliminary work indicates that such devices can approach performance levels obtained by active systems without the risk of destabilization or high power requirements (Spencer and Sain 1997, Symans and Constantinou 1999). This latter feature is particularly attractive. Since the devices do not introduce mechanical energy into the system, power requirements are relatively low, insuring that the system can remain operational even on battery power during extreme events such as earthquakes.

Semi-active devices range from impact configurations to variable orifice concepts for applications to conventional hydraulic fluid dampers (Symans and Constantinou 1996). Such concepts may also be extended to TLCDs by implementing the traditional u-shaped container with an adaptable valve (Yalla and Kareem 1998). By regulating the valve opening, and thus the valve head loss coefficient, the level of damping can be adaptively varied with changes in loading intensity. The use of a small voltage source thus permits the traditionally passive system to maintain optimal damping ratios for the device at all levels of excitation. In the case of TSDs, an analogous semi-active control would adjust the screen or vane openings or control a membrane over the free surface for optimum damping (Kareem and Tognarelli 1994). While semi-active devices which employ forces generated by surface friction have also been considered, the work in controllable fluid devices has gained much notoriety for potential semi-active applications, the details of which are briefly presented in the following section. Numerous experimental studies, including full-scale work on bridges for seismic retrofit (Sack and Patten 1994), confirms the applicability of this emerging technology.

6.6.1. Electrorheological/magnetorheological dampers

The motivation for the development of controllable fluids for semi-active applications was partially the result of the unsuccessful search for valves that would respond quickly enough to regulate semi-active orifice devices efficiently and effectively. Since these controllable fluid systems do not require moving parts such as valves, they are considered a viable technology for application in civil engineering structures. Currently, two forms of controllable fluid semi-active dampers are being investigated in the United States: the ER (Stevens *et al.* 1984, Gavin and Hanson 1994, Morishita and Mitsui 1992, Morishita and Ura 1993, Makris *et al.* 1995) and MR (Spencer *et al.* 1996, Spencer *et al.* 1998, Gordaninejad *et al.* 1998) dampers. The "smart" fluids provide the energy dissipative mechanism for these devices under the application of an electrical or magnetic field, as their respective names suggest. As a result, the degree of polarization of the fluid, and thus its dissipative capacity, may be modified by the regulation of the voltage source which controls the fields. Unlike variable orifice systems which are limited by the performance of their valves, the electro or magnetic fields utilized by these systems activate in mere milli-seconds.

Like hybrid devices, semi-active devices provide passive control under normal operations without any power requirements, but respond quickly to provide optimal levels of damping during seismic events. In fact, such systems can be powered in their "active" mode by traditional, low-voltage power sources. In light of these attractive features, semi-active controllable fluid dampers offer an alternative venue for the solution of the ongoing problem of structural vibrations.

7. Conclusions

A discussion of the various techniques used to mitigate building motion was presented, including structural and aerodynamic solutions. This paper also addressed a number of passive and active motion control devices for improving the performance of tall buildings under wind loads for human comfort considerations, as well as several seismic applications. Detailed examples of practical applications of such devices to buildings in Australia, Canada, China, Japan, and the United States were provided.

In light of the wide spectrum of methods to mitigate wind-induced motion presented in this paper, it is perhaps best to conclude with an innovative project which integrates several of these design approaches. The Millennium Tower concept, proposed for construction in Japan, soars 762 m skyward with a base the size of Tokyo's Olympic Stadium (Sudjic 1993). The structure exploits an aerodynamically favorable shape through its circular plan, coupled with the benefits of tapering with height, permitting it to perform efficiently in wind. The resulting cone shape, shown in Fig. 29a, concentrates its mass in the lower floors to additionally improve the structure's resistance to earthquakes. The performance in wind is further supplemented by the inclusion of a "through-building" opening near the top of the structure, shown in Fig. 29b. Meanwhile, the structural system relies on transfer girders, also shown in Fig. 29a, to distribute gravity loads to the exterior double helix and column system. This exterior helix casing not only carries the structure's loads, but also helps to disrupt the wind flow around the structure, further improving the vibration performance. In addition to these aerodynamic and structural modifications, the incorporation of an auxiliary damping system is also planned. As shown in Fig. 29c, the systems of water tanks would be located at two levels in the structure and serve as a hybrid liquid damper system, combining the benefits of passive control at low excitation levels with the optimum control provided by the active driving of the water levels in the tanks available for extreme events, an attractive feature in light of the typhoons which frequent this region.

While the incorporation of such technologies permit today's structures to reach even greater heights, it is interesting to note that these concepts are not new. Nearly 1200 years ago, the ancient Japanese builders were building their own Millennium Towers. The Japanese utilized many of the concepts presented here in the design of their famous pagodas (Fig. 30), making these structures resistant to both the action of typhoons and earthquakes. The secret of their

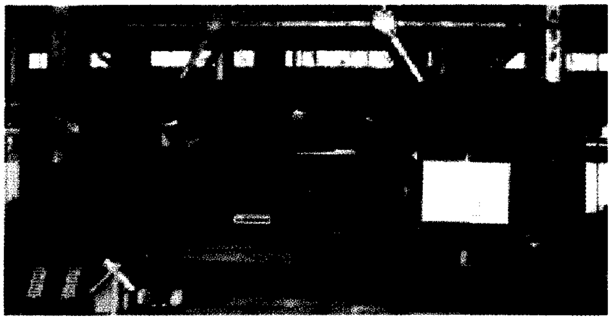


Fig. 26 HMD installed in NTT CRED Motomashi Building (taken from Mitsubishi Heavy Industries, Ltd.)

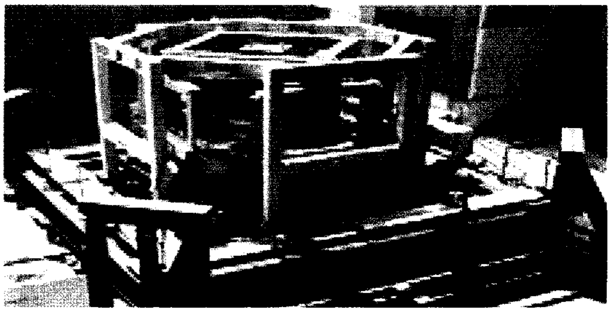


Fig. 27 HMD installed in Mitsubishi Heavy Industries Building (taken from Mitsubishi Heavy Industries, Ltd.)

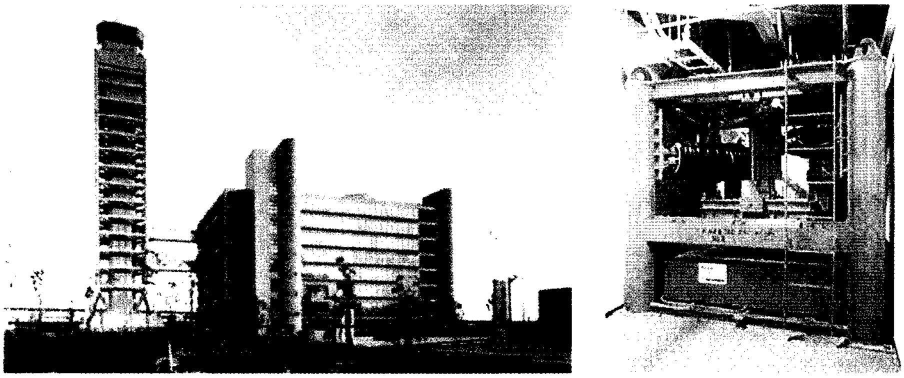


Fig. 28 Kansai International Airport Tower and HMD unit installed within (taken from Yasui Architects & Engineers, Inc.)

enduring strength and stability lies in their tapered configuration, the variation of their cross-section with height, and the fact that the energy dissipation occurs at each level, since the levels are not attached to one another and may independently slide back and forth. The shinbashira, the central pillar attached to the ground, serves as a snubber, constraining each level from swinging too far in any direction. As the independent levels impact this fixture, energy is introduced which is dispersed through soil damping. Thus, the concepts of secondary inertial systems, friction and

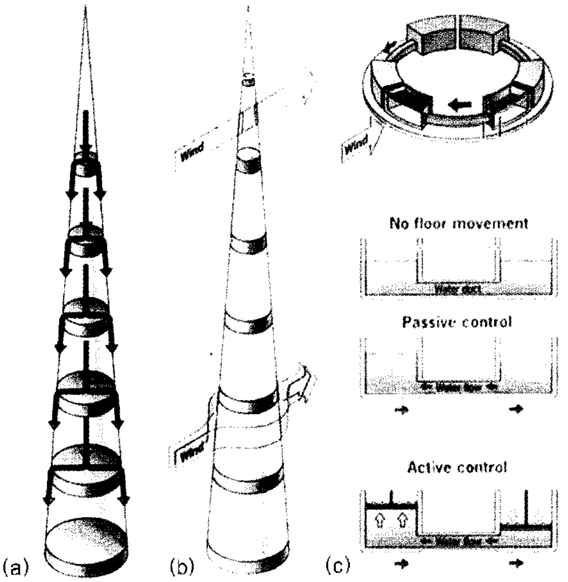


Fig. 29 Design concepts for Millennium Tower: (a) load transfer; (b) aerodynamic modifications; (c) auxiliary damping scheme (taken from Sudjic 1993)

Fig. 30 Schematic of Japanese Pagoda (taken from Winds 1998)

impact dampers, and aerodynamic tailoring are not so revolutionary. In fact, the same strategies exploited in modern times for urban skyscrapers, today's counterpart of the pagoda, have been ingeniously tapped by the ancient Japanese builders for centuries.

Acknowledgements

The authors gratefully acknowledge the funding for this paper, provided in part by NSF Grant # CMS95-03779-004, CMS95-22145, CMS94-02196 and ONR Grant # N00014-93-1-0761. The second author was supported by the NDSEG Fellowship through the Department of Defense and supported in part by the NSF GRT Fellowship.

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Appendix of tables

Table 1 Notation used in this Appendix

Notation	Meaning	Notation	Meaning	Notation	Meaning
AC	AC Servo Motor & Ball Screw	HR	Hallow Rubber Bearings	RS	Roller & Spring
AGS	Active Gyro Stabilizer	LB	Linear Bearing	S	Steel
AMD	Active Mass Damper	LCD (-PA)	Liquid Column Damper (with Period Adjusting Mechanism)	SLD	Sloshing Deep Water
C	Concrete	MP	Multiple Pendulum	SLS	Sloshing Shallow Water
CC	Circular	MR	Multiple Rubber Bearing	SRC	Steel and Reinforced Concrete
DD	Double Donut Type	HR	Hallow Rubber Bearings	TMD	Tuned Mass Damper
ξ	Damping Ratio (percent critical)	OD	Oil Damper	UT	U-Tube
EQ	Earthquake	PE	Pendulum	VD	Viscous Damper
HA	Hydraulic Actuator	R	Response	VED	Visco-elastic Damper
HMD	Hybrid Mass Damper	RC	Rectangular	W	Wind

Table 2 Applications of inertial type dampers

No	Building	Structure/ Use	Year	No. of Stories Height/ Area[m/m^2]	Building Weight generalized [t]	Period (x, y, tor) [s]	Equipment/ Mechanism*	Mass of Damper [t]	Mass Ratio (Generalized)	Max Stroke/ Actuator Specs	Intended Application/ Performance
· 1	ACT Tower, Hamamatsu	S/Office, Hotel	1994	+45.-2 212.0/ 151000	110000	4.52 4.73	TMD(x), HMD(y) MP/OE	90×2 (x, y)	0.16%	±90 cm (x) ±150 cm (y)	W, ξ=0.9%(ξ); ΔP=10% (ψ)
· 2	Akita Tower, Akita	S/Tower	1994	-- 112.1/ 4773	2186 (1012)	2.30 2.30	TMD/MR/OE	15 15	0.69% 0.69%	±60 cm	EQ/W
· 3	Ando Nishikicho Bldg., Tokyo	S/Office	1993	+14. -2 68/ 4928.3	2600	1.47 1.39 --	HMD/HMD HR AC	20 (TMD)+ 2(AMD) ×2	0.85%	TMD ±15cm AMD ±50 cm	EQ/W, ξ= +6.4%, +8.5% (ξ, ψ); P=-69% (ξ), -52%(ψ)
· 4	Building M, Osaka	S/ Residence, Office	1994	+9 30.4/345		0.75 0.65	TMD (2 dir) MR	2.44 +0.79	0.9%	2.6 cm	W R=-33%
· 5	Building S, Osaka	S/Office	1994	+9 30.9/--		0.59, 0.78, 0.55	TMD	1.0 (y) 1.75 (torsion)			ξ= +2.4% (y); +2.7% (torsion)
· 6	Chiba Port Tower, Chiba	S/Tower	1986	-- 125/2308	1950 (1200)	2.25 2.70	TMD/RS/VD	10.0 15.6	0.51% 0.80%	±100 cm	EQ/W R=55% (20 m/s)
· 7	Chifley Tower, Sydney	S/Office	1994	-- 209/--			TMD/PE	400	2%	±910 mm	W ξ=2-4%
· 8	Citicorp Building, New York	S/Office	1978	-- 278/--			TMD	410 bi-axial		2.18 m	W R=45-50%
· 9	CN Tower, Toronto	S/C/Tower	1975	-- 553/--			TMD/PE	20			W
· 10	Complex Building in Shinjuku	S/Complex		+36, -4 150/ 80000			AMD, Gear PD Linear Motor	30×2		±1 m	
· 11	Crystal Tower, Osaka	S/Office	1990	+37.-2 157/ 85994	44000	3.60 4.10	TMD (Ice Thermal Tank) PE, OE	180 (x) 360 (y)	0.41% 0.82%	±25 cm	W R=1/2
· 12	Dowa Kasai Phoenix Building, Osaka	S/Office	1994	+29. -3 145.4 30,370	27000	2.80 2.88 2.17	HMD/HMD HR AC	30 (TMD)+ 6x2 (AMD)		TMD: ±50 cm, AMD: ±100 cm	W/EQ, W R=-45%(x), -60%(y)
· 13	Elevator Tech. Lab., Tokyo	S/ Exp. Tower	1992	-- 60.0/630			AGS				W
· 14	Experimental Tower	S/Tower	1993	6 18.3	154.4	1.06 1.11 --	Active Fin 2 m×1 m, 0.1 m thick, AC				
· 15	Experimental Elevator Building	S/Tower	1997	-- 145	6877	2.86 2.44	HMD	34.5 (x) 26.8 (y)	1.73% (x) 1.41% (y)	±40 cm (x) ±40 cm (y)	W R=-1/3 ξ=+8-11%
· 16	Fujita Experimental Building	RC/Base Isolation System	1995	3 10		0.8 0.8 --	HMD AC	4.13 4.34	(1.3%)		ξ=+11%
· 17	Fukuoka Tower, Fukuoka	S/Tower	1989	-- 150.7/ 1808	4000	3.30 3.20	TMD/RS/ OE	30 25	0.75% 0.63%	C±110 cm	EQ/W
· 18	Gold Tower, Kagawa	S/Tower	1988	-- 144/ 1193		2.69 2.50	TLD/SLD/RC 45×250×10 0-16 (53 cm)	9.6			W R=1/2~1/3

Table 2 Continued-I

No	Building	Structure/ Use	Year	No. of Stories Height/ Area[m/m^2]	Building Weight generalized [t]	Period (x, y, tor) [s]	Equipment/ Mechanism*	Mass of Damper [t]	Mass Ratio (Generalized)	Max Stroke/ Actuator Specs	Intended Application/ Performance
· 19	John Hancock, Boston	S/Office	1977	-- 243.84/--			TMD	2×300 uniaxial			W R=40-50%
· 20	Hankyu Chayamachi Bldg. (Applause Tower), Osaka	S/Office, Hotel	1992	+34 -3 161/89686	13943	4.7 4.8 --	AMD/AMD (Helideck) MR HA	480 (x, y)	(x: 3.4%) (y: 3.9%)	5t×2×2 ±30cm	W ξ=+9.2%
· 21	Herbis Osaka, Osaka	S/Office, Hotel	1997	+40 -5 190/--	22749	5.1 5.4 5.5	AMD (Heat Storage Tanks)	2×160	1.41%	±30 cm 5t×2	W
· 22	Hibikiryokuchi Sky Tower, Kitakyushu	S/ Observatory	1991	-- 135/--			TMD				
· 23	Higashiyama Sky Tower, Nagoya	S/Tower	1989	-- 134/2929	(1870) (1960)	2.20 1.98	TMD/PE/OE	19.8 (x, y)		±15 cm	W
· 24	High-rise Housing Exp. Tower	S/Truss Tower	1995	-- 108/--	730		AGS	Flywheel 0.8×2	(0.7%)		W R=-50%
· 25	Hirobe Miyake Bldg., Tokyo	S/Office, Residential	1994	+9 30.65/--	273	0.63 0.81 0.48	HMD HR AC	2.11 -	(1.7%)	0.806t 2.2kW ±30 cm	W ξ=+14.4% R=-66%
· 26	Hobart Tower, Tasmania	S/Tower		-- 105/--			80 SLS				
· 27	Hotel COSIMA, Tokyo	S/Hotel	1994	+26 -3 106.35/9798	4600 (1160)	2.0 2.1	LCD-PA UT 600 x150x290 -4	58	1.3%	±72 cm	W R=-40%
· 28	Hotel Ocean 45, Miyazaki	S/Hotel	1994	+43. -2 154.3	83,650	3.6 3.9	HMD (x) TMD (y) MR AC	120×2 (x, y)	0.29%	±50 cm (y) ±100 cm (x)	W R=-35%
· 29	Huis Ten Bosch Tower, Nagasaki	S/Tower	1992	-- 105.0/--	4599	1.75 1.75	TMD/MR/ VED	7.8	0.17%	±80 cm	W R=1/2~1/3
· 30	Hyatt Hotel, Osaka	S/Hotel	1995	28 112/--	43000		LCD-PA	104	(0.24%)		W R=-30%
· 31	KS Project, Kanazawa	S/--	1993	-- 121/--			AMD				
· 32	Kansai New Airport Tower, Osaka	S/Tower	1994	-- 86/--	2570	1.25 1.25	HMD/HMD PE, AC	5×2 (x, y)	0.19%	7.5kW×2 ±30 cm	W R=-50% ξ=5~7%
· 33	Kyobashi Seiwa Bldg., Tokyo	S/Office	1989	+11.-1 32.8/423	340	1.1 1.5 1.9	2 AMD PE HA	4.0 1.0	1.5%	1t, 0.25t, 22 kW, ±25 cm	EQ/W R=1/3 (23 m/s)
· 34	Landmark Tower, Yokohama	S+SRC/ Office, Hotel	1993	+70 -3 296/231060	261000 (50000)	5.1 5.1 3.6	HMD/HMD MP, AC	170×2 (x, y)	(0.68%)	30t×2×2 60kW×2 ±170 cm	W R:-48% ξ=+10%
· 35	L.T.C. Bank of Japan Bldg., Tokyo	S/Office	1993	+21. -5 130.0/62821	39800 (30,400)	2.4 2.6	HMD (Heat Storage Tanks) MR HA	195 (x, y)	(0.65%)	30t×2×2 60kW×2 ±100 cm	EQ/W ξ=+6.3~14.3% W R=-30% EQ R=-50%
· 36	MHI Yokohama Bldg., Yokohama	S/Office	1994	+34 -2 151.9/ 110918	61800	3.9 3.8	HMD (2 dir) PE	80 (PD=60)	0.13%	0.8 m	W

Table 2 Continued-II

No	Building	Structure/ Use	Year	No. of Stories Height/ Area[m/m^2]	Building Weight generalized [t]	Period (x, y, tor) [s]	Equipment/ Mechanism*	Mass of Damper [t]	Mass Ratio (Generalized)	Max Stroke/ Actuator Specs	Intended Application/ Performance
· 37	MHS Design Office, Tokyo	S/Office	1988	+8.-1 26.8/940		0.7	TMD/PE/OE	2.5×2			EQ/W
· 38	MKD8 Hikarigaoka, Tokyo	S/Office, Hotel	1993	+24. -3 100/--	29000 (9200)	2.0 1.7	HMD/HMD PE, AC	22×2 (x, y)	(0.47%)	15kW×2×2 x: ±25 cm y: ±40 cm	W ξ=+6% R=-30%
· 39	NTT CRED Motomachi Bldg., Hiroshima	S/Hotel, Office	1993	+35 -2 150/ 170000	83000	3.8 3.6	HMD (1 dir) PE	110 (PD=80)	0.13%	0.6 m	W
· 40	Nagasaki Airport Tower, Nagasaki	S/Tower	1987	-- 42.0/--	170	0.93 0.93	TLD/SLS/CC 38 ϕ×7-175 (4.8 cm)	0.95	0.56% (1.5%)		W R=35% (20 m/s)
· 41	Nanjing TV Tower, Nanjing, China	C/Tower	1999	-- 340/--	-- (2431)	5.05	AMD	~60	1%	1.5 m 50 kN	
· 42	Narita Airport Tower, Chiba	S/Tower	1993	-- 87.3/--	4140	0.78 0.75 0.48	TLD/SLS/CC 50 ϕ×12.5-2310 (3.64 cm)	16.5	0.40%		W
· 43	O Building, Nagasaki	S/Office	1990	-- 27/--			TLD				
· 44	Ohita Prefecture Cultural Hall	S+SRC/ Office & Convention	1998	+21 -3 101 m 83,297	20,000		HMD (Linear Motor)	25×2	0.25%	±800 cm	Active: W R=1/3 (26 m/s) Passive: W R=1/2 (50 m/s)
· 45	ORC 200 Symbol Tower, Osaka	S/Office, Hotel	1992	+50. -3 200/ 72097	56680 (15600)	4.72 4.4 --	TMD (x) HMD (y)/ MR/VED	115×2 (x, y)	0.41% (1.47%)	x: ±50 cm y : ±100 cm	W R=50%
· 46	Osaka World Trade Center Building., Osaka	S/Office	1994	+52. -3 252.0/ 147000	75000	5.84 5.84	HMD/HMD MP AC	50×2 50×2		75t×2×2 37kW× 2×2 ±160 cm	W
· 47	Pipe Lab., Kanagawa	S/ Experimental Tower	1990	+12 33.8/300		2.09	TLD/SLS	1.1			EQ
· 48	Plaza Ichihara, Chiba		1995	+12 61/--	5760		HMD	14	0.24%		
· 49	Porte Kanazawa, Kanazawa	S/Hotel, Office	1993	+30 -2 131/--	10150	2.9 (x) 2.5 (tor)	HMD MR AC	50×2	(0.49%)	5 t×2 ±15 cm	ξ=+2% (x) +5.5% (tor)
· 50	Rinku Gate Tower Bldg, Osaka	S/Hotel	1995	+56 -2 254/--	75000	4.4 4.4	HMD MP AC				
· 51	Riverside Sumida Building, Tokyo	S/Office, Residential	1994	+33. -2 134.4/ 60000	52000	-- 2.86 2.32	AMD (y) LB AC	15×2	(0.058%)	8.7t×1×2 55kW× 1×2 ±100 cm	EQ/W D=+7.1%(y) EQ R= -20~30%
· 52	Rokko-Island P&G, Kobe	S/Office	1993	+31. -1 131/46076	27000	1.64 2.94 1.69	TMD (Ice Thermal Tank) PE/OE	y: 90×2 torsion: 90	1.0%		W ξ=y: +5.4% R=-60%
· 53	Sea Hawk Hotel, Fukuoka	S/Hotel	1995	36 143/--	42000		TMD (Water Tank) PE	112~132	(2-2.5%)		W ξ=+3.6% R=-50%
· 54	Sendagaya INTES Bldg., Tokyo	S/Office	1991	+11. -1 58.0/10602	3280	-- 1.7 2.1	AMD/AMD PE HA	36×2 (y)	(2.2%)	5t×1×2± 15 cm	EQ/W WR=-18% (y) WR=-28% (tors) ξ=+8%

Table 2 Continued-III

No	Building	Structure/ Use	Year	No. of Stories Height/ Area[m/m²]	Building Weight generalized [t]	Period (x, y, tor) [s]	Equipment/ Mechanism*	Mass of Damper [t]	Mass Ratio (Generalized)	Max Stroke/ Actuator Specs	Intended Application/ Performance
· 55	Sendai AERU	SRC Multi-purpose	1998	+33 -3 145.5/ 73,131			TMD Laminated Rubber+ Coil Spring	100 (6.7 m × 7.2 m × 3.4 m, two ways each) × 2	0.7%		W R=1/2
· 56	Shanghai World Financial Center	S+RC/Office, Hotel	2001 (?)	+94 -3 460/ 333600			SLS, ϕ 7.5 m	8 × 100 t			W
· 57	Shimizu Corp. Tech. Lab., Tokyo	S/ Experimental Tower	1991	+7. -1 30.0/-	400	0.88 1.05	HMD/AMD MR AC	4.95 (x, y)	(1.2%)	1.2t × 2 7.5kW × 2 ±23 cm	EQ/W ξ=20%(15 m/s)
· 58	Shin-Yokohama Prince Hotel, Yokohama	S/Hotel	1991	+42. -3 149.3/ 76027	26400 (10500)	4.43	TLD/SLS/ CC 200 $\phi \times$ 20.5-270 (9.85 cm)	101.7	0.39%		W R=1/2 (20 m/s)
· 59	Shinjuku Park Tower, Tokyo	S/ OfficeHotel	1994	+52. -5 226.5/ 264100	130000	4.5 5.24 3.98	HMD (y) VR AC	110 × 3 (y)	(0.25%)	75kW × 1 × 3 ±100 cm	W D=+4% R=~50%
· 60	Sydney Tower, Sydney	S/Tower	1981	305/--			TMD/PE	180 40		±150 mm	W R=-40-50%
· 61	T Building	S/Office	1997	9 31.1/--	842	0.81 0.66	TMD PE	8.5	1%		W: R=-50% ξ=+5.6%
· 62	TYG Building, Atsugi	--/Office	1992	-- 58.6/--	7450	1.89	TLD (DD) D1=68.6 cm d1=48.6 cm D2=42 cm d2=29.5 cm	18.2	0.24	N/A	W
· 63	Takenaka Corp. Tech. Lab, Tokyo	S/Exp. Tower	1989	+7 22.8/--		1.11 1.54	AMD/AMD PE HA	6.0 (x,y)		10t × 2 ±95 cm	EQ/W
· 64	Toda Corp. Tech. Lab., Tokyo	S/Exp. Tower	1992	+6 18.9/--		1.33 1.13	HMD/HMD LB HA	5.5 4.11	1.0%	1t × 2, 11 kW, 5 cm	EQ/W: R=1/3~1/4 ξ=2~5%
· 65	Tokyo International Airport Tower, Haneda	S/Tower	1993	-- 77.6/--	3240	1.3 1.0	TLD/SLS/ CC 60 ϕ 12.5-1404 (5.3 cm)	22.7	0.70%		W R=1/2 (20 m/s) R=2/3 (20 m/s)
· 66	Washington National Airport Tower, USA	S/Tower	1997	-- 67.5/--		1.42 1.52	TMD	20 kips			W ξ=+3%
· 67	Yokohama Marine Tower, Yokohama	S/Tower	1987	-- 101.3/3326	540	1.82	TLD/SLS/ CC 50 $\phi \times$ 5-390 (2.2. cm)	1.54	0.29%		W R=1/3 (20 m/s)
· 68	Yoyogi 3-Chrome Kyodo Building	S+SRC School	1998	+20, -2 89 56,300			TMD (x)+ HMD (y) AC Servo	40 (4.7 m × 4.7 m × 2.4 m, two ways each) × 2	0.1%		W: 1/2

*for TLD: container dimensions-no. of units (level of liquid in unit)

(*Communicated by Managing Editor*)