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A Study for Damping Application to Response-controlled Structure

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Abstract

Most of high-rise buildings in Japan[*1] are structure with damping systems recently. The design procedure is performance-based design (PBD), which is based on the nonlinear response history procedure (NRHP) using 2 or 3-dimentional frame model. In addition, hysteretic property of steel plates or velocity-dependent property of viscous dampers are common practice for the damping system. However, for the selection of damping system, the easy dynamic analysis of recent date may lead the most of engineers to focus attention on the maximum response only without thinking how it shakes. By nature, the seismic design shall be to figure out the action of inertia forces by complex & dynamic loads including periodic and pulse-like characteristics, what we call seismic ground motion. And it shall be done under the dynamic condition. On the contrary, we engineers engineers have constructed the easy-to-use static loads and devoted ourselves to handle them. The structures with damping system shall be designed considering how the stiffness & damping to be applied to the structures against the inertia forces with the viewpoint of dynamic aspect. In this paper we reconsider the role of damping in vibration and give much thought to the basic of shake with damping from a standpoint of structural design. Then, we present some design examples based on them.

Keywords: Response-controlled structure, Damping application, Viscous damper, Hysteretic damper, Design example

1. Objective of the Structure with Damping System

Compared to the conventional aseismic structure, the structure with damping system has a variety of performance objectives as shown below.

Structural engineer needs to propose these objectives and criteria with consideration of the space & economic aspect and select the damping system in answer to them.

Objective-1; Damage control for selected earthquakes (for example, application of BRB)

Design the structure to absorb the seismic energy exclusively by specific elements or equipment which are easy to be replaced for the prospected earthquakes by structural engineer.

Objective-2; Shake (Acceleration) Reduction for earthquake

Try to reduce the earthquake input energy into the structure.

Control the inertia forces which may cause the damage to nonstructural components.

Relieve the occupants' fear in earthquake.

Objective-3; Shake (Deformation, Story drift) Reduction for earthquake

Evaluate the structural damage.

Evaluate the deflection which acts on nonstructural components forcibly.

Objective-4; Shake Reduction after principal shock

Assess the occupants' anxieties during a long-period earthquake.

Control the damage of nonstructural components for a long-period earthquake.

2. Prehension of Damping Character: Vibration of Single-degree-of-freedom (SDF) System with Damping Proportional to Stiffness

First of all, take a SDF system with damping proportional to stiffness for a study of response-controlled structure.

2.1. Response to Sinusoidal Waves

The curve as shown in Figure 1 is presented in almost everyone's text book of vibration theory, which is the acceleration (displacement as well) response amplification ratio of SDF system for Sinusoidal waves. This curve is a theoretical result of harmonic oscillation for sinusoidal input and means following things,

Case-1) If the fundamental period of building is similar to the predominant period of ground motion, the response acceleration and displacement are amplified by resonance. Damping is very efficient for response reduction per resonance.

Case-2) If the fundamental period of building is enough long compared to the predominant period of ground motion, the response acceleration becomes bigger as the damping increases. The response displacement will not

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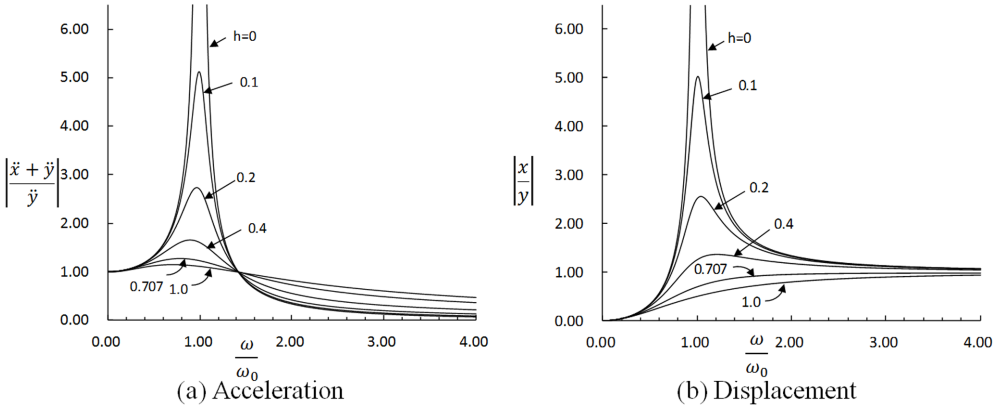


Figure 1. Response Amplification Ratio.

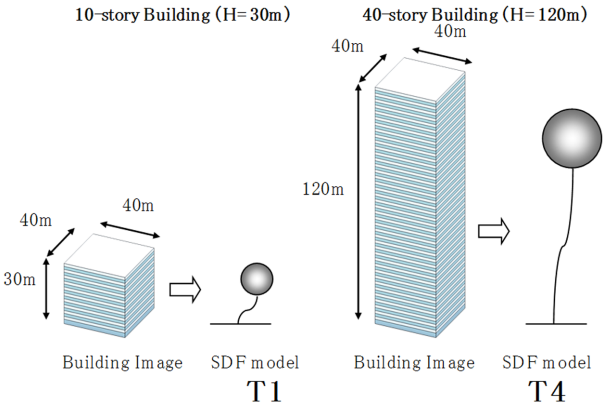


Figure 2. Models for Study.

increase but the reduction effect is small compared to Case-1.

For example, take the ground motion whose predominant period is T = 1.0 sec and add damping to a high-rise building with the fundamental period of 4 sec. The absolute response acceleration increases as the damping increases

Table 1. Property of Study Model

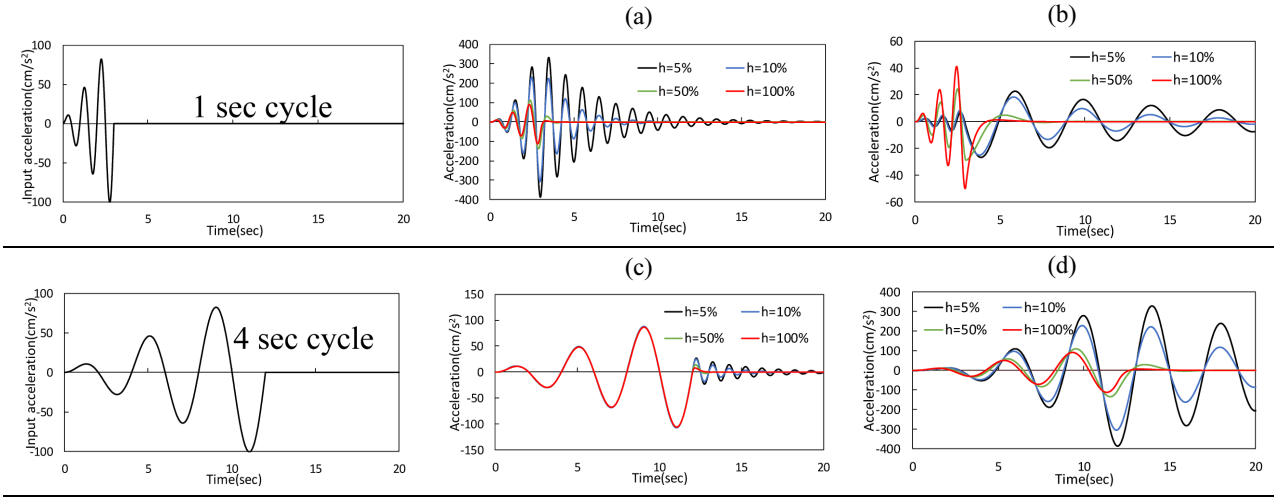
Item	T1	T4
Effective Mass; Me (t)	11,200	44,800
Effective Stiffness; Ke (kN/cm)	4,422	1,105
Fundamental Period; T (sec)	1.0	4.0

despite the intention of acceleration reduction. In terms of deformation reduction, the displacement will not increase but the efficacy is small.

The response of a SDF system to seismic ground motion can be treated as the summation of the harmonic oscillation response for sinusoidal input and the impact transient response for each pulse-like input. For simple explanation, take two simple buildings as shown Figure 2 for the models of SDF system, one is the fundamental period = 1 sec (named T1) and the other is the fundamental period = 4 sec (named T4). Response time history of acceleration and displacement are compared, which are calculated for three cycles of sinusoidal input as seismic ground motion. These stiffnesses are set as elastic and

Table 2. Response Acceleration for Sinusoidal Waves

Input wave	T1	T4
1 sec cycle	(a)	(b)
4 sec cycle	(c)	(d)



these damping are set as proportional to stiffness.

1) The figure in Cell (a) of Table 2 shows the response of T1 for 1 sec ground motion. The building shakes with 1 sec cycle. The building vibration (absolute acceleration & relative displacement) decreases as the damping increases.

2) The figure in Cell (b) of Table 2 shows the response of T4 for 1 sec ground motion. The building shakes with 1 sec cycle during the ground motion and shakes with 4 sec cycle (equal to the fundamental period of building) after the ground motion stops. When damping is small, the response becomes bigger after the ground motion stops. The absolute acceleration becomes bigger and the relative displacement becomes smaller as the damping increases.

3) The figure in Cell (c) of Table 2 shows the response of T1 for 4 sec ground motion. The effect of damping to the acceleration response during the ground motion is unremarkable. The acceleration response after the ground motion stops (after 12 sec) becomes very small compared to that of during the ground motion. The effect caused by different damping value is small after 12 sec.

4) The figure in Cell (d) of Table 2 shows the response of T4 for 4 sec ground motion. The effect of damping is big for both during and after the ground motion.

The above study indicates that the efficiency of response reduction by damping value is higher for the long-period building (T4) compared to the short-period building (T1) for Objective-2, Acceleration Reduction.

2.2. Response to Pulse Wave

A half cycle of sinusoidal input as in the left column of Table 3 is applied as a pulse wave ground motion. Other columns in the table show the response time history of SDF systems.

The center column of Table 3 shows the response of T1. The building shakes as simple harmonic motion with 1 sec cycle after the pulse input. The transient response of absolute acceleration becomes smaller as the damping increases. On the other hand, the first response for the pulse input does not change so much even when the damping increases. On the contrary, higher damping tends to make the initial shake bigger. The damping efficacy for the pulse-like ground motion is low if it is judged only by the maximum response. However, it can be said the efficacy is high if it is judged through the whole response.

3. Response of SDF System with Bi-linear Property for Damping

Following damping systems are common practice for actual structural design,

Table 3. Response Acceleration for Pulse Wave

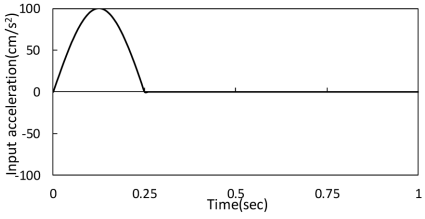
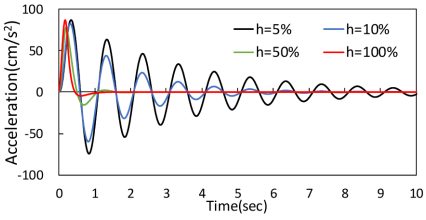
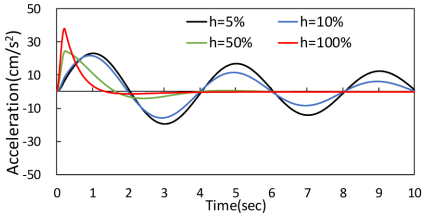
Input wave	T1	T4
		

Table 4. SDF System with Bi-linear Property for Damping

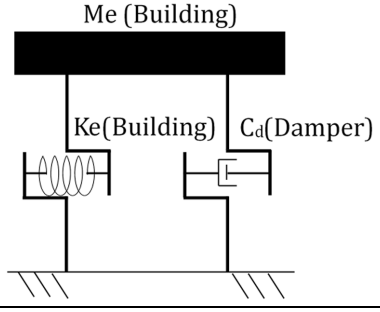
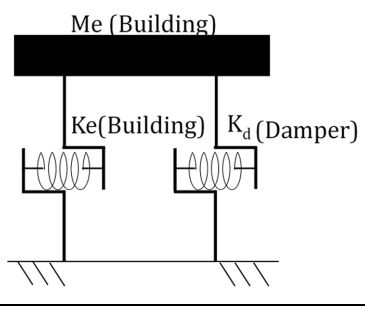
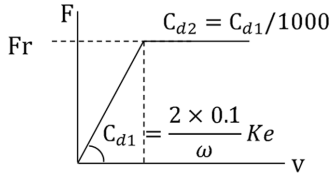
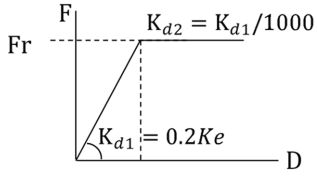
	Viscos damper type	Hysteretic damper type
Image		
Damper property		
Fr	$= 2h \cdot Ke \cdot d$	

Table 5. Property of BI-linear Dampers

Item	T1	T4
Maximum Displacement per 100cm/s^2; d (cm)	7	10
Damping Coefficient; C_{d1} / C_{d2} (kNs/cm)	1.41 / 0.00141	1.41 / 0.00141
Damping Stiffness; K_{d1} / K_{d2} (kN/cm)	177 / 0.177	44.22 / 0.04422
Relief Force; Fr (kN)	6,190	2,211

1) Viscous damping type as represented by Oil Damper
2) Hysteretic damping type as represented by Buckling-Restrained Brace (BRB)

In this chapter, the difference between linear damping as chapter 2 and bi-linear damping is studied. The relief force, Fr, is set to be nearly achieved at maximum deformation, d, for a small-medium earthquake (assuming Max Acc. = 100 cm/s^2). The initial damping coefficient, C_{d1}, for viscous damping type is set as 10% (h = 0.1) and proportional to stiffness. The initial stiffness, K_{d1}, for hysteretic damping type is set as 20% of building stiffness. The image of each analysis model is shown in Table 4.

Table 5 shows the values of damping property for each model, respectively.

Both sinusoidal input and pulse input (with maximum acceleration is 100 cm/s^2 & 400 cm/s^2) are applied to each model. Then, the vibration behavior is compared.

3.1. Response to Sinusoidal Waves

Table 6 shows the response results for 100 cm/s^2 ground motion. The results of viscous damper model show similar results to those with h = 10% in chapter 2 for 1 and 4 sec ground motion, respectively. On the other hand, the results of hysteretic damping model do not show attenuation because the damper does not yield and does not absorb the energy. Table 7 shows the response results for 400 cm/s^2 ground motion. The response velocity exceeds the border value to produce the relief force. Therefore, the damper can not produce the enough damping force as h = 10%. While, the hysteretic damper absorbs the energy by yielding. Especially, the results of T4 model show almost same for 4.0 sec ground motion whose response is

Table 6. Response for Sinusoidal Waves (Max. Acceleration = 100 cm/s^2, Refer to Table 2)

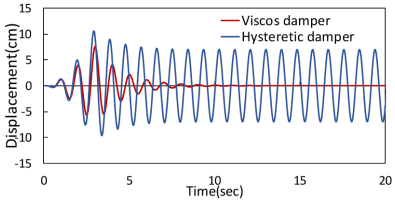
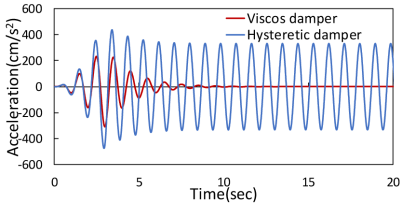
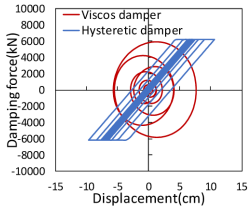
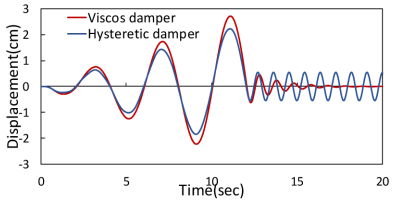
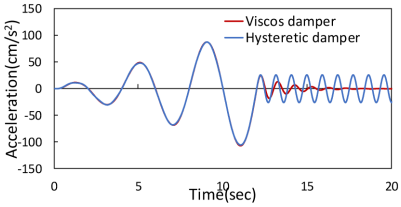
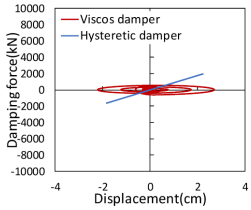
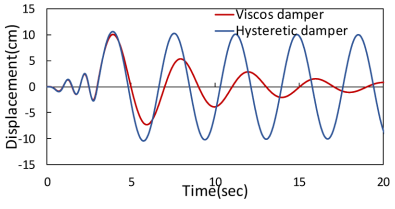
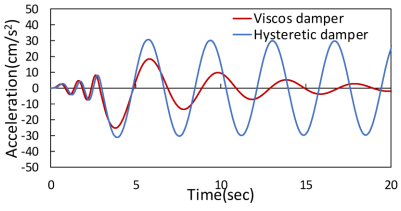
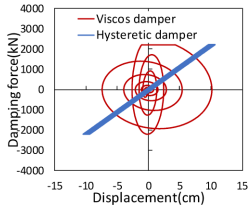
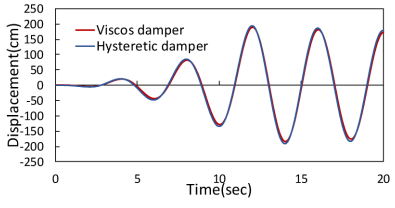
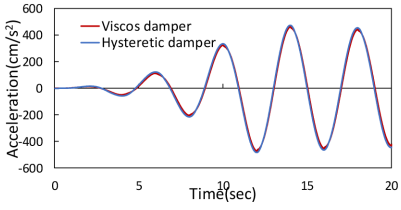
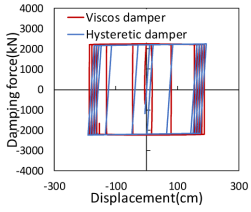
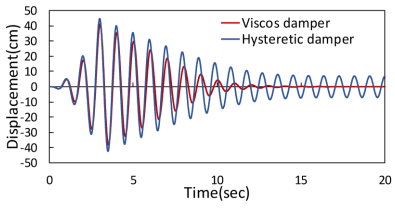
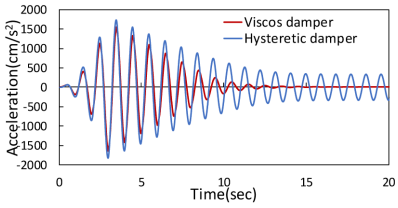
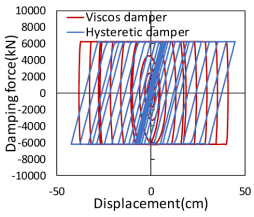
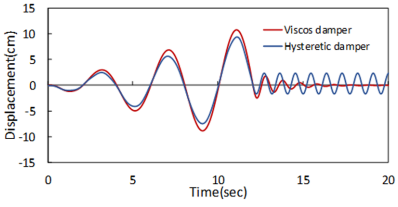
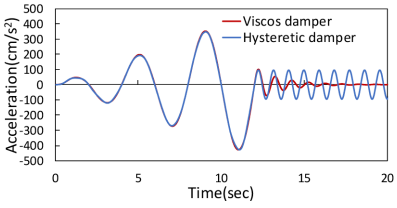
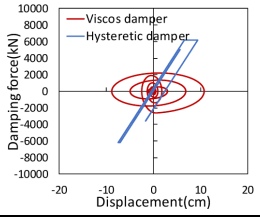
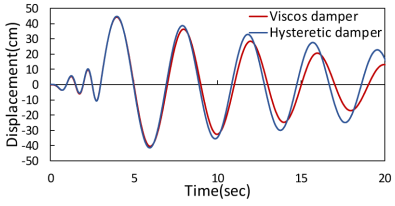
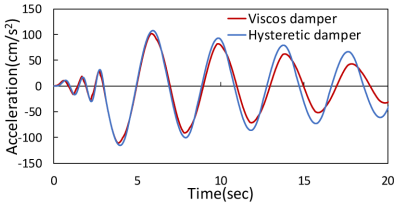
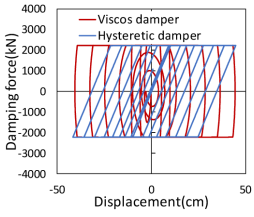
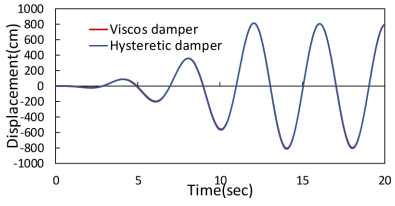
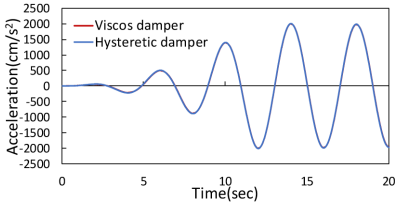
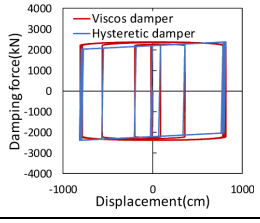
Model	Input	Displacement	Acceleration	Damping Force- Disp.
T1	1.0 sec			
	4.0 sec			
T4	1.0 sec			
	4.0 sec			

Table 7. Response for Sinusoidal Waves (Max. Acceleration = 400 cm/s^2, Refer to Table 2)

Model	Input	Displacement	Acceleration	Damping Force- Disp.
T1	1.0 sec			
	4.0 sec			
T4	1.0 sec			
	4.0 sec			

large. The damping efficacy becomes same for the case of the damping force reaches the limit besides the deformation increases.

3.2. Response to Pulse Wave

Table 8 shows the response results for 100 cm/s^2 ground motion. Table 9 shows the response results for 400 cm/s^2 ground motion. The damping forces are smaller than the relief forces and the results of hysteretic damping model do not show attenuation when 100 cm/s^2 ground motion is applied. The maximum response occurs at the steady state. Remarkable difference can't be seen and the results

Table 8. Response for Pulse Wave (Max. Acceleration = 100 cm/s^2, Refer to Table 3)

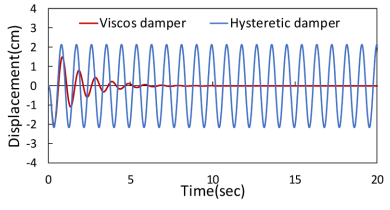
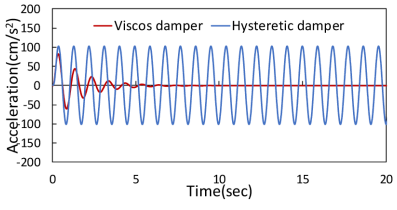
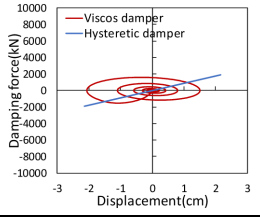
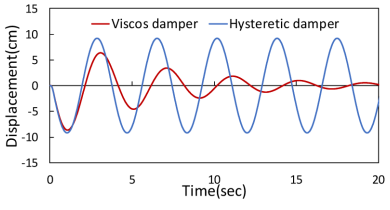
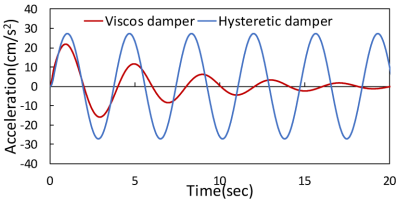
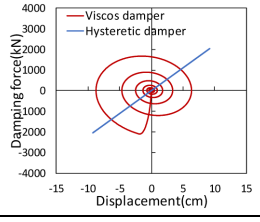
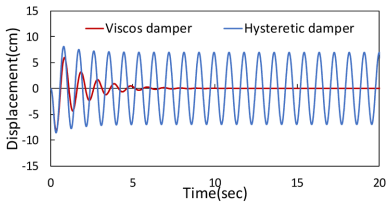
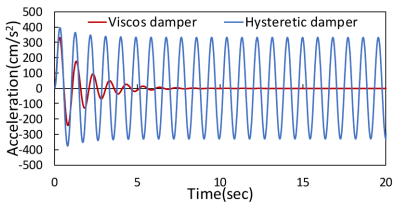
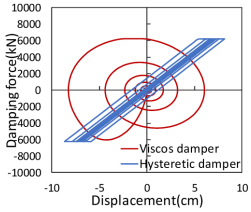
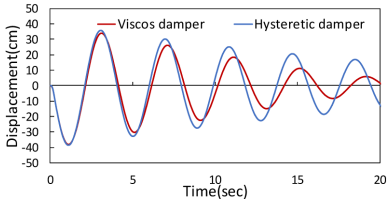
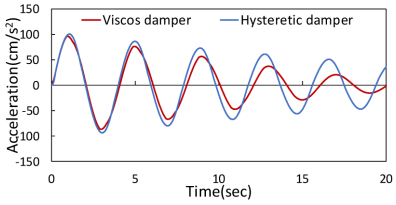
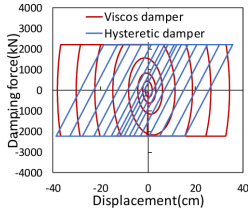
Model	Displacement	Acceleration	Damping Force- Disp.
T1			
T4			

Table 9. Response for Pulse Wave (Max. Acceleration = 400 cm/s^2, Refer to Table 3)

Model	Displacement	Acceleration	Damping Force- Disp.
T1			
T4			

do not depend on each damping capacity. These results indicate that it is not easy to control the maximum response for pulse-like ground motion by damping force.

There is not big difference between the bi-linear viscous damping system (assuming oil dampers) and the bi-linear hysteretic damping system (assuming BRB) for harmonic oscillation per sinusoidal input assuming a big ground motion. Because the qualitative trend is similar to linear damping system. On the other hand, from the other point of view, such as reduction after principal shock or response reduction for small-medium earthquakes, the viscous damping system is valid because it works even in small vibration.

4. Perspective in Design of Response-controlled Structure

As mentioned above, the damping application for response-controlled structure requires to consider the character of seismic ground motions and damping systems. The points to notice are shown below,

1) For example, when the high-rise building is designed at the soft ground area where the predominant period is 1 to 2 sec, the damping system which is targeted the building's 1st mode is not efficient. The damping system and the damper layout which are aimed for the building's 2nd mode is preferable because it is expected that the building's 2nd mode may be resonated with the ground motion.

2) The damping effects is not expected where the site is close to an active fault and the pulse-like ground motions are foreseen. It would be ventured to say that the viscous damping type, which is velocity dependent, is preferable because response velocity is amplified abruptly for pulse input.

3) To control the response for big earthquakes, there is not remarkable difference in response between the structure with viscous dampers and the structure with hysteretic dampers.

4) Viscous dampers are suitable for reduction of a long-term vibration, the shake after principal shock and the shake per small-medium earthquakes.

Hereafter, two design examples with damping system are provided, which were designed based on the points above.

4.1. Design Example-1

4.1.1. Narrative

The design for a tall and slender building requires caution for the pullout of column bases, the overturn of building, the excessive horizontal deformation and so on. It is easy solution to enhance the building stiffness for the reduction of the horizontal deformation. However, it leads the increase of the response acceleration. Then it results in the increase of the overturning moment. In addition to above concerns, the horizontal deformations are likely to concentrate on one particular story with low stiffness and story collapse may be led.

Structure with damping system is effective for these buildings because it enables to reduce the horizontal deformation by adding the damping capacity without adding stiffness. Besides, the structural system with robustness is required to prevent from story collapse, which can exert the structural potential fully without too much dependence on the design assumption such as vertical distribution of horizontal loads. For example, 1 bay moment frame is taken up as shown in Figure 3. The horizontal deformation and stress in a particular floor are unlikely to be transferred to other floors. This structure is deemed as a low-robustness structure which the structural performance depends on the load distribution. On the other hand, if a kind of strong-backbone (SB) is installed into the moment frame, the frame becomes to transfer the stress easily from one certain floor to other floors. This is the robustness frame system which accommodates the unexpected load distribution.

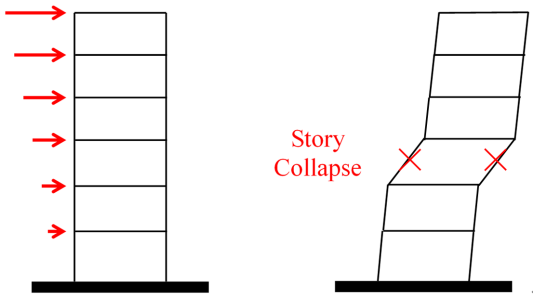


Figure 3. 1 Bay Moment Frame.

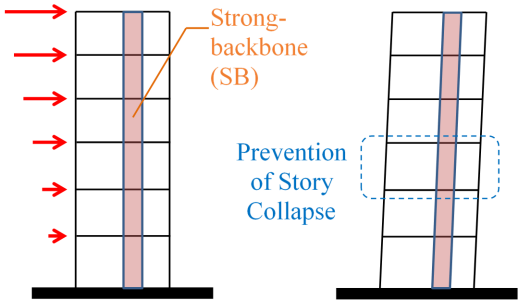


Figure 4. Moment Frame with Strong-backbone.

This design example introduces the new structural system with damping system & SB to enhance the robustness. For damping system, the viscous type dampers were applied. Because the design objective for this building was to reduce the deformation even in a small-medium earthquake.

4.1.2. Building Outline

This building is a retail shop in Tokyo. It is 2 stories below ground and 13 stories above ground. The building height is approx. 55 m with typical floor height of 4.5 m. This building has a characteristic narrow plan with 8 m width & 38 m length and a remarkable slender elevation with aspect ratio approx. 7.



Figure 5. External View of the building.

The superstructure is steel structure with concrete-filled-tube (CFT) columns (Moment frames for short direction, Moment frames with BRB for long direction). The substructure is composite concrete encased structure (Moment frames & shear walls for both direction).

Based on Japanese regulations, this building scale is under the limitation of 60 m height which allows to design by the normal procedure per static analysis. However, the structure with supplemental oil dampers were applied for short direction by consideration of the fact that the building height was very close to the limitation of 60 m and the value of aspect ratio was approx. 7. To enhance

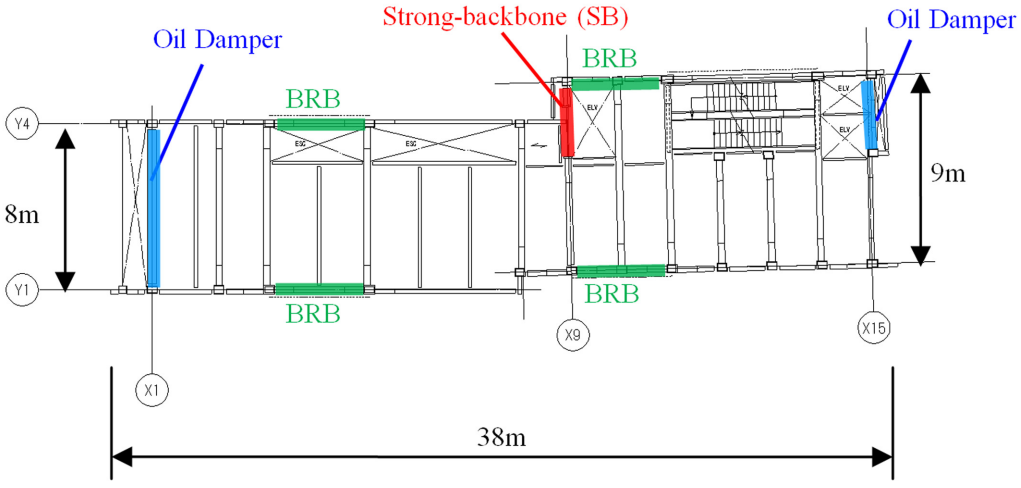


Figure 6. Framing Plan of 2nd Floor.

Table 10. Ground Motions

Name of Ground Motion			MAX. ACC.	MAX. VEL.	Time Duration
Type	Name	Phase	(cm/sec^2)	(cm/sec)	(sec)
Observed Wave	EL CENTRO_NS	Original	510.8	50.0	53.8
	TAFT_EW	Original	496.6	50.0	54.4
	HACHINOHE_NS	Original	333.5	50.0	51.0
Artificial Wave	K_EL CENTRO_NS	EL CENTRO_NS	362.3	78.0	120.0
	K_TAFT_EW	TAFT_EW	356.7	44.4	120.0
	K_HACHINOHE_NS	HACHINOHE_NS	375.1	63.4	120.0
	K_RANDOM	RANDOM	402.9	103.4	120.0

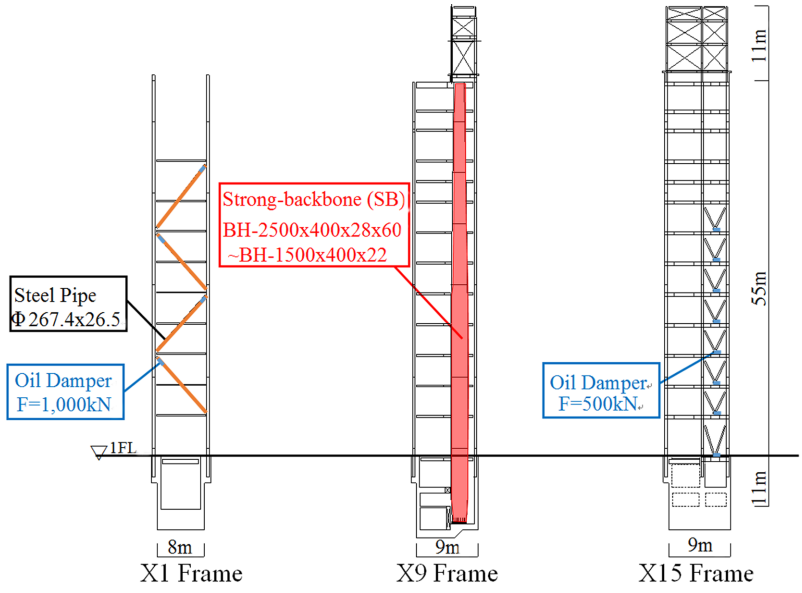


Figure 7. Framing Elevation for Short Direction.

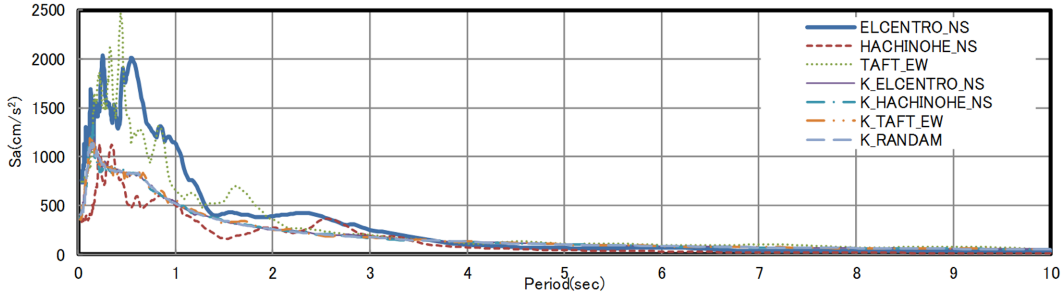


Figure 8. Acceleration Response Spectrum.

the robustness, a SB was installed at the center of floor from the basement to the top of the building.

4.1.3. Design Criteria

4.1.3.1 Ground Motion

Table 10 shows the ground motions applied for the time history analysis. Observed waves which were scaled the maximum velocity = 50(cm/sec) and artificial waves whose return period were 500 years were applied. Figure 8 shows the acceleration response spectrum of each ground motion.

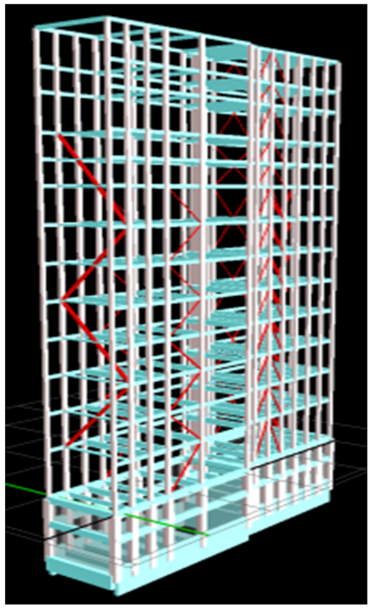
4.1.3.2 Seismic Design Criteria

Table 11 shows the seismic design criteria for each ground motion.

4.1.4. Analysis Model & Results

4.1.4.1 Analysis Model

Figure 9 shows the analysis model. Oil dampers were provided at the face of both narrow sides. The configuration of frames with a SB was adjusted as Figure 10 to prevent too much stress from acting on the connected beams with the SB. The rubber bearings were installed at the bottom



Fundamental natural period	2.033sec for short direction
Damping Ratio	h = 2%
Typical Frame Cross Section	
Steel Column (6~13th Floor)	Built-up Box-390x500x16x19
CFT Column (1~5th Floor)	Built-up Box-390x500x16x19~390x600x40x40
Steel Girder	H-450x200x9x14~H-800x300x16x25

Figure 9. Analysis Model.

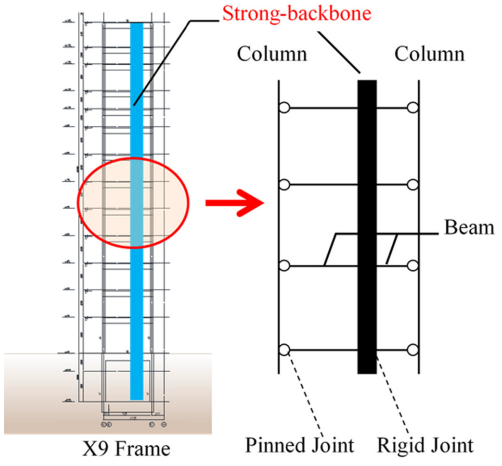


Figure 10. Frame Model with Strong-backbone.

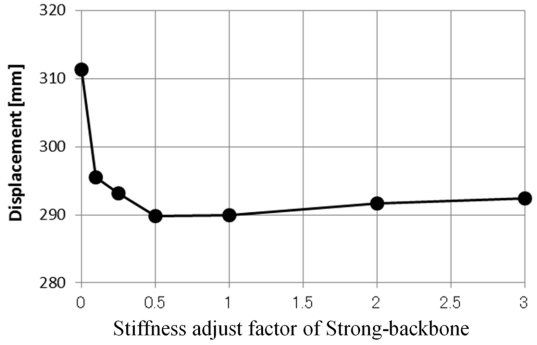


Figure 11. Drift at building top per stiffness of strong-backbone.

of the SB to control the shear force on the SB. Fundamental natural period for short direction is 2.033sec. This mode governs the building behavior.

4.1.4.2. Study for the strong-backbone

A built-up wide flange steel was selected for the cross section of SB because the building is a steel structure and it is easy to be fabricated. To figure out the optimum stiffness of SB for response reduction, the lateral displacement at building top and the strain energy stocked by each member were compared by a parametric study. For the study, Built-up-2500 × 400 × 28 × 60 was set as the base cross section. Then, each study was carried out by changing the moment of inertia of the SB per the stiffness adjust factor, ×0.1, ×0.3, ×0.5, ×1.0 ×2.0 and so on. The lateral displacement at building top was minimum when the stiffness adjust factor is 0.5 to 1.0 as Figure 11. The strain energy stocked by the SB is level off when the stiffness adjust factor is 1.0 or larger as Figure 12. Therefore, Built-up-2500 × 400 × 28 × 60 was applied for this building.

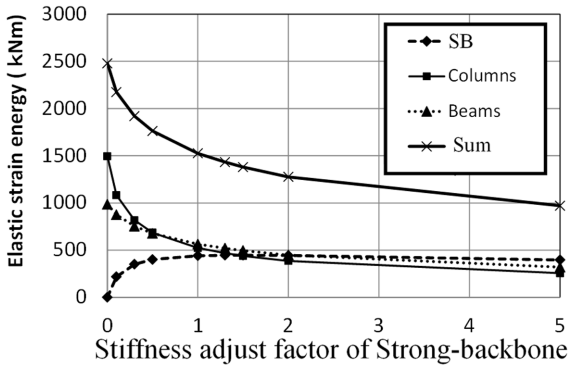


Figure 12. Strain energy of each member per stiffness of strong-backbone.

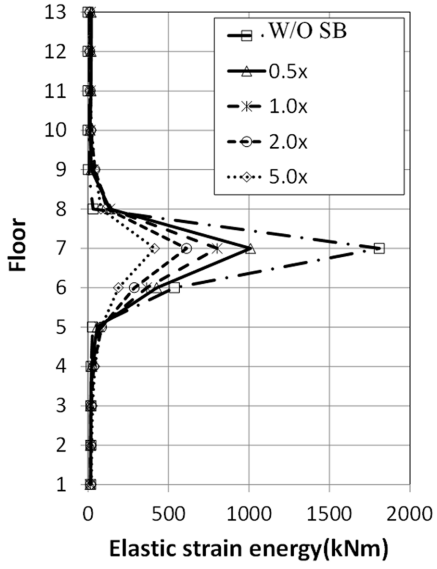


Figure 13. Vertical distribution of strain energy.

For actual application of the SB, the stiffness was reduced at the upper floor per the proportion of supported flexural moment.

Figure 13 shows the vertical distribution of strain energy of SB assuming the load of story-collapse at 7th floor. As the stiffness of SB increases, the strain energy at 7th floor decreases and the strain energy at upper & lower floor increases. The result indicates this structure enables to distribute the local stress and has robustness which the story collapse is unlikely to occur.

4.1.4.3. Analysis Results

3D time history analysis was conducted to verify the effect of SB. Herewith, the results for short direction are presented. The seismic design criteria are 300 mm or less for the drift at building top and 1/100 or less for story drift angle as Table 11.

Table 11. Seismic Design Criteria

Stress of Each Member	Elastic State
Story Drift Angle	≦ 1/100
Horizontal Drift at the Building top	≦ 300 mm

To clarify the effect, following three different models were set up, 1) Without SB & without oil dampers; 2) With SB & without oil dampers; 3) With SB & with oil dampers. Figure 14 shows the drift and story drift angle. The drift at building top was reduced approx. 25% by the SB and further 20% by oil dampers. These results prove the efficiency for deformation reduction by each item.

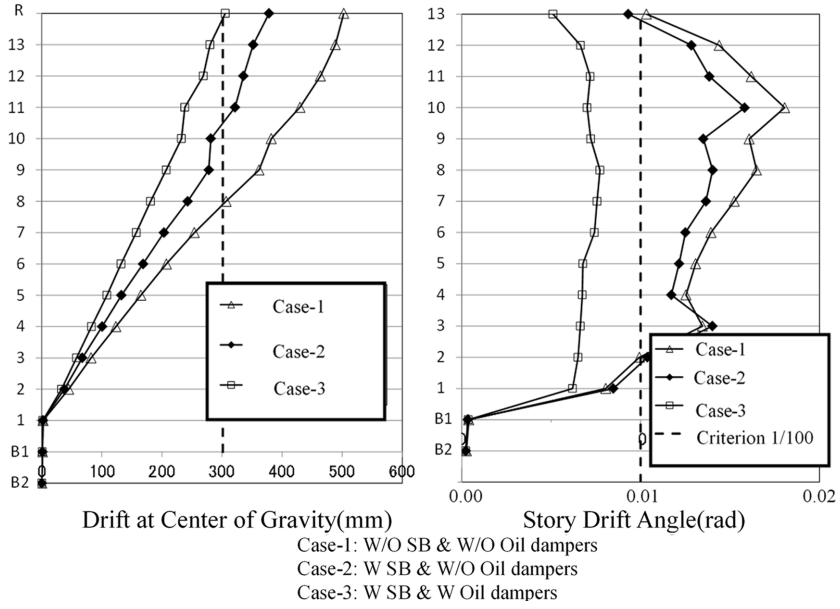


Figure 14. Drift and Story drift angle of each model (K_RANDOM).

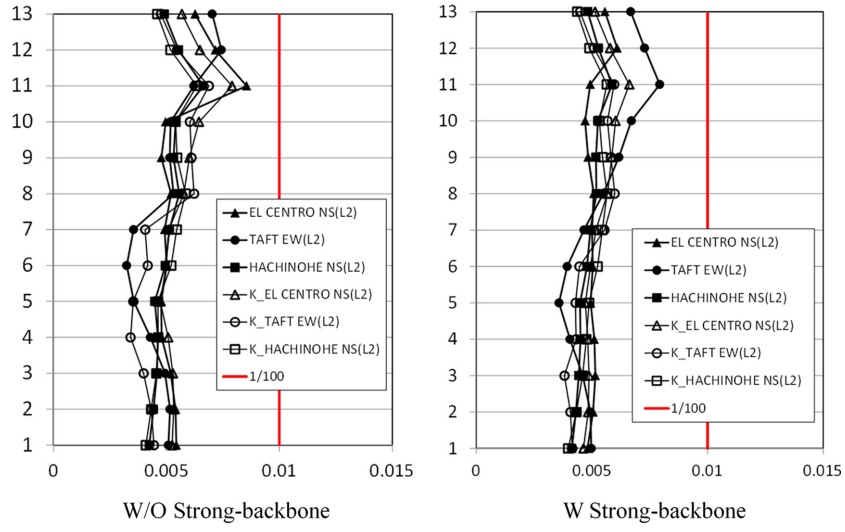


Figure 15. Story drift angles at the center of gravity.



Figure 16. Bottom of Strong-backbone & Rubber bearing.

Figure 15 shows the story drift angle at the center of gravity for the case with or without the SB, respectively. Even though both cases satisfy the design criterion of 1/100, the remarkable deformation concentration at 11th floor was observed for the case without SB.

The story drift angles for the case with SB were reduced approx. 10 to 20% compared to those of without SB. It proves the effect of SB.

4.1.5. Project Specific Design and Detailing

Following special details and consideration were required at the bottom of the SB to prevent too much shear stress from acting on it.

1) At the ground level, horizontal deformation is not allowed and only rocking action can be allowed.

2) The bottom of the SB needs to have a moderate stiffness. The deformation orbit of it shall describe a vertical arc with the center at the intersection between 1st floor and center of the SB.

3) The SB is free from axial force.

For actual design detail at the ground level, the horizontal deformation is restricted by the rigid diaphragm supported by the surrounded soil pressure and the rocking action is achieved by flexural deformation of the connected beams.

The stiffness at the bottom was adjusted by the horizontal stiffness of the rubber bearings which were installed between the bottom of the SB and foundation. Two rubber bearings were set.

The erection procedure of the SB was coordinated not to support the gravity load. The SB is supported by the connected beams at each floor level and the axial stress at the bottom is zero.

These details make the stiffness of the SB adjustable per structural engineer's intention and help to settle the typical issues for a tall & slender building, such as the pullout of column bases and the overturn of building.

4.2. Design Example-2

4.2.1.Narrative

A long-term business suspension by earthquake damage of the large commercial buildings in the city is not only the economic loss. It also causes the big social problem.



Figure 17. External View.

Therefore, the seismic structural design which controls the structural damage as minimum as possible and enables the immediate business resumption (immediate occupancy) is expected recently.

It is required to prove the structural safety against the prescript ground motions such as Level 1(Small-medium scale earthquake) and Level 2(Large scale earthquake) per dynamic design procedure in Japan. On the other hand, the concern for the site-specific ground motion which has the highest possibility of occurrence at the site is growing nowadays.

This design example introduces the project which was designed not only for the prescript ground motions by Japanese regulations but also for the site-specific ground motions. Besides, this example shows the application of the damage-controlled structure design. The expected yield locations are intended to limit within the supple-

mental damping elements and the damage of the main structure is minimized. The supplemental damping elements were designed to be checked up and to be replaced in the future. The seismic monitoring system was installed in the building. It makes easy to figure out the building behavior under the seismic events and the timing of replacement of the supplemental damping elements. For damping system, the hysteretic type dampers were applied. Because the design objective for this building is to reduce the response to the site-specific ground motion, whose scale is similar to the large earthquakes, Level 2.

4.2.2. Building Outline

This is a complex building with retail shops and hotel in Tokyo. The building height is approximately 70 m. The floor shape is approx. 53 m for longitudinal(X) direction & 70 m for short(Y) direction, respectively. The area is surrounded by several railroad stations. Actually, one exit from the railroad station was located in the building site as an existing building. Therefore, this building was constructed over the existing building. The building outline is shown below.

Occupancy: Retail Shops, Hotel
Building Area: 1,400 m^2

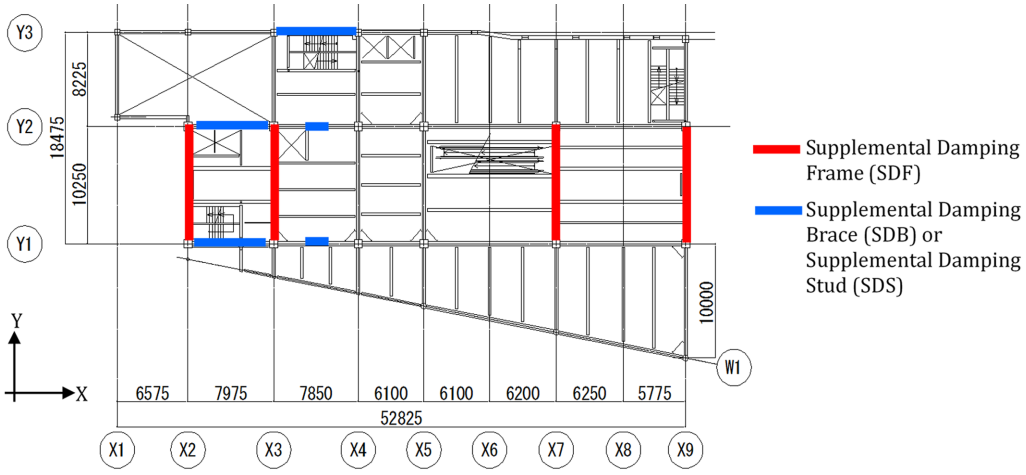


Figure 18. Framing Plan (2nd Floor).

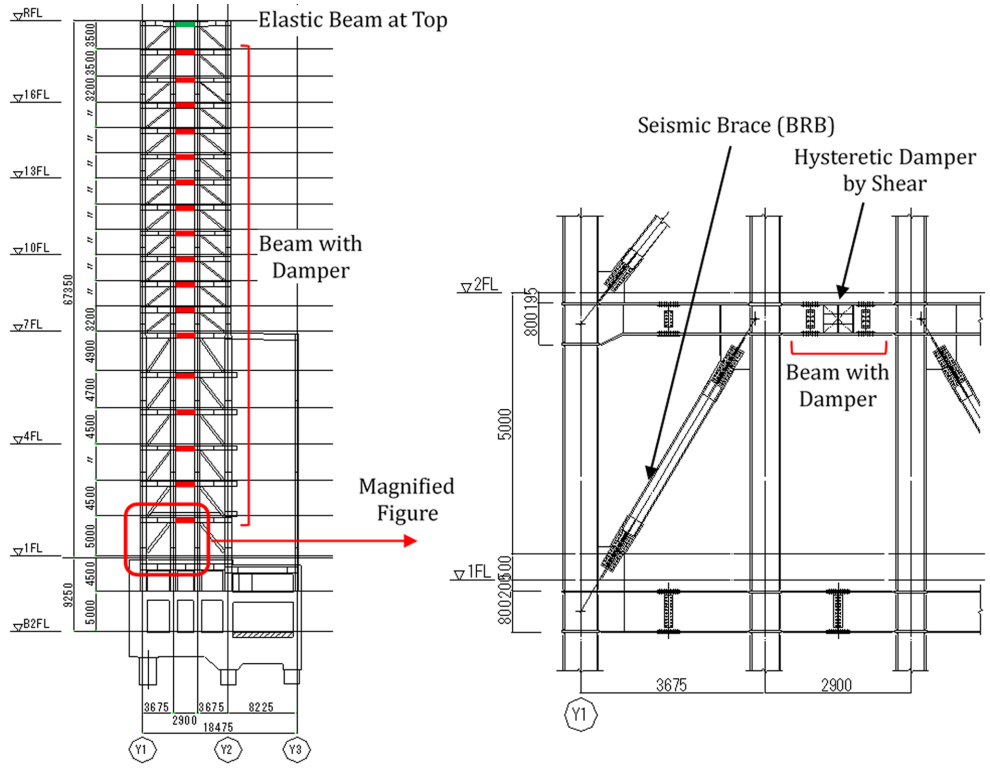


Figure 19. Framing Elevation of Supplemental Damping Frames (SDFs).

Total Floor Area: 14,700 m^2
Building Height: 68.55 m
Floor Number: 2 stories below ground +18 stories above ground +2 stories for penthouse
Structure: Steel Structure for superstructure & Composite Concrete Encased Structure for substructure

The superstructure is steel structure with concrete-filled-tube (CFT) columns. The substructure is concrete encased structure for B1 floor and concrete structure for B2 floor, respectively. Concrete shear walls are provided at sub-structure to achieve enough strength and stiffness.

"Immediate business resumption even after big earthquake" was the request from clients. The damage-controlled structure, which the damage of main structure is controlled by the intended plastic components, was applied to accomplish it. The intended plastic components were made of low-yield-strength steel and installed to absorb the seismic energy exclusively. "Supplemental damping braces (SDBs)", "supplemental damping studs (SDSs)" and "supplemental damping frames (SDFs), composed of seismic braces and damping beams" were applied as the intended plastic components.

Figure 18 shows the layout of the intended plastic components. The aspect ratio for short(Y) direction is approx. 7. Therefore, four SDFs were provided for this direction because these frames are valid for flexural deformation as well. The low-yield-strength steel, LYP225 (Fy = 225 Mpa), was applied for the web plates of coupling beams of the SDF. These web plates yield even in the small earthquakes such as Level 1 and absorb the seismic energy by hysteretic energy. The coupling beam of the top floor was designed to keep elastic state even in the big earthquake such as Level 2 to reduce the residual deformation after the earthquake.

4.2.3. Design Criteria

4.2.3.1 Ground Motion

Table 12 shows the ground motions applied for analysis. In addition to grand motions of Level 1 & Level 2 required per Japanese regulation, the site-specific ground motion which has the highest possibility of occurrence at the site was applied. "Assumed South Kanto Earthquake" which was created by modeling the fault of Kanto earthquake in 1923 was selected for it. The site-specific ground motion was made by Hybrid method, which the ground motion by three-dimensional finite element method for T≧4sec and the ground motion by Empirical Green's function method for T<4sec was merged in the time domain. Figure 20 shows the velocity response spectrum at the level of building foundation.

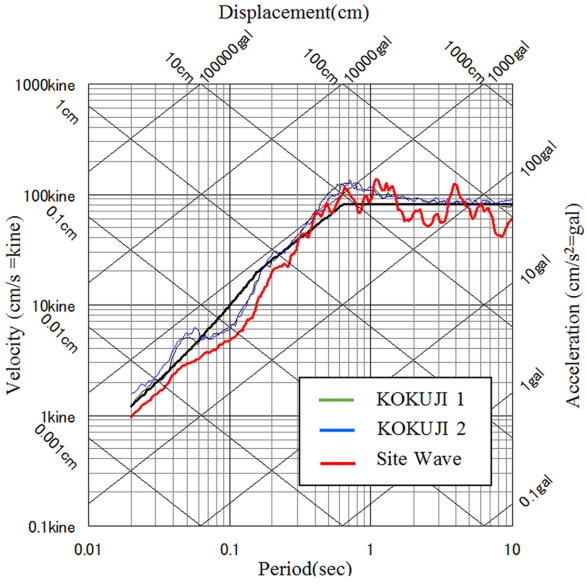


Figure 20. Response Spectrum (at Foundation Level).

Table 12. Ground Motions

Name of Ground Motion		Level 1		Level 2	
Type	Name	Max. Acc.(cm/s²)	Max. Velo.(cm/s)	Max. Acc.(cm/s²)	Max. Velo.(cm/s)
Prescript Ground Motion by Japanese Regulation	KOKUJI 1 (Random Phase)	71	13	334	62
	KOKUJI 2 (EL CENTRO NS Phase)	66	15	329	73
	EL CENTRO 1940 NS	255	25	511	50
	TAFT 1952 EW	248	25	497	50
	HACHINOHE 1968 NS	167	25	333	50
		↓↑			
Site-Specific Ground Motion	Site Wave	Max. Acc. = 284(cm/s²)		Max. Velo. = 48(cm/s)	

Table 13. Seismic Design Criteria

Item	Level 1 Ground Motion	Site-Specific Ground Motion	Level 2 Ground Motion
Stress of Member	Allowable Stress State	Elastic State	Plastic State
Story Drift	≦ 1/200	≦ 1/130	≦ 1/100
Supplemental Damping Element	D-Value (Cumulative fatigue damage) ≦ 0.4		

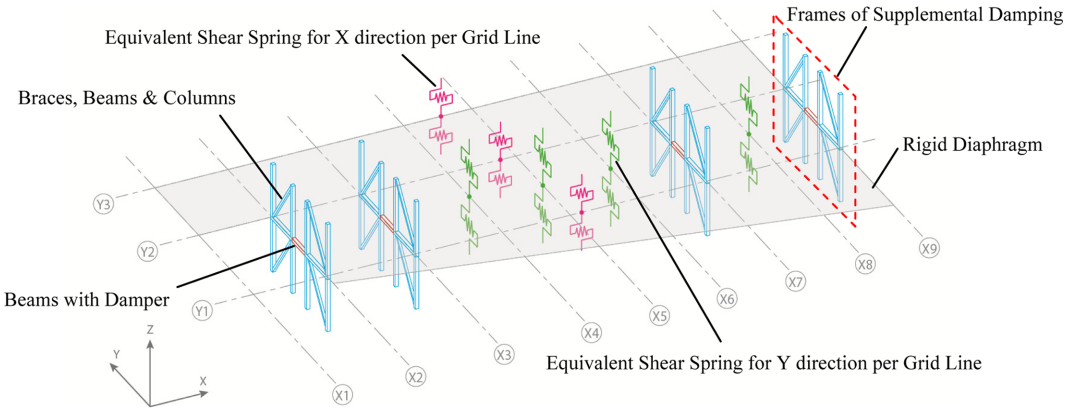


Figure 21. Analysis Model for Time History Analysis (Pseudo 3D).

4.2.3.2 Seismic Design Criteria

Table 13 shows seismic design criteria for each ground motion such as Level 1, Level 2 and site-specific ground motion, respectively.

4.2.4. Analysis Model & Results

Pseudo 3D model was applied for time history analysis to focus on the primal information and to evaluate the flexural behavior and horizontal twist behavior of SDFs. Frames for longitudinal (X) direction were converted into equivalent shear spring per each grid line. Frames for short (Y) direction were converted into two types. SDFs were evaluated as frame model and other frames were converted into equivalent shear spring per each grid line. Each floor was evaluated as a rigid diaphragm. A Seismic mass for horizontal direction and a rotary inertia per vertical axis were applied at the center of gravity of each floor. A damping ratio was assigned as 2% per 1st mode and proportional to the momentary stiffness. Fundamental natural period is 1.98sec for longitudinal (X) direction and 2.00sec for short (Y) direction, respectively.

Figure 22 shows the result of time history analysis for short (Y) direction. For site-specific ground motion, the maximum story drift angle was 1/150 and the maximum story ductility factor was 0.997. For Level 2 ground motion, the maximum strain of web panel in the SDFs was 6.60% (14th Floor). The maximum D-value for cumulative fatigue damage was 0.267 (5th Floor), which was calculated by the rainflow-counting algorithm based on the result of low-cycle fatigue testing for the low-yield-strength steel. Each result satisfies the design criterion.

Figure 23 shows the time history of energies for Level 2 ground motion. Strain energy by supplemental dampers is approximately 50% of the seismic input energy. The proportion of increment strain energy by supplemental dampers to that of viscous damping at the maximum response is 3 to 1.

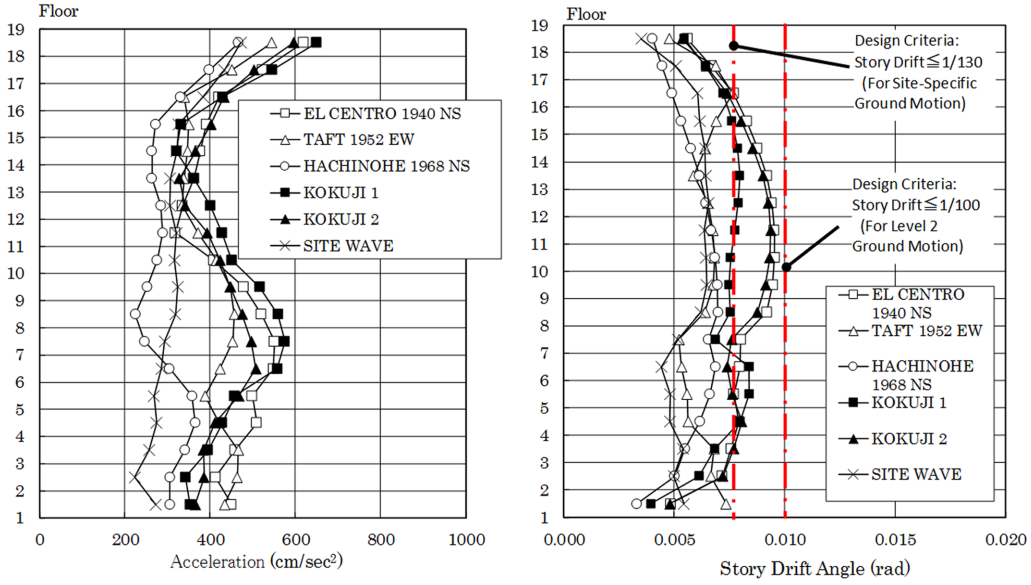


Figure 22. Results of Time History Analysis (Y direction).

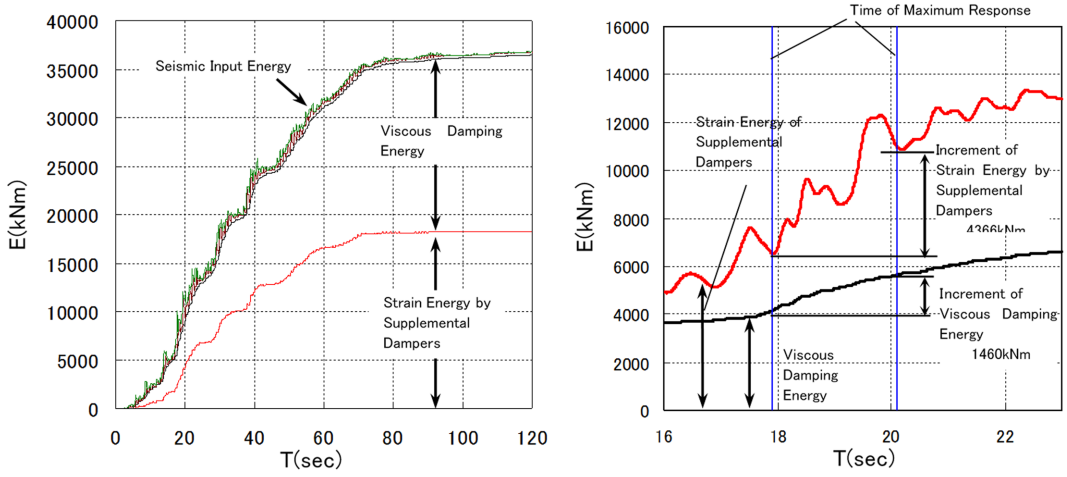


Figure 23. Time History of Energies (Y direction).

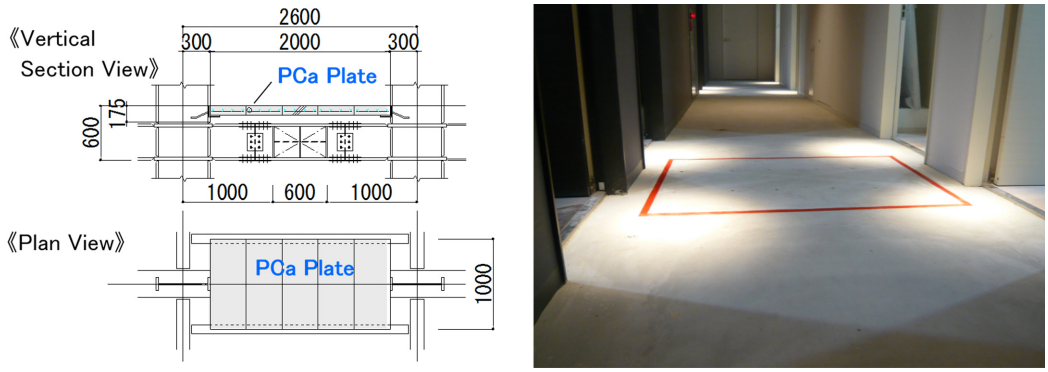


Figure 24. Floor Detail in the vicinity of Beam with Damper.

4.2.5. Project Specific Design and Detailing

4.2.5.1. Floor Detail in the Vicinity of Beam with Damper

Beams with damper are isolated from the slabs to prevent from the deformation restriction by them (See Figure 24). The slabs in the vicinity are precast plates. It enables to check or replace the dampers.

4.2.5.2. Seismic Monitoring System

Seismic monitoring system with strain sensors and accelerometers are installed to figure out the building behavior under the seismic events and to monitor the cumulative fatigue of the hysteretic dampers (See Figure 25). The strain sensors are set on the hysteretic dampers at 2nd, 5th, 9th and 13th floor. The accelerometers are set on the floor of 1st, 9th and 18th. Floor accelerations and maximum strain & cumulative fatigue of the hysteretic dampers can be monitored automatically. This monitoring system makes it easy to figure out the replacement timing

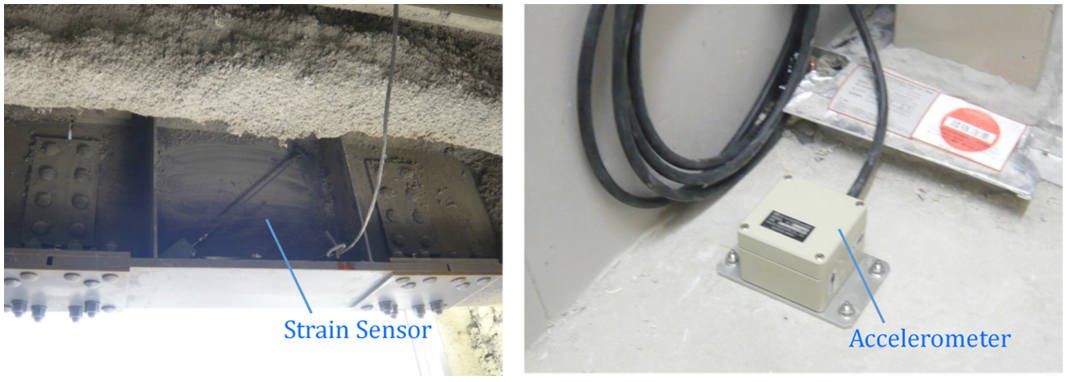


Figure 25. Seismic Monitoring System (Strain Sensor & Accelerometer).

of the beams with damper and to maintain the structural performance.

5. Conclusions

This paper presented the importance to consider the character of both the damping system and the ground motion when we design the response-controlled buildings. We, structural engineers, are requested to propose and provide the suitable damping system based on the objective and design criteria for each building.

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