Recent Applications of Structural Control Systems to High-Rise Buildings

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ABSTRACT

This paper reports recent applications of structural control systems to high-rise buildings built in downtown Osaka. Two types of structural control, one for earthquakes and the other for strong winds, are presented. For control of earthquakes, two buildings are outlined: one building is equipped with hysteretic steel dampers made of low-yield steel and another building is equipped with oil dampers. Their design concept, design criteria, and verification are presented. For control of strong winds, two design projects are outlined: one building has tuned mass damper (TMD) systems in which an ice thermal storage tank is adopted as the moving mass, and another building has dampers made of high-damping rubber installed in the precast concrete panels used for exterior walls. On-site tests are conducted to verify the performance of the dampers, and numerical analyses are conducted to measure the enhanced habitability performance caused by the introduction of the dampers.

INTRODUCTION

Lessons learned from the 1995 Hyogoken-Nanbu earthquake became a catalyst to change structural engineering practices in Japan. The essence of the change was to improve the building's function after an earthquake as well as to enhance the seismic performance of structural members. This change translated into three engineering considerations:

(1) Concentration or reduction of damage in structural members to facilitate restoration after the earthquake.
(2) Reduction of damage (including prevention of toppling) to furniture inside the building to reduce human injury and to maintain the building's function.
(3) Reduction of damage in nonstructural elements to facilitate the recovery of building function.

These aspects led to the adoption of seismic isolation and response control technologies in structural engineering. The employment of these technologies is categorized into three types:

(1) Concentration of structural deformation into structural elements, including seismic isolators, that absorb energy effectively. This method ensures good seismic performance but requires special expertise in the engineering of devices.
(2) Distribution of energy absorbing devices over the structural system. This method is less efficient than the method described in (1) but requires less special expertise to implement since this technology is only a slight extension of the conventional seismic design. Because of its readiness in application, use of this method has been widespread.
(3) Response control technologies are also available to improve performance under strong wind excitations. The technologies have been developed to improve the inhabitants' comfort during strong wind.

This paper introduces a few recent applications of the technologies in (2) and (3) in real building construction. Structural control is commonly categorized into passive and active (including semi-active) systems based on whether a force-driving device is incorporated. This paper focuses on technologies related to passive systems. Although seismic isolation is regarded as a variation of the passive system, the paper does not cover this technology.

APPLICATION OF STRUCTURAL CONTROL SYSTEMS IN JAPAN

Recently, two symposiums, "The Second Japan National Symposium on Structural Control [1]" and the "2002 Passive Control Symposium" [2], were held in Japan. "The Third World Conference on Structural Control" [3] was also held in Italy between the two symposiums, but the conference was mainly for the active system, semi-active system and building isolation system.

"The Second Japan National Symposium on Structural Control" included both the active and semi-active systems, but papers on passive systems constituted more than half of the symposium. About one third of the papers on the passive system were associated with actual constructions [4]. This reflects the active and intensive participation of practitioners in the passive system design and construction.

Popularly studied devices were steel dampers, friction dampers, lead dampers, oil dampers, viscous dampers, visco-elastic dampers, magneto-rheological dampers, Zinc-Aluminum (Zn-Al) alloy dampers and tuned mass dampers. All of these dampers were applied in real constructions in Japan.

Among these dampers, steel dampers and oil dampers are most commonly adopted, because of their stable, robust characteristics, which facilitate the structural engineering process. Steel dampers are also advantageous in cost.

In the "2002 Passive Control Symposium" held after the above symposium, passive systems applied or planned to be applied in real construction were presented. In this symposium, combined use of multiple types of dampers was also explored [5,6].

This symposium proposed "Theme Structures" [7] that can be commonly used in evaluating the performance of passive control systems, and introduced a manual for the design and construction of passive controlled buildings [8]. These activities shall enhance the sound development and dissemination of passive control technologies. Examples of the latest applications of passive control systems designed by the writers' groups are introduced in the following sections. The examples include four types of passive systems applied to three buildings, designated as Building A, B, and C, whose sites are shown in Fig. 1. All of them are located in downtown Osaka, near the JR Osaka station.

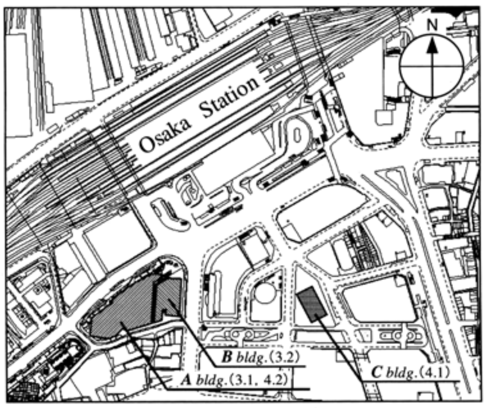


Fig. 1 Construction sites of Buildings A, B, and C

PASSIVE SYSTEMS EFFECTIVE FOR LARGE EARTHQUAKES

Application of Passive Systems with Steel Dampers

(1) Outline of building

Building A has four stories underground and 28 stories above ground. The building complex functions as parking and shops in underground floors, theater and shops in lower floors above the ground, and offices in floors eight to 27 stories above the ground. The outline of the building is shown in Table 1. The sectional layout and the perspective of the building are shown in Fig. 2 and Fig. 3, respectively.

(2) Outline of structural control design

In this building, buckling-restrained braces are used for the structural control system. The brace has a wide-flange cross-section surrounded by a steel tube that prevents the brace from buckling. The deformations of an ordinary brace and a buckling-restrained brace are shown in Fig. 4. Thanks to the restraining tube, the brace can sustain equal force in both compression and tension. It is also notable that this system is very easy to fabricate. The mid-portion of the brace is arranged with low-yield steel (LY225), and the cross-section area of this portion is set to be smaller than the other portion to restrict the yielding and energy dissipation to the mid-portion. The outline of this brace is shown in Fig. 5, and a photo of the brace installed in the building is shown in Fig. 6.

Table 1 Building A description

Location	Osaka, Japan	
Main use	Office, Theater, Shops	
Total floor area	106,629.6 m^2	
Building area	6,241.5 m^2	
Height	138.6 m	
Completion	September, 2004	
Floors	28 above ground 4 underground	
Structure	Columns: Concrete-filled steel tube others: Steel	
Natural periods	X-direction	3.70s
	Y-direction	3.69s
	Torsional direction	4.18s

Fig. 3 Perspective of Building A

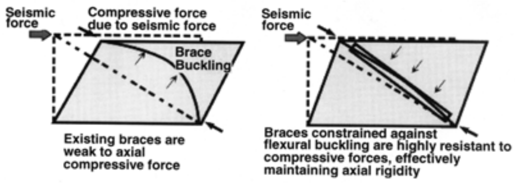


Fig. 4 Deformation of existing brace and brace constrained against flexural buckling

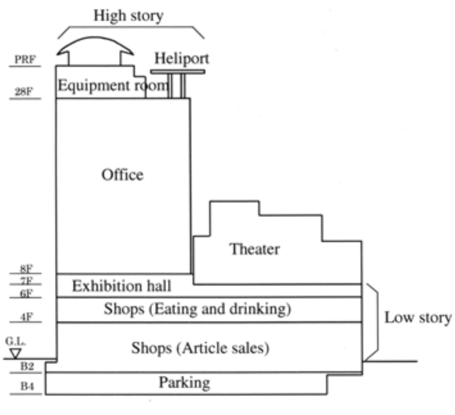


Fig. 2 Sectional layout of Building A

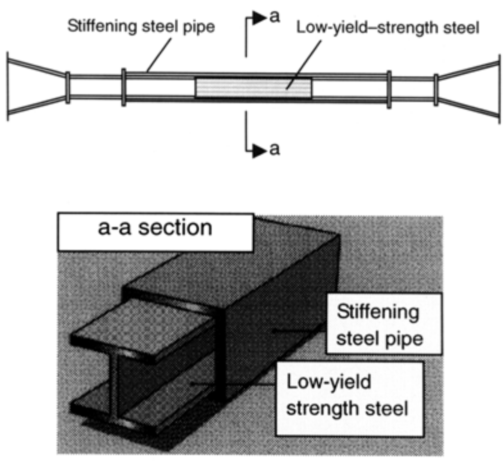


Fig. 5 Outline of buckling restrained brace

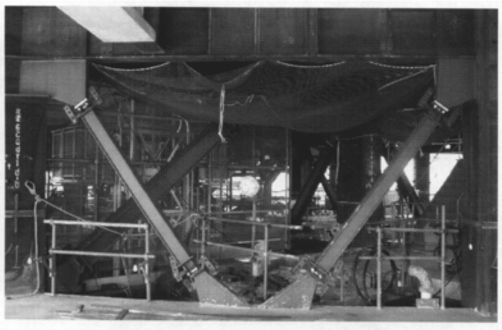


Fig. 6 Photo of buckling-restrained brace installed on site

Design of buckling-restrained braces

The lateral load resistance of this building is shown in Fig. 7. Here, the buckling-restrained braces are designated as dampers because of their capacity to dissipate energy. The optimum strength for the dampers to maximize their energy dissipation is controlled by the ratio of the damper stiffness to the frame stiffness. This stiffness ratio is referred to as k, and the larger the stiffness ratio, the larger the equivalent viscous damping becomes [9]. Based on this finding, buckling-restrained braces were placed in as many places as possible, while minimizing the eccentricity in the structural plan. The arrangement of the braces in the fifth and twentieth floors is shown in Fig. 8, and a framing elevation is shown in Fig. 9. The cross sectional areas of the buckling-restrained braces were determined using the following processes:

(a) Determine the cross-sectional area of the portion of the brace having the low-yield steel so that the optimum strength can be achieved.
(b) Maximize the brace stiffness by minimizing the length of the smaller cross-section of the brace while maintaining the length of its plastic deformation.
(c) Enlarge the cross-sectional area of the brace in the other portion to increase the stiffness ratio k.

Plastic deformation capacity of buckling-restrained brace

Plastic deformation capacity of the buckling-restrained braces is controlled by local buckling and fatigue fracture. For local buckling, the brace's flanges and web were selected so that buckling would not occur under repeated loading. As for fatigue fracture, cumulative ductility experienced by the braces during their lifetime would not exceed the allowable cumulative ductility. Based on the associated fatigue tests [10], the braces adopted in the design would be able to sustain a large earthquake twice and a medium-scale earthquake four times.

(3) Seismic response analysis for confirmation of seismic performance

Outline of time history response analysis

Three synthesized ground motions, all of which fit the spectrum stipulated in the Building Standard Law in Japan, were adopted for the time history analysis. The spectrum was specified at the engineering bedrock located at 50m below the ground level, and the subsurface amplification of the motions were considered. In addition to the three motions, three recorded ground motions, i.e., the 1940 El Centro N-S component, 1952 Taft E-W component, and 1968 Hachinohe E-W component, were adopted. The three ground motions were scaled so that the maximum velocity would reach 0.5m/s. The adopted ground motions are summarized in Table 2, and their displacement response spectra are shown in Fig. 10. Also, the criteria adopted in design are summarized in Table 3.

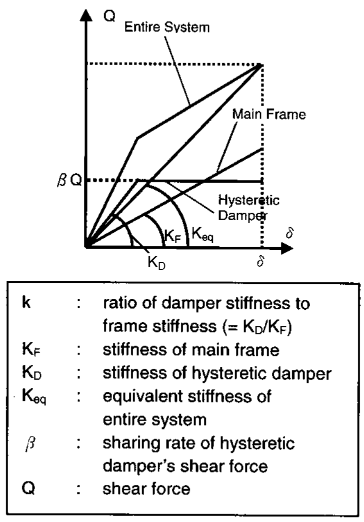


Fig. 7 Typical hysteresis characteristics of building with steel hysteretic dampers

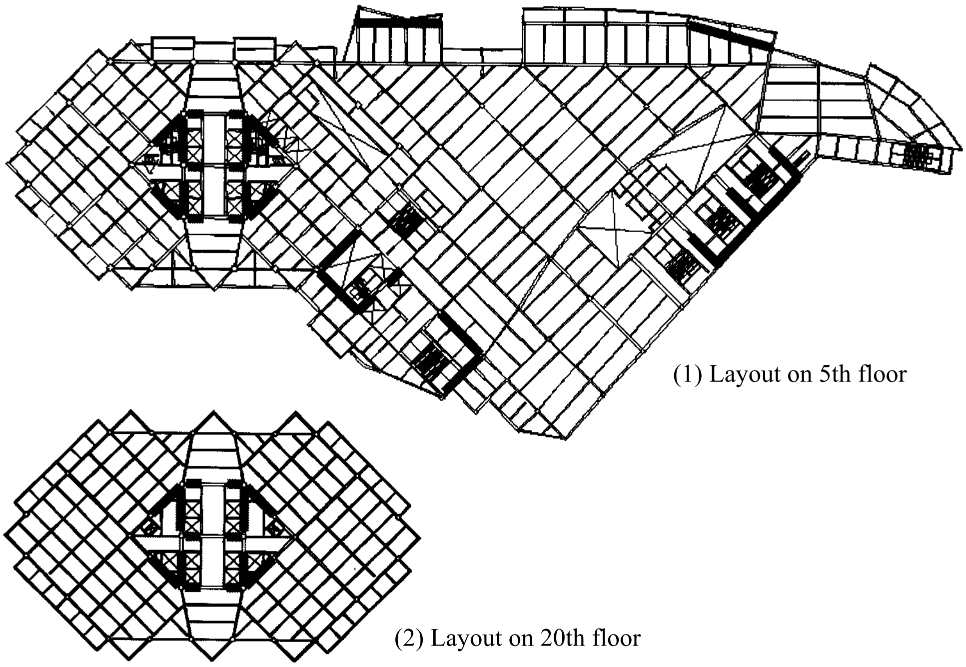


Fig. 8 Layouts of 5th and 20th floors

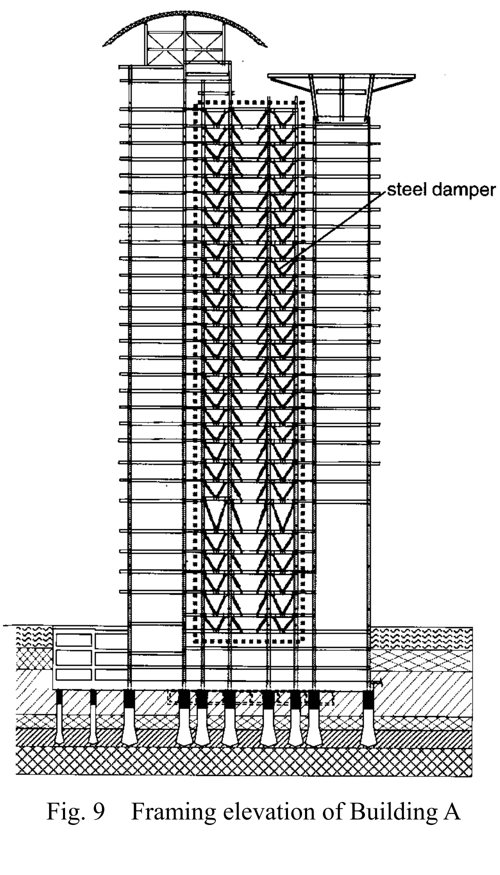


Fig. 9 Framing elevation of Building A

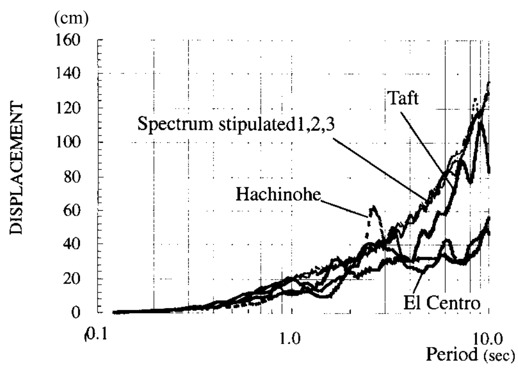


Fig. 10 Displacement response spectra (5% damping)

For the nonlinear time history analysis, a stick model consisting of 39 lumped-masses was used (Fig. 11). In each story, the shear stiffness was taken to be the sum of the stiffness provided by the main frame and the stiffness provided by the buckling-restrained braces. In order to obtain the backbone curve, which in turn was used to specify the story hysteresis, nonlinear pushover analyses were conducted. The backbone curve of the 15th story is shown in Fig. 12. The damping

Table 2 Ground motions for dynamic analyses

Ground motion types	Ground motion	V	A	Duration (s)
Synthesiezed ground motions obtained from code stipulated spectrum	Synthesized ground motion 1	53	356	120.0
	Synthesized ground motion 2	56	334	120.0
	Synthesized ground motion 3	61	366	120.0
Observed earthquake	El Centro 1940 NS	50	511	53.8
	Taft 1952 EW	50	497	54.4
	Hachinohe 1968	50	330	60.0

V: maximum velocity (cm/s) A: maximum acceleration (cm/s²)

Table 3 Criteria for structural design

Superstructure	Story drift	Below 12.0×10^{-3}rad
	Ductility factor of story	Below 2.0
Steel damper	Plastic deformation capacity 1	Below allowable ductility factor of member evaluated based on brace the width-thickness ratio (for a large earthquake)
	Plastic deformation capacity 2	Below allowable cumulative ductility factor evaluated from fatigue tests (for twice a large earthquake and four times a medium-scale earthquake)

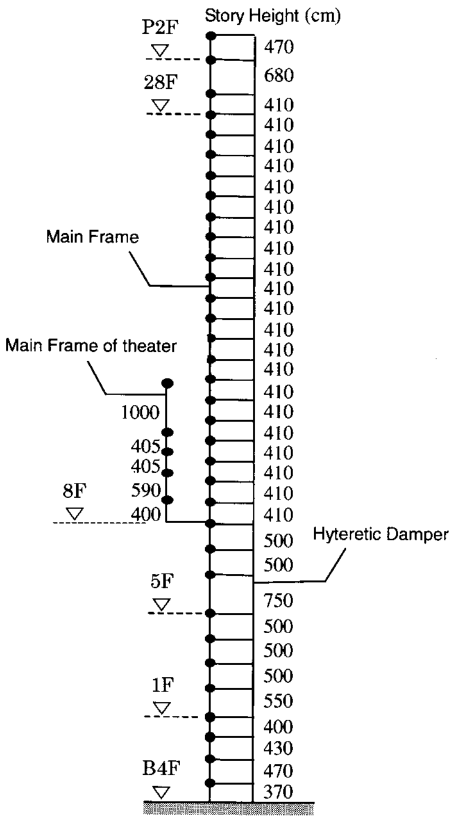


Fig. 11 Analysis model

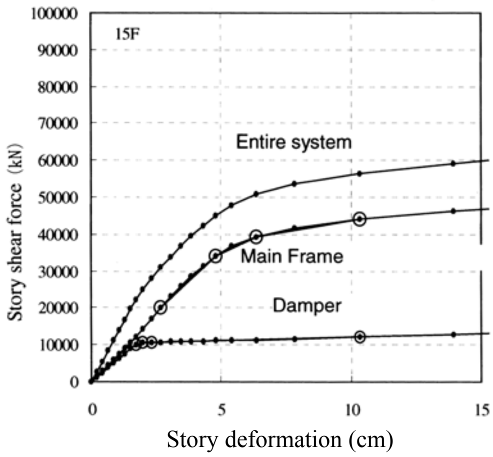


Fig. 12 Backbone curves of 15th story

matrix used in time history analysis was constructed using Biggs-Whittman's method. That is, for each mode, the equivalent damping ratio was evaluated from the strain energy of each structural component (i.e., 2% for the main frame, 0% for the buckling-restraint brace, and 3% for the reinforced concrete structure).

Results of seismic response analysis

Examples of story drift responses and ductility factor responses of the main frame in the

X-direction are shown in Fig. 13. The maximum story drifts in the superstructure are greater than 10×10^{-3} radians in some low stories, but story drifts for the rest of the floors are below 10×10^{-3} radians (1/100), which satisfies the design criterion of 12.0×10^{-3} radians (1/83). The maximum ductility factors of the main frame are about 1.5 in all stories, which is significantly smaller than the design criterion of 2.0. The ductility factor responses and cumulative ductility factor responses of the low-yield portion of the braces are shown in Fig. 14. Both responses also satisfy the design criteria.

Application of Structural Control Systems with Oil Dampers

<u>Outline of building</u>

Introduced next is Building B, which is located at the southwest side of the JR Osaka station, and has four stories underground and 20 stories above ground. This building has multiple functions, i.e., parking and shops in the underground, shops in lower floors above ground, and offices in other floors. The outline of the building is shown in Table 4, and the rendering is shown in Fig. 15. This building is located on the east side of Building A, introduced in the previous section.

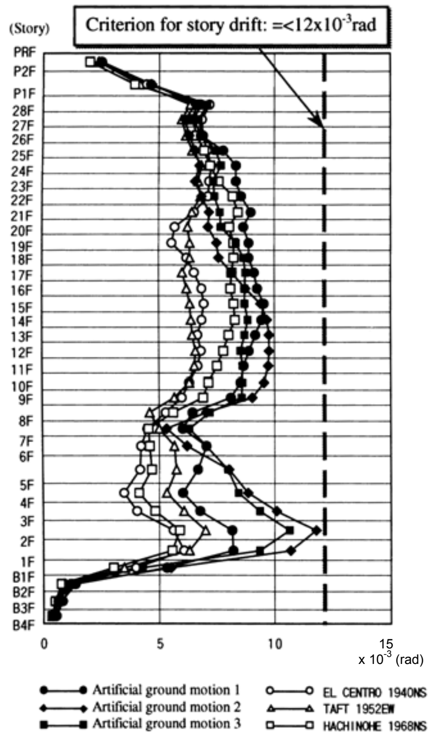
<u>Outline of structural control design</u>

(a) Structural characteristics of frame

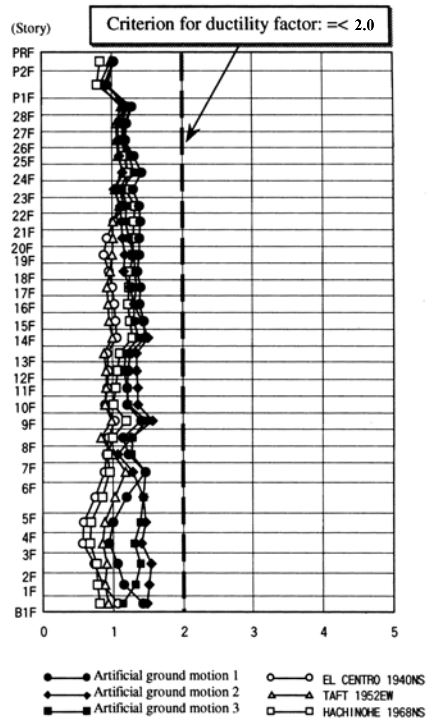
The framing plans and elevation of the building are shown in Figs. 16 to 18. An entrance atrium extending for six stories from the ground is located at the north side of this building. The

Table 4 Building B description

Location	Osaka, Japan
Main use	Office, Shops
Total floor area	44,000 m^2
Building area	2,500 m^2
Height	100 m
Completion	September, 2004
Floors	20 above ground 4 underground
Structure	Columns:Concrete-filled steel tube other members:Steel
Natural periods	X-direction 3.28s Y-direction 3.14s Torsional direction 3.17s

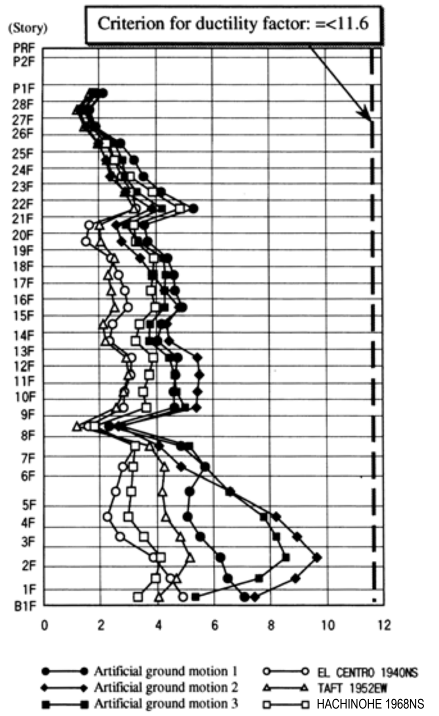


15 (a) Maximum story drift

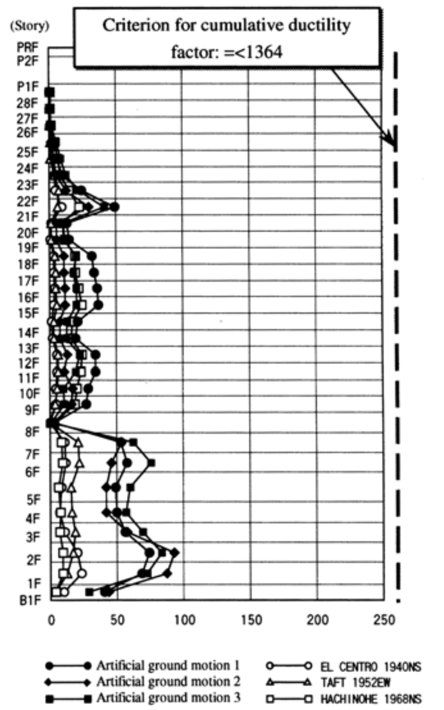


(b) Maximum ductility factor

Fig. 13 Maximum responses of main frame (X-direction)



(a) Maximum ductility factor of core member



(b) Maximum cumulative ductility factor of core member

Fig. 14 Maximum responses of hysteretic damper (X-direction)

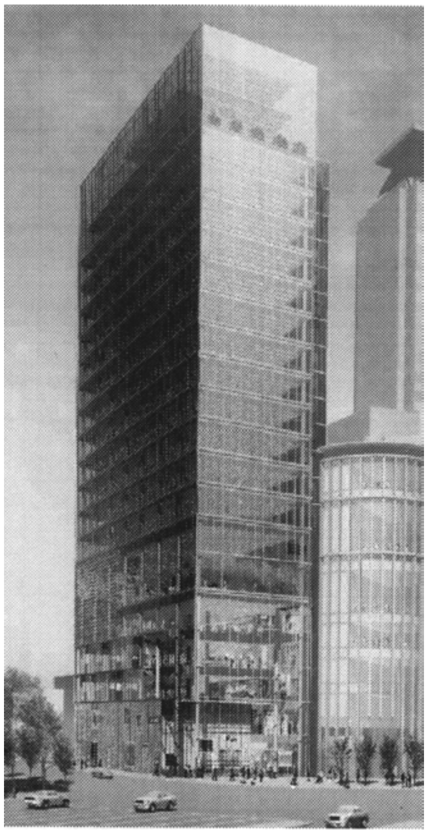


Fig. 15 Rendering of Building B

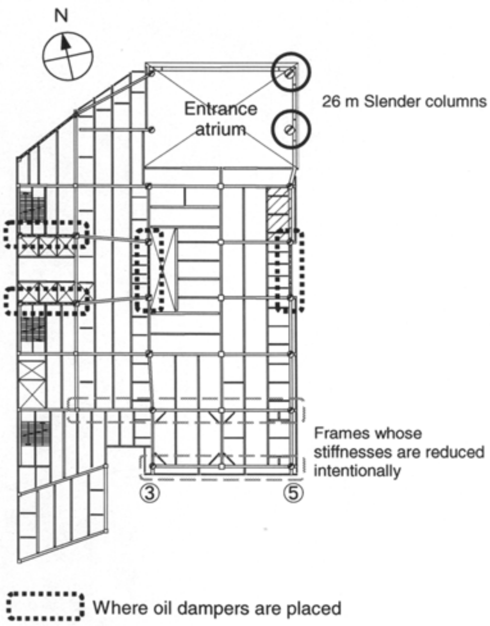


Fig. 16 Framing plan of lower (3rd) floor

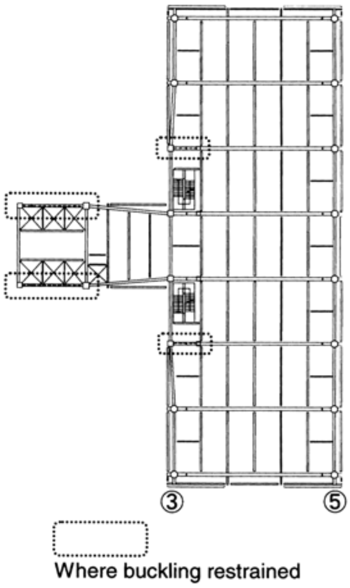


Fig. 17 Framing plan of typical floors

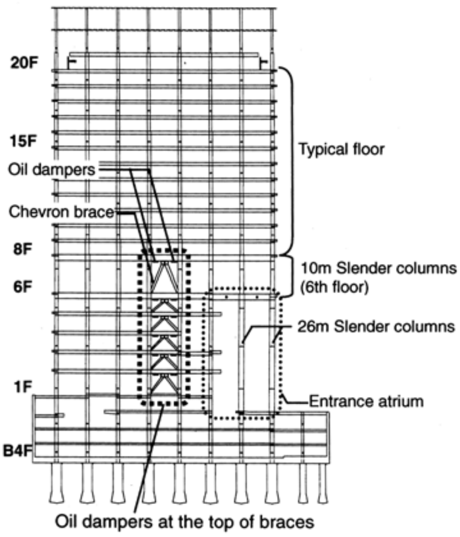


Fig. 18 Building elevation of Building B

atrium made the seismic design of this building unique. The atrium consists of two slender columns, which are 26m long. Furthermore, since the sixth story has twice the height of other floors, all columns in this story are 10m long. In order to prevent eccentricity caused by the biased layout of the atrium, the stiffness of the frames located at the opposite side of the atrium was reduced intentionally. Therefore, the lateral stiffnesses of the lower six stories were designed to be significantly lower than the lateral stiffnesses in upper stories. This significant difference in structural characteristics between the lower and other stories can be amended by the introduction of structural control devices in the lower stories.

In this building, oil dampers were adopted to control the stiffness and dissipate energy for the lower six stories. The dampers were installed horizontally at the top of chevron braces. Four oil dampers were placed in each direction at each story. Hence, a total of forty-eight dampers were installed in the building. An example of the oil dampers attached at the top of chevron braces is shown in Fig. 19. The force-velocity relationships of the oil dampers are shown in Fig. 20. The oil dampers during manufacture are shown in Fig. 21(a), and installation of an oil damper is shown in Fig. 21(b).

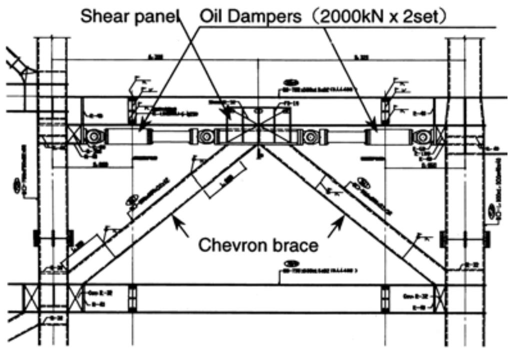


Fig. 19 Oil dampers installed in frame

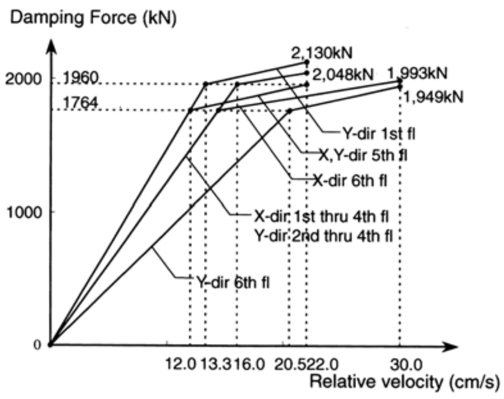


Fig. 20 Force-velocity relationships of oil dampers

Fig. 21(a) Oil dampers in manufacture

Fig. 21(b) Installed oil damper

(b) Characteristics of oil dampers

Energy dissipation devices are classified into two categories: displacement-dependent and velocity-dependent. Which one of the two types should be adopted depends on the functions expected in the devices. The most common displacement-dependent damping devices are metallic-yielding dampers such as the buckling-restrained braces, and the most common velocity-dependent damping devices are viscous fluid dampers. The metallic-yielding damper is generally less expensive when compared with other dampers of the same strength capacity, and it is the most common choice. Nevertheless, we chose oil dampers in this particular building project for the following reasons:

(1) Unlike the metallic damper, the oil damper does not result in residual permanent displacement after the damper is engaged. Permanent displacement is prohibitive in the flexible structural system adopted in the lower six stories of this building.

(2) The building has very few spaces in which to install dampers. Sufficient damping would not have been provided if metallic dampers had been adopted.

(3) The oil damper's damping force is proportional to the relative velocity but not dependent on the relative displacement. If the metallic device were used in this building, the initial elastic stiffness would become too high, so that the natural period of the structure would be less than 3 seconds, which in turn would increase the force induced by the earthquakes. On the other hand, the initial stiffness, and thus the natural period of the structure would barely be affected when combined with oil dampers.

<u>*Time history analysis to confirm seismic performance*</u>

(a) Outline of analysis

(1) Ground motions, design criteria, and analysis model

The three synthesized and three recorded ground motions used for the earthquake response analysis of the building described in the previous section (Building A) were also used for the analysis of this building. The design criteria adopted for this building are also the same as those employed for the previous building, but criteria for the oil dampers were added. For the nonlinear time history analysis, a stick model consisting of 24 lumped masses and springs that represent the story shear stiffnesses was used as is shown in Fig. 22. The damping matrix of the structure was constructed in the same manner as in the previous building, with the damping ratio of the first mode set at 2% with respect to the main frame. In addition, an individual damping coefficient, C_D, was assigned to represent each oil damper.

(2) Notes on structural design using oil dampers

The oil damper with the chevron brace was modeled as the classical Maxwell model. As stiffness K_D in the Maxwell model increases, the

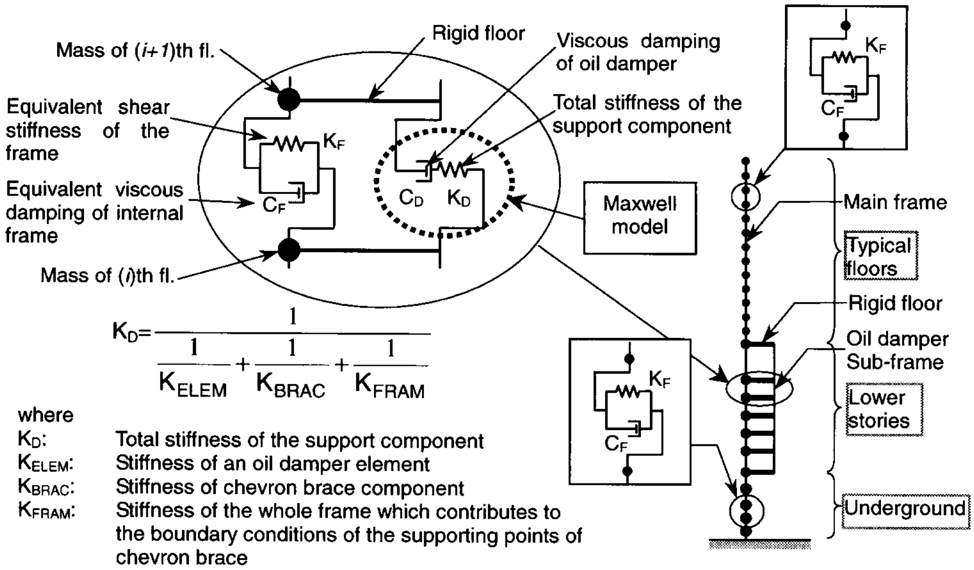


Fig. 22 Analysis model

damper functions more effectively. The value of K_D, however, is limited because of the flexibility of other elements. K_D is determined by the formula shown in Fig. 22, given as the reciprocal of the sum of reciprocals of K_{ELEM}, K_{BRAC} and K_{FRAM}. Here, K_{ELEM} is the inherent stiffness of the oil damper, K_{BRAC} is the stiffness of the chevron brace, and K_{FRAM} is the boundary stiffness provided to the damping system by the frame itself. From this equation, we find that setting a large value in one or two of the three stiffness components does not ensure a larger K_D. For example, suppose a very stiff chevron brace and oil damper. If the frame's columns located on both sides of the chevron brace are not axially rigid (small K_{FRAM}), the rigid rotation of the chevron brace together with the damper will reduce the overall K_D of the damper, brace, and frame system, resulting in an overall reduction in the system's effective damping. In this building, large column axial stiffness was achieved by the adoption of concrete-filled tubular columns with a steel thickness of 60mm.

When K_D in the Maxwell model is determined, a maximum value of C_D corresponding to the value of K_D can also be estimated. C_D is dependent on K_D because if C_D is excessively large, most of the displacement will be concentrated in the spring element with the smallest stiffness, whilst movement in the dashpot becomes very small. Therefore, the oil damper design must focus on first determining K_D, then selecting C_D such that a large amount of displacement occurs in the oil damper, and finally, determining the relief load Q of the dashpot (i.e., the maximum force that can be sustained by the dashpot), so that it is able to resist forces dependent on the selected C_D and K_D values.

(3) Variation in properties of oil dampers

The performance of actual oil dampers varies by a certain degree from the originally specified values. The range of performance given by the maximum and minimum values adopted in the design is shown in Table 5. All the dampers to be used in the building were inspected to find the actual properties before installation. Analyses that considered the variation were also conducted, and the adequacy of the design was confirmed. In reference to the inspected properties, individual dampers were carefully chosen for installation at respective locations so as to minimize the overall eccentricity.

Table 5 Possible variablility of damping coefficient

Contributing factor	Range of variety	
	Minimum	Maximum
Error in product	−10%	+10%
Variation by atmospheric temperature 0 to 30 degrees Celsius	−2%	+0%
Difference in direction (compression and tension)	0% (Combination of direction)	
total possible error	– 12% (case 1)	+ 10% (case 2)

The results of real eigenvalue analysis and complex eigenvalue analysis, both in the X direction, are shown in Table 6(a) and Table 6(b), respectively. In reference to the difference between the real and complex damping ratios, the oil dampers contributed to about 7% additional damping in the first mode. An example of the response analysis results is shown in Fig. 23.

Table 6(a) Real eigenvalue analysis (X-direction)

Mode	Eigenvalue	Radians	Cycles	Damping coefficient
1	3.66E + 00	1.91E + 00	3.05E – 01	2.00E – 02
2	2.28E + 01	4.78E + 00	7.60E – 01	4.99E – 02
3	6.32E + 01	7.95E + 00	1.27E + 00	8.31E – 02

Table 6(b) Complex eigenvalue analysis (X-direction)

Mode	Eigenvalue (real)	Eigenvalue (imaginary)	Cycles	Damping coefficient
1	– 8.67E – 02	– 1.92E + 00	3.05E – 01	9.05E – 02
2	– 3.80E – 01	– 4.77E + 00	7.60E – 01	1.59E – 01
3	– 7.18E – 01	– 8.15E + 00	1.30E + 00	1.76E – 01

STRUCTURAL CONTROL SYSTEMS FOR STRONG WINDS

Strong winds cause vibrations in high-rise buildings. These vibrations are irrelevant to structural safety, but make the residents in the building uncomfortable. To reduce such vibrations, structural control is adopted on some occasions. In ordinary cases, displacements induced by strong winds are significantly smaller than those induced by large earthquakes. Thus, a control device for reducing wind effects must be effective for small displacements, and at the same time should be durable and safe enough to withstand large displacements during large earthquakes. This requirement complicates the design of such a device. One of the most effective devices for this purpose is a tuned mass damper (TMD) system. This system counteracts the vibrations induced in the building by the wind

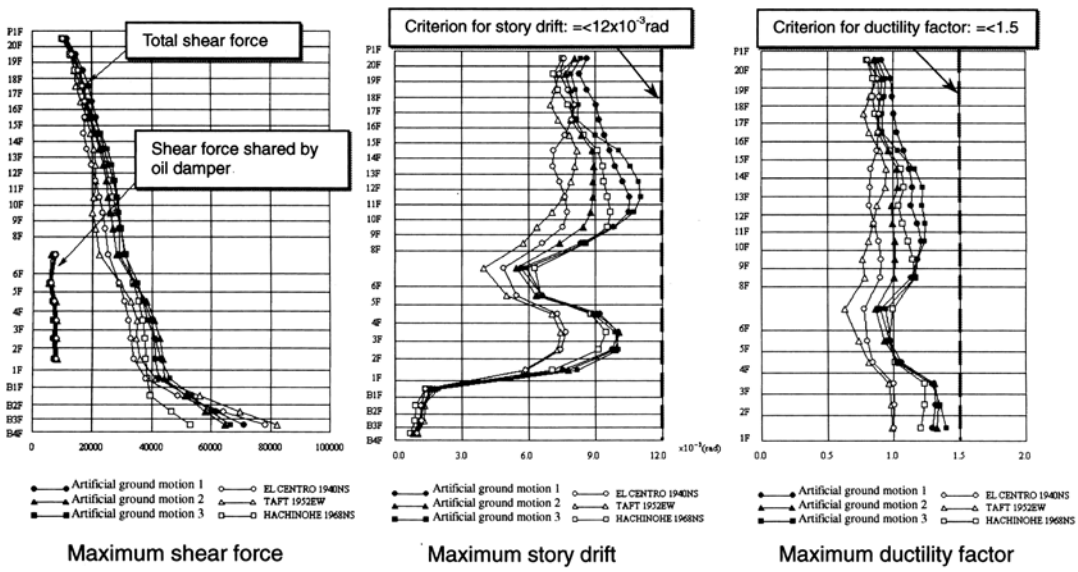


Fig. 23 Maximum floor responses (X-direction)

with an out-of-phase vibration of the TMD at the building's natural frequency. Other systems, such as hysteretic or viscous fluid dampers, commonly arranged to absorb energy in the relative story drift, are regarded as less effective against strong winds. This paper introduces a type of damper that is effective for control of vibrations with small deformations.

Application of Structural Control System with TMDs

Applications of TMD systems to high-rise buildings began with an office building named "Crystal Tower" in Osaka, which was completed in 1990 [11]. The exact number of existing buildings equipped with TMDs is not reported, but the firm with which the writers work has designed and constructed seven high-rise buildings with TMDs, including "Crystal Tower." The latest application among the seven is shown below.

Outline of building

A new type of tuned mass damper was developed and applied in an office building in Osaka (Building C). As shown by the vertical section of the building in Figure 24 this building is seismically isolated below the third floor level. The isolating system of this building uses newly developed mechanical bearings called a linear slider, each of which supports 30,000kN of the building weight. Basic properties of the building [12] are summarized in Table 7.

Such seismic isolation is very effective for earthquakes, but not necessarily beneficial for winds. Especially for high-rise buildings, the application of a seismic isolation system sometimes causes adverse effects under wind loading. This building is designed so that the friction of the linear sliders prevents the isolation floor from moving and disturbing residential comfort during winds. Furthermore, when wind load is large enough to move the isolation floor, a locking system consisting of multifunction oil dampers installed in the isolation floor would automatically lock the isolation floor. Hence, this building is essentially identical to conventional (non-isolated) high-rise buildings during strong winds.

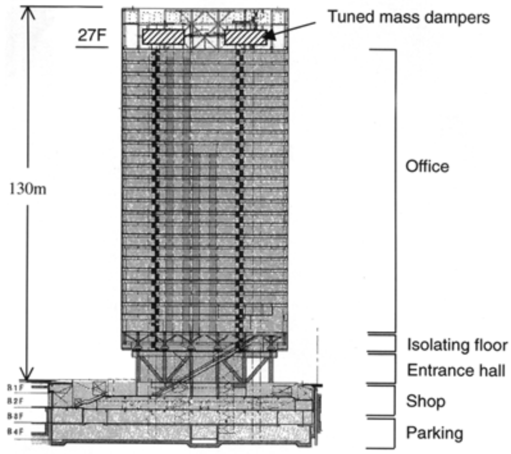


Fig. 24 Vertical section of Building C

Table 7 Building C Description

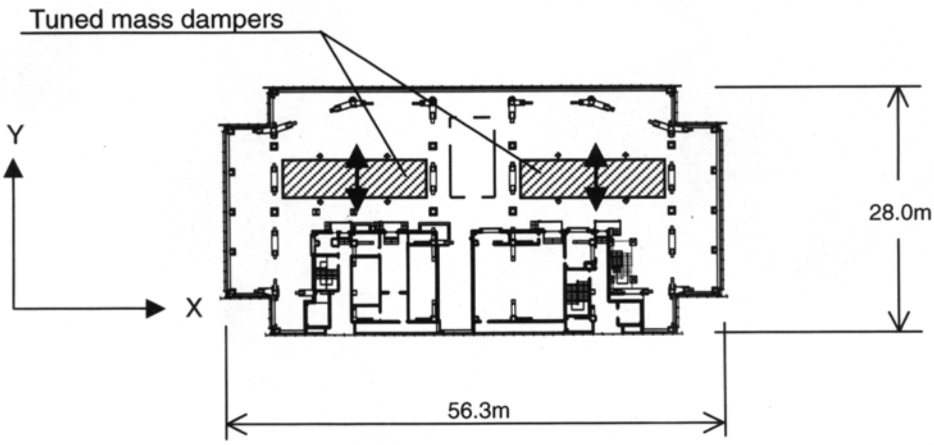
Location	Osaka, Japan	
Main use	Office	
Total floor area	47,613.2 m^2	
Building area	1,613.8 m^2	
Height	130 m	
Completion	December, 2002	
Floors	27 above ground 4 underground	
Structure	Steel	
Natural periods (measured based micro-tremor)	X-direction Y-direction Torsional direction	3.45s 2.83s 2.96s

Outline of structural control design

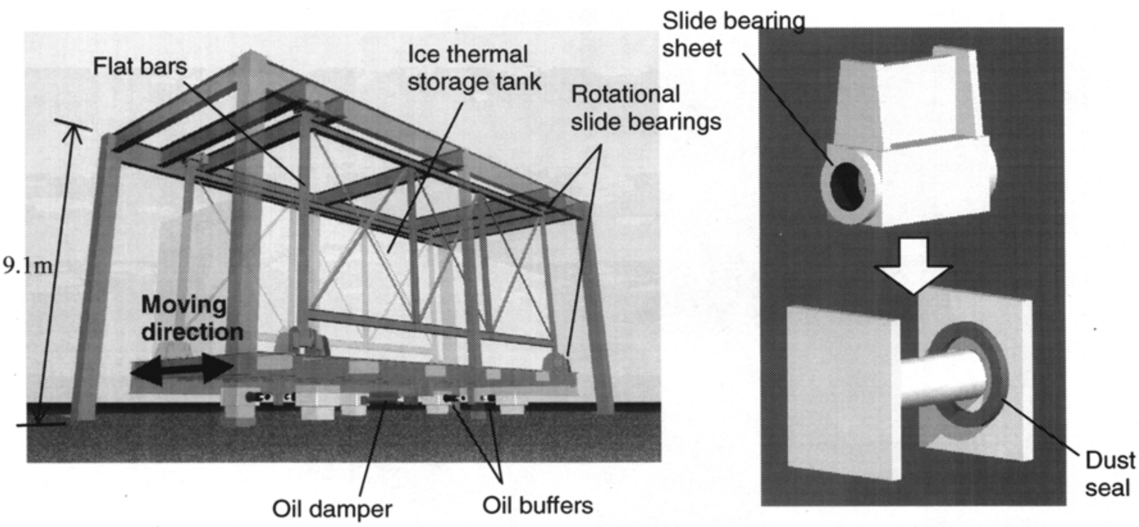
Wind tunnel tests were conducted, and the results indicated that the residential comfort would not be ensured in the Y and torsional directions. Therefore, two TMDs were developed and installed in the roof to control vibrations in these directions. Each of the two TMDs moves in one direction as shown in Fig. 25. The main features of this system are described below.

(1) Use of ice thermal storage tank as a moving mass

Design of the moving mass is important in the aspects of both cost and space. In the adopted TMD system, an ice thermal storage tank used for air conditioning is used as a moving mass. This technique was applied in a few design cases [11,13], because it ensures the use of a relatively large



(a) Layout of the TMDs



(b) Setup of TMD (c) Rotational slide bearing

Fig. 25 Layout and setup of TMD and slide bearing

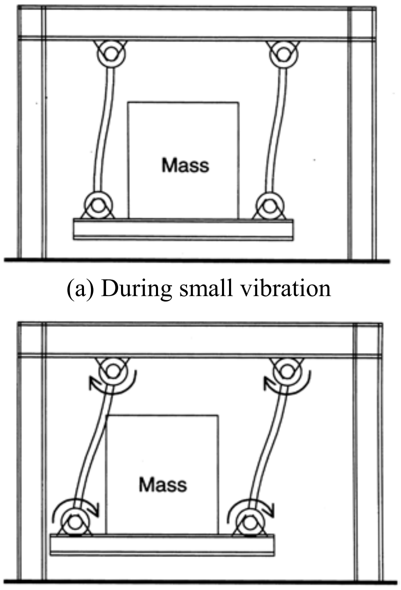
mass, which is the key for good control. The moving mass of this TMD weighs 2.7MN. The mass ratio, defined as the ratio of moving mass to the effective first modal mass of the building, is 4.2% in the Y direction and 2.2% in the torsional direction.

(2) Flat bars suspension system

In many cases, a TMD is supported by mechanical bearings. These bearings have friction, which often harms the control performance of the TMD. In the worst case, the friction disables the TMD completely. In this TMD system, the moving mass is suspended by flat steel bars, which also provide resistance against the movement in the horizontal direction. This suspension system enables movement of the mass from elastic deformation of the bars, and consequently the system is frictionless (Fig. 26). In order to restrain vibrations in the direction orthogonal to the moving direction, braces are used instead of bearings.

(3) Rotational slide bearing

The flat bars are connected to the frame and moving mass with a rotational slide bearing.



(a) During small vibration

(b) During large vibration

Fig. 26 A view of TMD motion

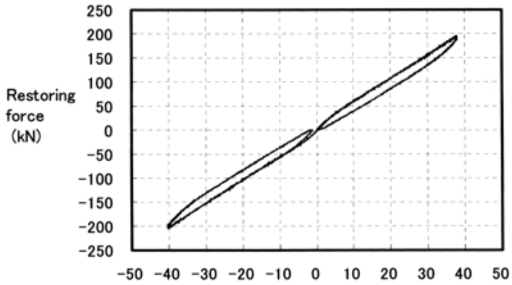


Fig. 27 Force-displacement loops of slide bearing

This bearing is a mechanical hinge consisting of a cylinder and its housing, which is covered with slide bearing sheets reinforced by wire mesh (Fig. 25(b)). This slide bearing is able to regulate the maximum bending moment induced into the flat bars, thereby ensuring the safety of the flat bars.

(4) Oil damper having damping force proportional to the square of velocity

Usual TMD systems use oil dampers whose damping force is proportional to the stroke velocity. The TMDs developed for this building use oil dampers whose damping force is proportional to the square of the stroke velocity. Achieving this relationship is simple, because oil dampers originally exhibited this behavior. With this relationship, during large response amplitudes, the damping force rapidly increases in the system. This behavior is beneficial for preventing excessive stroke displacements.

On-site confirmation of dynamic characteristics of TMD

(1) Static loading test

Static loading tests were conducted on the site by pushing the moving mass with oil jacks, and the corresponding force-displacement relationships were obtained (Fig. 27). The initial stiffness, second stiffness and friction of the TMD are estimated as 7.3kN/cm, 4.75kN/cm and 9.5kN, respectively. The friction coefficient estimated for the overall TMD system is 3.6/1000.

(2) Free vibration test

After initially displacing the TMD mass, the mass was released and allowed to oscillate freely, which caused the building to sway. The results of this test are shown in Fig. 28. In this test, the moving mass of the TMD (designated as No. 1) was pushed to activate the vibration. It was observed that the other TMD (designated as No. 2) also responded smoothly for small building accelerations less than $1 cm/s^2$.

Control performance

Based on the dynamic characteristics identified by on-site tests, response analyses in strong wind conditions were conducted. The results are plotted against the habitability performance index curves for horizontal vibration [15] as shown in Fig. 29. It is demonstrated that the habitability performance is improved greatly and that the installation of the TMDs both in the Y and torsional directions satisfies the design criteria.

Structural Control System with High-Damping-Rubber Dampers

Outline of structural control system

The building described in the previous section (Building A) has another control system that aims at controlling wind-induced vibrations. Figure 30 shows the standard floor plan of the tower portion of this building. Stiff elements and structural control braces, arranged to provide lateral stiffness, are concentrated in the core of the

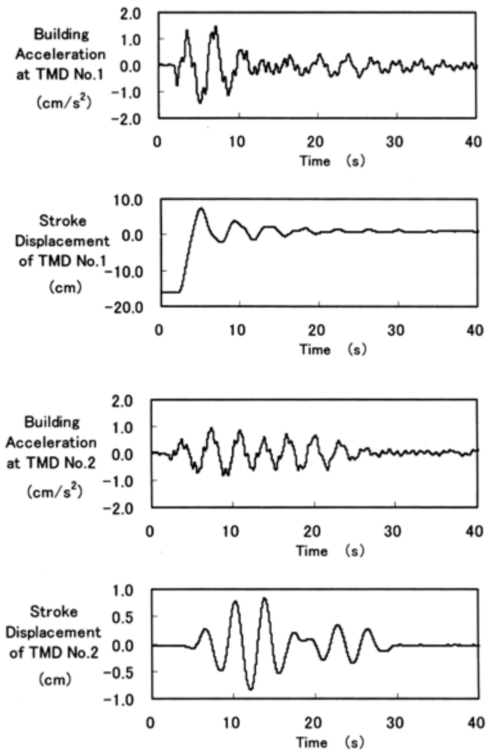


Fig. 28 Responses obtained from free vibration test

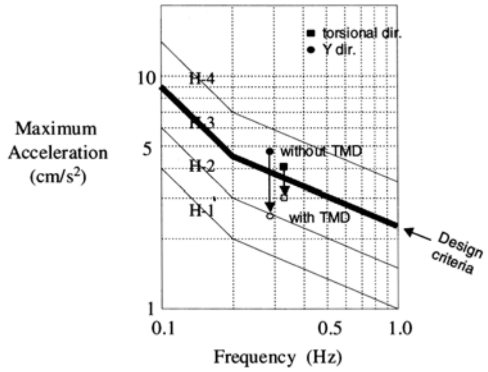


Fig. 29 Curves of AI evaluation of habitability performance

building. This configuration allows earthquakes and strong winds to shake the building easily in the torsional direction rather than in horizontal directions. To ensure comfort against torsional vibrations, the following structural control system was adopted.

Torsional vibrations produce large story drifts at places located away from the center of the building. To design an effective control system, dampers should be installed in zones around the

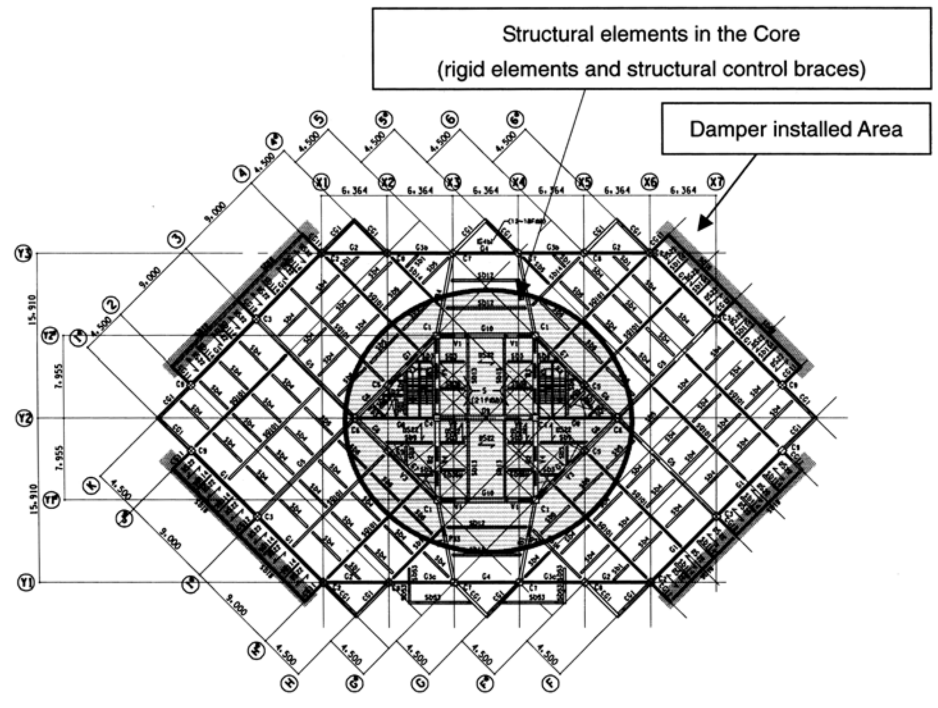


Fig. 30 Standard floor plan at high-rise part of Building A

building circumference. In this building, precast (PCa) concrete boards were used for exterior walls, and dampers were arranged in those boards. Dampers were designed to be installed in a symmetrical pattern on the 16 stories of the tower section from the twelfth floor to the 27th floor. A total of 256 dampers were installed on the PCa boards. Generally, TMDs or AMDs (active mass dampers) would have been adopted to control vibrations. However, to be effective against torsional vibrations, at least two such devices would have needed to be installed at the roof. The roof of this building has an unsymmetrical plan, which made the application of such devices unfeasible. For this reason, the control system with PCa boards was adopted in this building.

Design consideration

The design concept of this structural control system is as follows:

(1) For strong winds that are expected once a year, the habitability criteria specified for offices in the Evaluation Guidelines of the Architectural Institute of Japan (AIJ) [13] should be satisfied.
(2) For large earthquakes that are expected about once every 500 years, the exterior walls should not fall off and structural control devices should not be damaged. The velocity of strong winds expected once a year are estimated statistically based on the design guidelines of the AIJ. The maximum wind speed (given as the average over ten minutes) at the top of the building is calculated as 20.0m/s by the following equation:

$U_{H=139.8} = V_1 \times (139.8/53)^{0.27} = 20.0\text{m/s}$

U_H: The wind speed for examination of habitability (m/s)

V_1: The wind speed (= 15.4m/s), expected once a year in the Osaka observatory (for the measurement height of 53.0m)

Classification of habitability performance is listed in Table 8. Figure 31 is used to judge the vibration levels, classified as H-1, H-2, H-3 and H-4. The target level of vibrations was chosen as H-3, because the tower part of this building is used as offices for which the target rank is “Rank II” in most cases.

Table 8 Classification of habitability performance

Rank / Use	I	II	III
Residence	H-1	H-2	H-3
Office	H-2	H-3	H-4

‘Rank’ indicates level of comfortableness; Generally ‘Rank-II’ is adopted

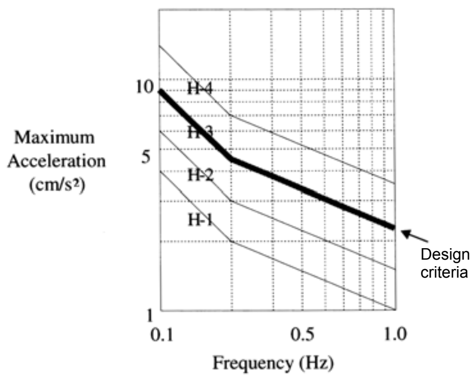


Fig. 31 Evaluation of habitability performance

Outline of structural control device

The outline of the structural control system is shown in Fig. 32. Two high damping rubber dampers are used to support an exterior wall made of a PCa board. When subjected to an interstory drift, this configuration allows the board to rotate around the virtual center, located under the boards. In this condition, dampers sustain shear deformations, which in turn produce a damping force. The system consists of the following elements:

(1) PCa board that forms the exterior wall and is supported on the steel beams.
(2) Steel frames that support the circumference area of the building. During strong winds and earthquakes, inter-story drifts occur in the frames.
(3) Dampers made of high damping rubber with two end steel plates, which absorb vibration energy.

Effect of structural control

The effect of structural control is estimated as follows:

(1) The torsional damping coefficient C provided by the structural control devices installed in each story is computed in reference to the

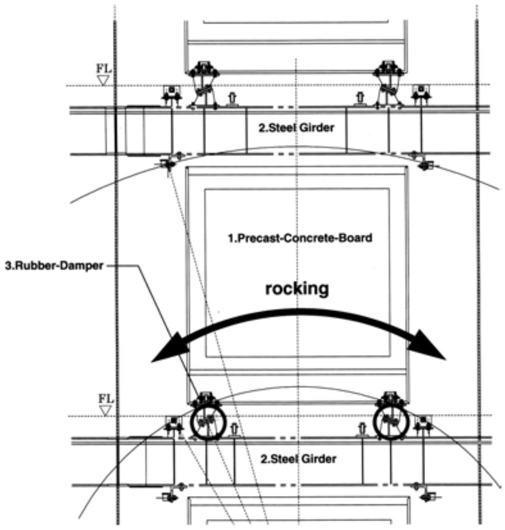


Fig. 32 Outline of structural control device using high-damping rubber

first torsional modal shape. The contribution to the equivalent damping coefficient for the entire structure is estimated.

(2) The contribution of the structural control devices to the equivalent damping ratio h is computed using the equivalent damping coefficient computed in 1), the torsional inertia moment of the entire building, and the first natural period of the building.

(3) Considering 1% damping as the structural damping of the building itself, the rate of acceleration reduction that can be expected from the equivalent damping ratio h is estimated by the following equation:

$$1/\sqrt{1+h} \times 100\ (\%) \quad (1)$$

The effects of the dampers calculated by Eq. (1) are shown in Fig. 33, which indicates that the target “H-3” habitability criteria are satisfied for torsional vibrations.

Behavior of structural control device against large earthquakes

The specifications of the rubber and the installation angle are set so that excessive deformations and damage should not occur in the exterior walls and dampers even when a large earthquake occurs. It is expected that the

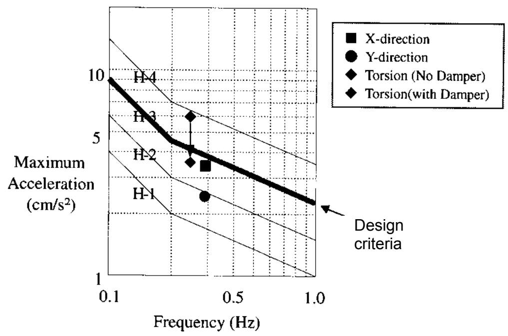


Fig. 33 Effect of structural control on reduction of vibration

dampers would not have to be replaced after such events, because the functionality of structural control shall be maintained. Note that the damping performance is expected to be somewhat reduced immediately after the earthquake but to recover in about three months.

Maintenance of structural control device

Any maintenance work, including regular inspection, partial exchange, or inspection after a major earthquake is deemed unnecessary.

CONCLUSIONS

Two applications of structural control that aim at improving the seismic performance of buildings and two more applications that aim at improving habitants’ comfort during strong winds are introduced in this paper. Buckling-restrained braces, one of the most popular types of dampers in Japan, and oil dampers, also popular among velocity-proportional dampers, were applied to control earthquake motions. Reasons for selecting these dampers among various alternatives were also given in reference to the functions, constraints, and other design factors of the designed buildings. Performance criteria adopted for the designs were noted, and verification of the criteria using nonlinear dynamic response analyses was described. Control against earthquakes and control against strong winds are not necessarily complementary. In some cases, design attempts to control one hazard may in fact have a contradictory effect on the other. Issues to be considered for control against strong winds are identified, and two applications, one with oil dampers and the other with rubber dampers, are introduced. In both cases, use of other building components (an ice thermal storage tank for the moving mass and supports of precast boards for rubber dampers) contributed significantly to economizing cost and space. It is a recent trend in Japan to design high-rise buildings with structural control systems like those presented in this paper. This trend results from building owners' gradual recognition of the benefits of these systems for enhanced seismic safety and comfort.

REFERENCES

[1] *Proceedings of the Second Japan National Symposium on Structural Control* (2000).

[2] *Proceedings of 2002 Passive Control Symposium* (2002), Structural Engineering Research Center, Tokyo Institute of Technology.

[3] *Proceedings of the Third World Conference on Structural Control* (2002), International Association for Structural Control, Como, Italy.

[4] Yamamoto, M. and Suzuki, Y. (2001). "Practical applications of structural control systems to building in Japan," *Earthquake Engineering and Engineering Vibration* (China), Vol. 21, No. 4, pp. 32–39.

[5] Kanzaka, Y., Hashimoto, Y., Akiyama, T., Yamanaka, M., Sano, T. and Yagi, S. (2002). "Structural design of high rise building with parallel-type damper which combines a hydraulic damper and a hysteresis damper," *Proceedings of 2002 Passive Control Symposium*, pp. 89–96, in Japanese.

[6] Narihara, H., Maseki, R., Kimura, Y. and Isshiki, Y. (2002). "Hybrid energy dissipating bracing system combined with elasto-plastic damper and visco-elastic damper," *Proceedings of 2002 Passive Control Symposium*, pp. 97−105, in Japanese.

[7] Ohbuchi, T., Mori, H., Yoshie, K., Hara, H., Arima, F., Takeuchi, Y., Saito, Y., Ishi, M. and Kasai, K. (2002). "Proposed theme structure and analytical modeling study for passive control system," *Proceedings of 2002 Passive Control Symposium*, pp. 195–219, in Japanese.

[8] Kasai, K., Kibayashi, M., *et. al*. (2002). "Current status of design and construction manual for passive controlled buildings," *Proceedings of 2002 Passive Control Symposium*, pp. 195–219, in Japanese.

[9] Inoue, K. (1995). "Seismic design of the building installed hysteretic damper," *Proceedings of One New Direction of Seismic Design Symposium*, Architectural Institute of Japan, pp. 95−109, in Japanese.

[10] Yoshinobu, O., Hirofumi, K., Naomiki, S. (2000). "Fatigue characteristic on H-shaped brace damper constrained flexural buckling," *Proceedings of the Second Japan National Symposium on Structural Control*, pp. 531−536, Kyoto, Japan, in Japanese.

[11] Nagase, T., *et. al*. (1992). "Tuned pendulum mass damper installed in Crystal Tower," *The Structural Design of Tall Building*, Vol. 1, pp. 35–56.

[12] Tanaka, T., Tsubaki, H. and Yamane, K. (2002). "Umeda DT Tower," *Menshin*, Japan Society of Seismic Isolation, No. 37, pp. 3–8, in Japanese.

[13] Yamamoto, M., Aizawa, S., Higashino, M. and Toyama, K. (2001), "Practical application of active mass dampers with hydraulic actuator," *Earthquake Engineering and Structural Dynamics*, Vol. 30, pp. 1697−1717.

[14] Yamamoto, M., Higashino, M. and Aizawa, S. (2002), "Development of semi-active mass damper system using oil damper with solenoid valves," *Proceedings of the Third World Conference on Structural Control*, Vol. 2, pp. 343–348.

[15] Goto, T., *et al*. (1991), "Guidelines for the evaluation of habitability to building vibration," Architectural Institute of Japan, in Japanese.