CHALLENGING APPLICATIONS OF SEISMIC DAMPERS FOR RETROFIT OF TALL BUILDING

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ABSTRACT :

This paper describes two cases where seismic retrofitting work was carried out by using seismic damping members for greater seismic resistance in conjunction with the large-scale renovation work of the tall buildings. Renovation work requires design consideration and invention specific to it, differently from that required for new construction, such as approaches to use the existing building frames so as to reduce the amount of additional members to use, or work methods and execution consideration to minimize noises and vibration. One of the cases described is a renovation work of a tall building which achieved an enhancement in seismic resistance by avoiding removal of existing members and frames or addition of new members wherever possible but instead, installing viscoelastic dampers into the connections between existing brace members, thereby improving the damping performance of the building. The other case is about a tall building which increased its seismic energy absorption capacity by replacing existing braces with buckling restrained braces. This work adopted a bonding work system using epoxy resin so that a minimum of noise and vibration would be caused at the connections between the damping braces and the existing building structure. Each of the cases described in this paper is a successful project in effectively improving the seismic resistance of an existing tall building. It can be expected that these approaches will be applied to renovation cases faced with similar problems in the future.

KEYWORDS: Seismic retrofitting, Viscoelastic damper, Buckling restrained brace

1. INTRODUCTION

An increasing number of buildings are being renovated in Japan lately instead of reconstructing new buildings, partly out of consideration to reducing environmental burden. With the aim of creating new demands through refurbishment and expansion of existing buildings, there are also cases such as building conversion for its use change, or a large-scale renovation where the interior and exterior finishes are completely altered to renew the image of a building in an attempt to increase the property value. In these renovation works, the seismic capacity of the existing buildings often presents a problem. To enhance the capacity, seismic retrofitting is undertaken in a growing number of cases at the time when the buildings are renovated to renew the interior and exterior finishes. This paper describes two cases where seismic retrofitting work was carried out by using seismic damping members for greater seismic resistance in conjunction with the large-scale renovation work of the tall buildings.

2. EXAMPLE OF THE USE OF BUCKLING RESTRAINED BRACES FOR SEISMIC RETROFITTING OF HIGH-RISE HOTEL

2.1. Background and Outline of Retrofitting Work [1)]

This building is one of the major international hotels in Japan, constructed in connection with the holding of the Tokyo Olympics in 1964. It was the first high-rise building more than 60m in height to obtain the approval for the structural design of high-rise building from the Ministry of Construction. It also marked the first time in Japan's building design history that many new technologies were adopted (such as full surface glass curtain walls and modern bathroom units) and represented the pinnacle of structural and construction technology at the time.

However, it has been more than 40 years since the hotel was constructed, and the following design modifications were conducted to improve its image.

- State-of-the-art techniques were used to completely replace the exterior aluminum panels around the building with full-height glass curtain walls (Fig. 1).
- The guestrooms were converted from single to twin or deluxe type rooms (interior renovation).
- The water distribution pipes, hot and cold water pipes and other facilities were upgraded.
- Seismic retrofitting was conducted to increase earthquake-resistant performance.

A refined exterior design was pursued. In addition, the materials for the exterior were carefully selected to fulfill the condition of maintaining generally the same weight as the existing exterior so as to place as little loading on the structural members as possible.

Fig. 1After the retrofit

This hotel has many guestrooms as well as numerous banquet halls, restaurants and other commercial functions not exclusively related to guest accommodations. As a result, it was important to ensure that these functions were unimpeded during the renovation process, and the project schedule had to ensure that the period in which guestrooms were unavailable due to construction was as brief as possible.

In addition, due to the building's function as a hotel, a renovation method that produced as little noise and vibration as possible during the renovation work was needed.

2.2. Overview of Seismic Retrofitting

Prior to renovation, a time-history response analysis was conducted for the existing structure, comparing it with the current Building Standard Law and official notifications. Six earthquake ground motions were used for the study: three simulated ground motions specified in official notifications and three previously monitored ground motions (El Centro 1940 NS, Taft 1952 EW and Hachinohe 1968 NS). In preparing the three simulated ground motions specified in official notifications, the phase characteristics of three types of ground motion observed in Japan (the records for the EW component measured at Hachinohe Harbor during Tokachi-oki earthquake in 1968, the NS component measured at Tohoku University during Miyagiken-oki earthquake in 1978, and the NS component measured at the Kobe marine-observatory during Hyogo-ken Nanbu earthquake in 1995) were used in order to enable different ground motion characteristics to be considered. As a result, it was confirmed that although the buildings were safe with respect to a large earthquake ground motion, the maximum story drift angle would exceed 1/100 in the event of some of the simulated ground motions specified in the official notifications currently in effect occur, as shown in Fig. 2. This is because the level of the ground motion used for the study, noted in official notifications that are currently in effect, is greater than the ground motions (El Centro and Taft) used for the study conducted at the time the building was designed.

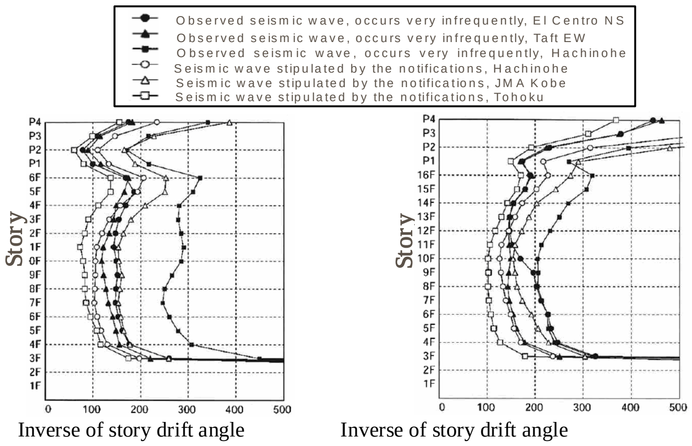


Fig.2 Before the seismic retrofit

Fig.3 After the seismic retrofit

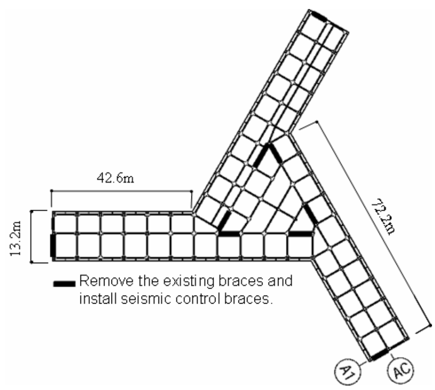
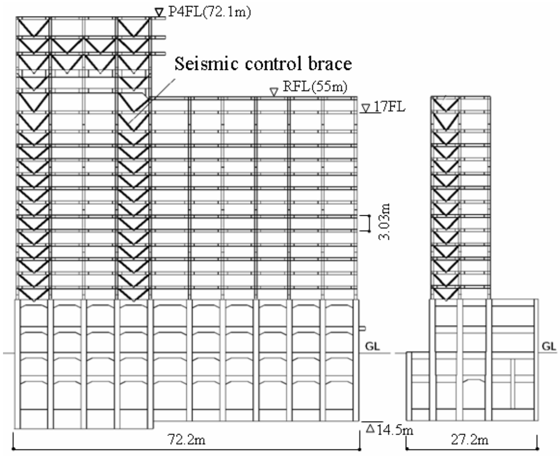


Fig.4 Beam plan of standard floor

For this renovation project, it was decided to use buckling restrained braces to increase damping force, primarily in order to improve the earthquake-resistant performance of the structure in the event of a large earthquake. Buckling restrained braces are designed to yield during a medium-intensity earthquake or greater, thereby absorbing energy and reducing the load on the existing columns and beams. The target values after retrofitting were to keep maximum story drift angle to 1/100 or less and to keep the maximum ductility factor of the members to 4.0 or less (ultimately max. 1.7). Figure 3 shows the maximum story drift angle following retrofitting.



AC grid line framework drawing   A1 grid line framework plan

Fig.5 Framing elevation (After retrofit)

With regard to the locations at which buckling restrained braces were placed, in order to avoid expanding the scope of the retrofitting, the existing earthquake-resistant braces (channel steel) that had been placed on multistory levels were removed and the buckling restrained braces were placed at the same locations (Fig. 4). In addition, in order to reduce noise and vibration to the greatest extent possible, epoxy resin was used to connect the buckling restrained braces with the existing structural members wherever possible. The conventional welding and anchoring methods were also used in places.

LY225 was used as the core material for the buckling restrained braces, and the placement of the braces was done so as to maintain a proper balance between the triple arrows shaped outermost ends and the core (Fig. 5). At the core in the center of the building, reverse V-type braces were placed using the bonding and anchoring methods (Photo 1 and Photo 2). As there were stairwells at the outermost ends, a V-shaped brace was placed using the conventional method of welding a gusset plate directly to the existing steel frame. In this way, braces were placed on multistory levels.

2.3. Bonding Method Using Epoxy Resin

The bonding method used for this project was to inject epoxy resin in the gap (approximately 6 mm) between the steel plate (hereafter "shear-key plate") and the concrete slab in order to bond the plate to the slab (Photo 3). The horizontal component of the tensile force produced in the brace is transmitted by means of bearing pressure from the horizontal plate (not bonded to the slab) attached to the gusset at the end of the brace to the shear-key plate that touches the metal, and it is transmitted from there to the existing slab and main beam by means of the bonding force from the epoxy resin. The shape of the shear-key plate is rectangular (550 x 2,500 mm). The horizontal component of the compressive force produced in the brace is transmitted through integration of the horizontal plate and column by welding. Figure 6 shows a detailed view.

Photo 1. Seismic control braces

For the bonding strength of the epoxy resin, a safety factor that was double that of the load-carrying capacity in actual-size model tests was adopted. The left and right shear-key plates were integrated in the center to create a space with a fail-safe mechanism capable of transmitting the compressive force from the opposite side as well.

Photo 2. Seismic control brace joint

Photo 3. Epoxy resin being injected

Considering the building's function as a hotel, these connections represent an effective joining method. This method made unnecessary the site welding and concrete slab chipping and other processes involved in conventional techniques and produced almost no noise, vibration, dust or the like. Strength and properties of the joints were obtained from an actual size model tests and then evaluated and reflected in the section design. [2)]

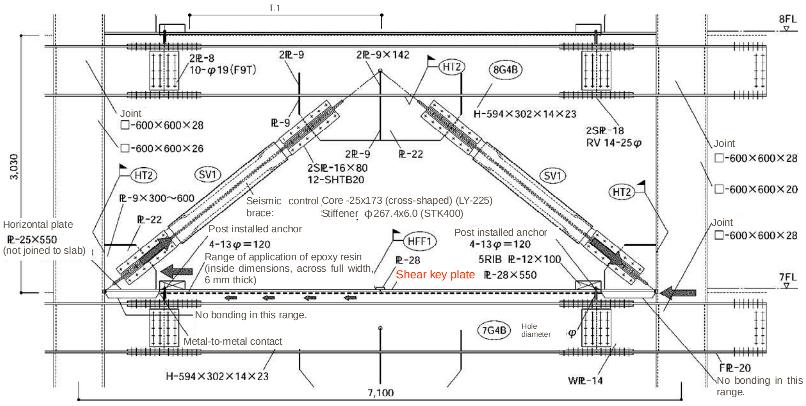


Fig.6 Detailed drawing of seismic control braces

3. EXAMPLE OF SEISMIC RETROFITTING OF HIGH-RISE HOTEL USING VISCOELASTIC DAMPER

3.1. Background and Overview of Renovation Work[3)]

This building is an accommodation facility with 22 aboveground floors and 4 below-ground floors. The designer at the time obtained the approval for structural design of high-rise building from the Minster of Construction in 1990. The building's highest section is SGL + 90.0 m and it has a floor area of 29,376 m^2 (original design 39,180 m^2). Figure 7 shows the plan and section views of the existing building.

In order to conserve resources by reducing the environmental burden, it was decided to not demolish and rebuild the building but rather to renovate it and provide additions and modifications to the existing building that would enable it to satisfy new needs. Accordingly, building conversion was implemented, in which additions and renovations were conducted for sections of the lower floors of the existing high-rise building and the standard floors were upgraded in order to accommodate a change in building use.

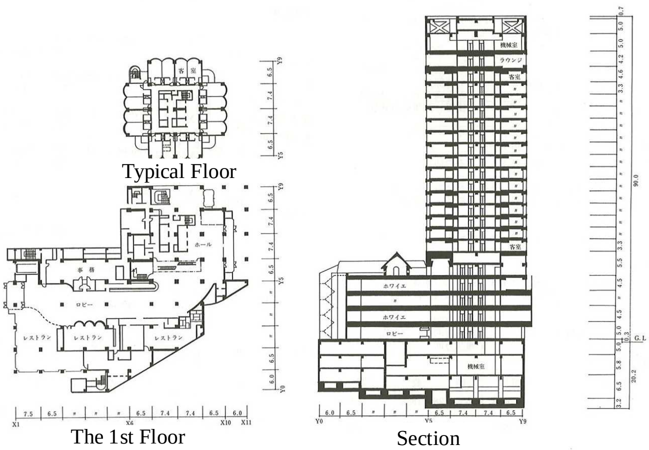


Fig. 7 Existing building plan and section views

3.2. Overview of Structural Planning in Original Design

The existing building comprises mainly a high-rise part with guestrooms and a low-rise part with banquet facilities. The high-rise part is of steel frame construction beginning from the 5th floor. The structural form is rigid frame construction with braces. Figures 8 and 9 show a cross-shaped rigid frame that is generally symmetrical in both X and Y directions. Floors 6 through 19 have pre-cast concrete walls (see Fig. 10) with buckling restrained braces for the door perimeters of each guestroom as earthquake-resistant elements. Floors 5 and 20 through penthouse 1st floor have V-shaped braces made of H-shaped steel at all four corners.

The low-rise part from floors 1 through 4 is of rigid frame construction using steel-reinforced concrete. In accordance with the floor plan, reinforced concrete earthquake-resistant walls are placed in a balanced configuration. This is omitted on sections of the below-ground floors; down to the B1 floor steel-reinforced construction is used, while below that is reinforced concrete construction. Sufficient earthquake-resistant walls are placed in underground sections to provide a rigid construction. The basement is mat foundation, which supports the whole weight of the building on the diluvial sand.

The beams of the steel structure consist of wide-flange shape steel and the material is SM50A in all cases. The width-thickness ratio is kept in the range to guarantee the stable plastic behavior. Main beams are restrained by enough quantity of the orthogonal beams to prevent the lateral buckling.

In the 5th and the upper floor, box columns assembled by welding are used. The material is SM50A or SM50B. The width-thickness ratio is kept in the range to guarantee the stable plastic behavior.

The braces on the top four floors (Floors 20 through P1) comprise a combination of V-shaped and K-shaped braces made of rolled H-shaped steel measuring 300 x 300. The braces on the 5th floor are V-shaped braces made of rolled H-shaped steel measuring 350 x 350. The material in both cases is SM50A. From the 6th floor through the 19th floor, FB-340 x 32 (SM50A) is placed in a double X configuration. This steel, in unbonded status, is driven into a pre-cast concrete wall measuring 200 mm in thickness.

Table 2 and Fig. 11 show the earthquake-resistant performance of the existing building. In the original design, the primary natural period of the building was 1.84 sec in the X direction and 1.87 sec in the Y direction.

Table 2 Earthquake-resistant performance of existing building

Intensity of earthquake motion	Maximum velocity	Earthquake-resistant performance		
		Response story shear force	Response maximum story drift angle	Response maximum ductility factor of story
Moderate	25 cm/s	Elastic limit strength or below	1/188	Elasticity range
Large	50 cm/s	Confirmed ultimate horizontal resistance or below	1/81	1.88

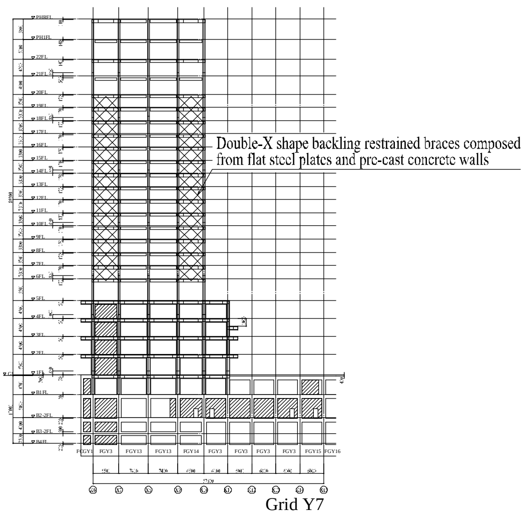


Fig.8 Structural frame in original

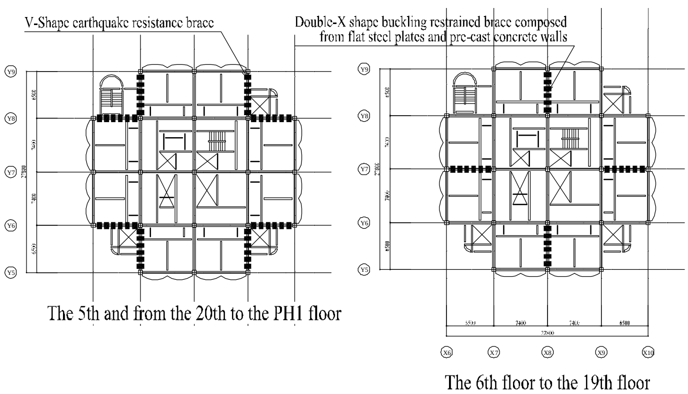


Fig.9 Structural plan in original

3.3. Outline of Structural Design by Means of Conversion Plan

In the conversion planning, the major changes in the high-rise part were the change of use (conversion of floors 1 and 2 to store use, conversion of floors 3-11 to condominium use, and change of below-ground floor use) and the addition of maisonettes in the condominium section on standard floors (floors 3 through 21). In addition to the floor and bearing wall removal and the reinforcement work that accompanied the conversion, the following reinforcement work was conducted to improve earthquake-resistant performance:

(a) Reinforcement and upgrading of the earthquake-resistant braces provided on the 5th floor
(b) Upgrading of pre-cast concrete walls with buckling restrained braces provided on standard floors (floors 6 through 19) through the addition of viscoelastic dampers

(a) involved adding stiffening material to the earthquake-resistant braces on the 5th floor to increase the brace strength to a degree that would not exceed the strength of the joint.
With regard to (b), in the original design, buckling restrained braces had been used as energy absorbing members during a large earthquake. The following modifications were made to the joints of the pre-cast concrete walls with buckling restrained braces in order to provide them with damping force functions:

In the area around the column and beam joints, the top end brace joint sections were left completely force-free so they would not be constrained in the event of horizontal deformation in the in-plane direction. Falling protection support was provided in the out-of-plane direction.
As shown in Fig. 13, acrylic viscoelastic members (ISD111) were inserted in the area around the joints in the center of the beams (9 mm in thickness on floors 6 through 10 with an injection area of 13,500 cm^2; 8 mm in thickness on floors 11 through 19 with an injection area of 12,000 cm^2) to create viscoelastic dampers. These prevent excessive force from being produced in the event of horizontal deformation in the in-plane direction, and they also add damping force to produce a damping effect.

As a result of these modifications, the overall deformation of the building during a large earthquake as well as the degree of ductility of the column and beam members were reduced, improving the earthquake resistance of the building during a large earthquake.

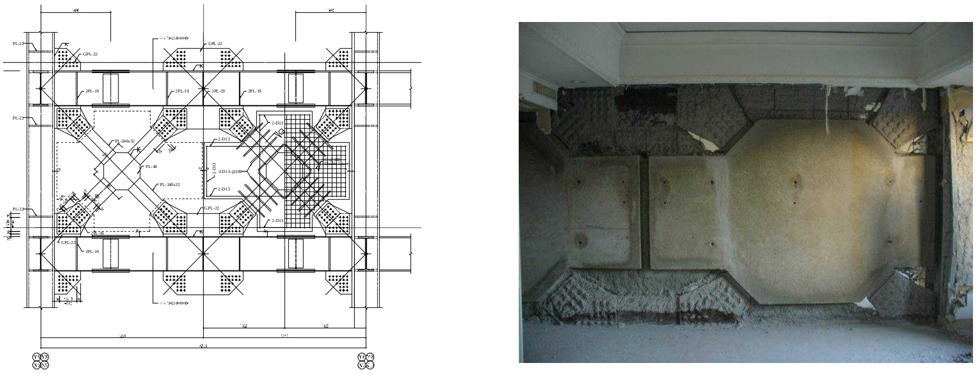


Fig. 10 Double-X shape buckling restrained brace

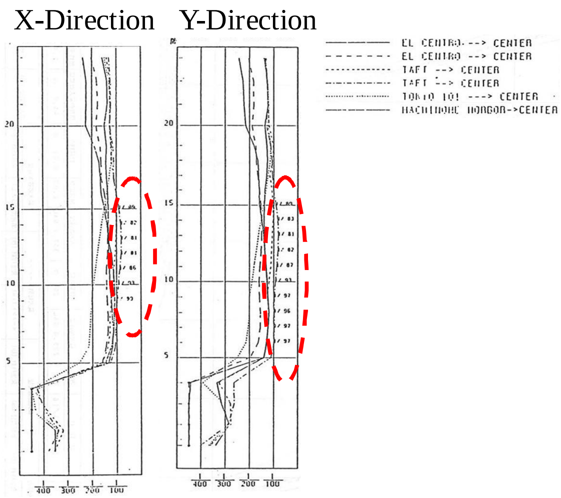


Fig. 11 Response maximum story drift angle during large ground motion in original design

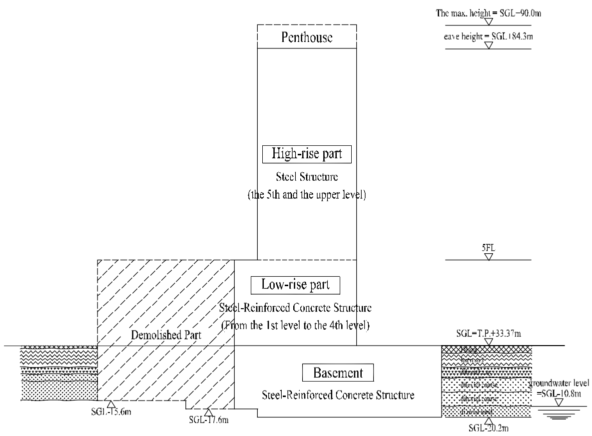


Fig. 12 Overview of structural planning in conversion plan

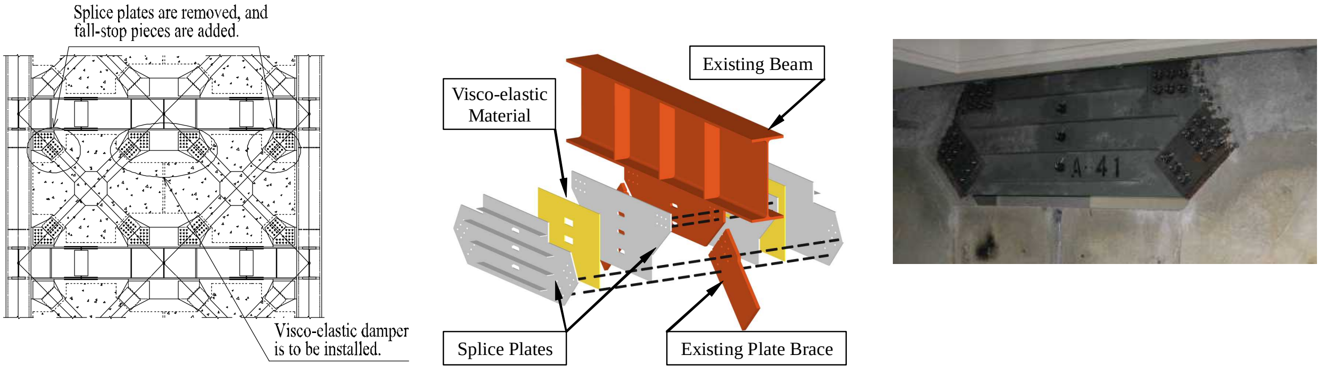


Fig. 13 Overview of installation of viscoelastic damper at the joint between brace and beam on standard floor

3.4. Confirmation of Earthquake-resistant Performance by Means of Time-history Response Analysis

Table 3 shows the design criteria used in the conversion planning for this building.

Table 3 Earthquake-resistant performance objectives following renovation

Intensity of earthquake motion	Earthquake-resistant performance		
	Response story shear force	Response maximum story drift angle	Response maximum ductility factor of story
Moderate	Elastic limit strength or below	1/190 or below	Elasticity range
Large	Confirmed ultimate horizontal resistance or below	1/90 or below	2.0 or below

The earthquake ground motion used for the study is the same as that shown in section 3.2. The 1st floor was chosen as the building location at which the ground motion was applied. This was because at this location the S-wave velocity of the diluvial gravel layer at the base of the foundation was generally the same value as that of the engineering bedrock, and because the rigidity of the building ground floor was sufficiently high as compared to the upper structure. A space frame model in which the building was modeled at the member level with one mass point per story was used as the vibration system model. Out of consideration for internal viscous damping, the damping force characteristics were derived by superimposing a damping coefficient of h = 0.02 for steel construction (floors 5 and above) and h = 0.03 for steel-reinforced concrete construction (floors 1 through 4) in order to derive a damping force matrix. For the viscoelastic damper sections, generalized Maxwell damping force elements (6 elements) capable of considering frequency dependence were taken into consideration in the spaces between nodes.

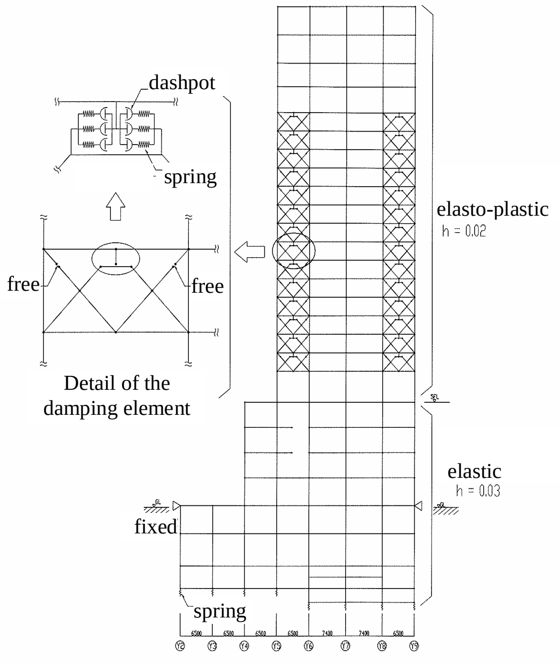


Fig. 14 Vibration response analysis model

The primary natural period of the building following renovation was 2.02 sec in the X direction and 2.07 sec in the Y direction, indicating that the primary natural period had increased as compared to the original design. This is mainly due to the dismantling of the low-rise part and the addition of viscoelastic dampers to the joint sections of the pre-cast concrete walls with buckling restrained braces on standard floors (floors 6 through 19).

Figure 15 shows the results of the vibration response analysis conducted for large earthquake ground motions. The maximum response for story drift angle on each floor was 1/120 in the X direction and 1/90 in the Y direction. The response maximum ductility factor for each story was 1.00 or less in the X direction (the members did not reach the full plastic moment) and 1.37 in the Y direction. These values fulfill the design criteria. Moreover, the maximum ductility factor for major members was 1.50 (for response to Hachinohe NS in the Y direction). The response maximum shear strain of the viscoelastic damper sections was approximately max. 270% in both directions, lower than the target of 300%.

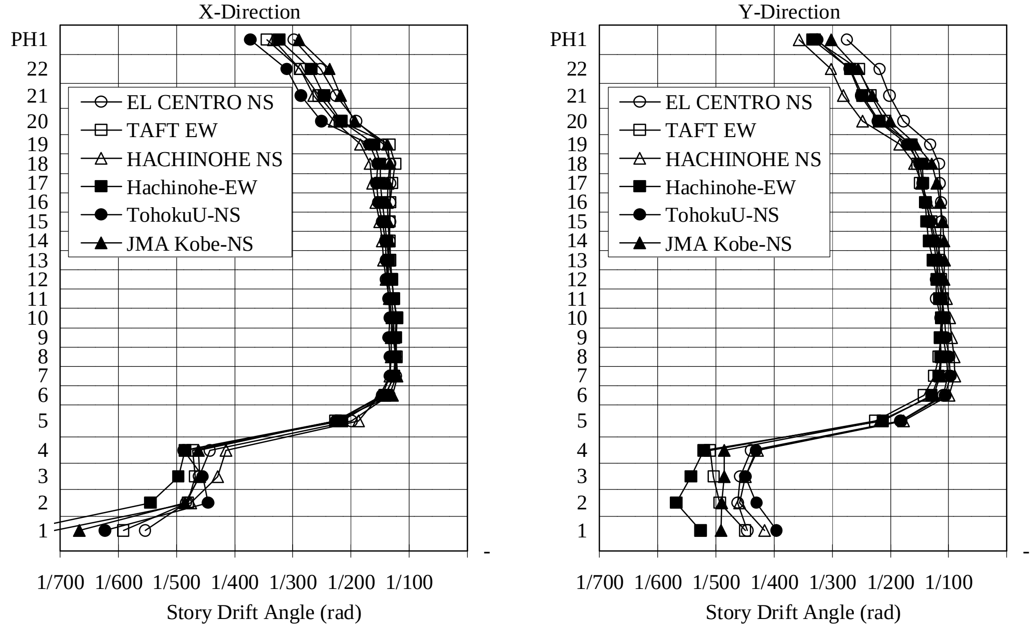


Figure 15 Response story drift angle against large ground motions (after rehabilitation)

4. CONCLUSION

Renovation work requires design consideration and invention specific to it, differently from that required for new construction, such as approaches to use the existing building frames so as to reduce the amount of additional members to use, or work methods and execution consideration to minimize noises and vibration.
Each of the cases described in this paper is a successful project in effectively improving the seismic resistance of an existing tall building while using the characteristics of existent building framework through the effective use of damping members in an effort to overcome the constraining conditions specific to renovation work. It can be expected that these approaches will be applied to renovation cases faced with similar problems in the future.

REFERENCES

1. Hiroaki Harada, Yasuhiro Tsuneki, Hiroyuki Hayashi. (2007). Seismic retrofit design for high-rise hotel using seismic control steel braces by the bonding method. Structural Engineers World Congress 2007
2. Yasushi Ichikawa, Kazuaki Suzuki, Hiroaki Harada, Satoshi Yamada, Akira Wada. (2006). Estimation of strength of seismic retrofitting brace joint using shear-key plate adhered to concrete slab for steel structure. Journal of structural and construction engineering **610**,177-184
3. Kohei Shintani, Michio Keii, Katsuhide Murakami, Yozo Shinozaki. (2007). Seismic Rehabilitation of a Tower Building with Visco-elastic Damper. Structural Engineers World Congress 2007