# FULL-SCALE SHAKE TABLE TESTS OF 5-STORY STEEL BUILDING WITH VARIOUS DAMPERS

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**Abstract:** Realistic simulations of earthquake responses were conducted in March 2009 for the full-scale 5-story building specimens with dampers using the E-Defense, the world's largest three-dimensional shake table. The building was tested repeatedly, inserting and replacing each of 4 damper types, i.e., steel damper, oil damper, viscous damper, and viscoelastic damper. This paper discusses test concept, method and test results as well as details of the 5-story building specimen. Performance improvement by the dampers will be addressed for moderately tall buildings that constitute a major portion of the building stock.

## 1. INTRODUCTION

The E-Defense shaking table facility, whose construction was completed in early 2005, is the largest earthquake simulator capable of subjecting full-scale structures to the strongest earthquakes recorded in the world.

Using the facility, three major research projects were completed on geostructures, wooden buildings, and reinforced concrete buildings, respectively. Currently, projects on steel buildings and bridges are being pursued. Figure 1 shows overall organizations for the steel building project that focuses on moment-resisting frames, innovative methods for new or existing buildings, nonstructural elements, and protective systems. It is pursued by four working groups (WG's) shown in Figure 1.
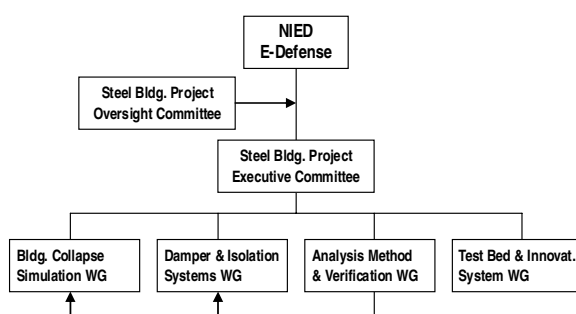


Figure 1. Organization of E-Defense Steel Project

This paper addresses the work of Damper and Isolation Systems WG. This paper discusses test concept, method and test results, as well as details of the 5-story building. The building was tested repeatedly, inserting and re-placing each of 4 damper types, i.e., steel damper, oil damper, viscous damper, and viscoelastic damper. Moderately tall buildings that constitute a major portion of the building stock will be considered.

## 2. TEST CONCEPT AND METHOD

### 2.1 Validation of Passive Control Technology

Japan has constructed the largest number of passively-controlled buildings, and is believed to have conducted the most extensive research to realize various control schemes. A variety of dampers are being produced by more than twenty manufacturers and more than ten general construction companies in Japan. Numerous technical papers on the schemes are also published.

Most major Japanese buildings designed and constructed after the 1995 Kobe earthquake are either base-isolated or passively-controlled in order to better protect the building and its contents. However, because of their short histories, the schemes have never been attested under the major and catastrophic quakes, while they are increasingly used in Japan. Thus, it is extremely important to validate these advanced schemes by realistic experiments, before occurrence of such earthquakes.

The full-scale shake table test made possible by the E-Defense would be the best option for such validation.

## 2.2 Moderately Tall Steel Buildings

A 5-story steel building is considered as the specimen, since it represents many office buildings seen in Japan; it is about the tallest of the majority of steel building stock, and; it tends to deform, if not damped, much more than taller steel buildings under the major quake. The last point is discussed below:

Figure 2 shows spectral displacement $S_d$ vs. acceleration $S_a$ for the design basis earthquake (DBE) and maximum considered earthquake (MCE). The spectra for the ground motion recorded at JR Takatori Station during the 1995 Kobe Earthquake are also shown. Damping ratio 5% is considered. The plots are for elastic responses, but they will be used here as crude estimates for inelastic responses.

It is customary in Japan to estimate the building vibration period T by the building height $H$. Possible range of $T$ for a building with $H$=16m, similar to the specimen height (Sec. 3) is shown in Figure 2. Story drift angle averaged over the building height $\theta_{avg}$ is also shown, by approximating it as $S_d/(2H/3)$. The $2H/3$ is the effective height, corresponding to a case of straight mode shape.

It was statistically found that the Japanese code formula $T$=0.03$H$ is reasonably accurate for taller Japanese MRFs. In contrast, for moderately tall MRFs considered here, the range of $T$=0.05$H$ to even 0.07$H$ is found to be more appropriate. From Figure 2, this would lead to $\theta_{avg}$=0.017 to 0.027 under the DBE, suggesting significant structural and no-structural damage, as vibration period is longer.

Such an MRF would be vulnerable against stronger quakes such as the Takatori motion whose spectra are much larger (Fig. 2). Moreover, consequently large ductility demand will produce $\theta_{avg}$ larger than estimated by this elastic approach. Further, inelastic drift may be highly non-uniform and concentrate in a particular story, largely exceeding $\theta_{avg}$. These were demonstrated by the Building Collapse Simulation WG (Fig. 1), testing a full-scale 4-story steel building ($H$=14.4m) with $T$=0.063$H$ (Suita 2009, Yamada 2009). The building collapsed under the Takatori ground motion, due to the large drift and non-ductile column behavior at the 1st story.

Accordingly, moderately tall steel MRF tends to be vulnerable due to its larger deformation caused by the relatively long vibration period. The Japanese code limits the story drift angle to 1/200 under the static force estimated from the acceleration spectrum similar to that of DBE (Fig. 2). However, since acceleration is constant and thus limited in the shorter period range (Fig. 2), the drift limit is easily satisfied by the short and moderately tall buildings. This results in larger flexibility and thus longer vibration of such buildings.

Based on these, and since simply increasing the MRF member sizes to control drift would be very uneconomical, the use of dampers is believed to be an alternative and attractive option. In spite, passive control is hardly utilized for short and moderately tall buildings in Japan, since the code does not yet adequately address such applications. Moderately tall building, therefore, was selected as the specimen, because of its significant potential to impact the above circumstances.
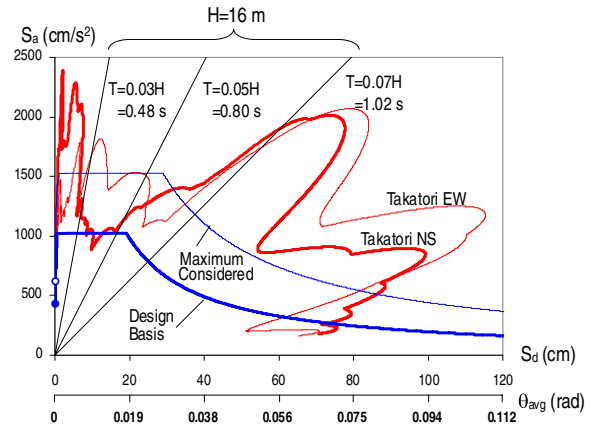


Figure 2. Displacement vs. Acceleration Spectra (Design Basis Earthquake, Maximum Considered Earthquake, and JR Takatori Ground Motions, 5% Damping Ratio)

## 2.3 Test Method

The objective of the present test is to validate reliability of the passive control technology by conducting realistic experiments. Four major types of dampers are selected, and for the economical reason the building will be tested repeatedly, inserting and replacing each of the damper types. In this regard, the frame members must be kept almost elastic without much residual deformation. Such requirement is considered to be important for design of actual damped building, thus, a practical beam column connection detail to enhance the elastic limit of the frame is developed and used for the specimen.

The aforementioned JR Takatori ground motion that caused collapse of the 4-story specimen is used in order to demonstrate contrasting performance and appeal to the community for promoting safer seismic environments. In this regard, the building without dampers will be tested at the end of the test series, which is expected to demonstrate that even the MRF with improved design will suffer significant deformation and damage in contrast to the damped case. The JR Takatori motion will be scaled 0.05, 0.1, 0.2, 0.4, and 1.0 times, in order to compare the performance among the four damped frames as well as un-damped frame at various seismic levels. White noise excitation as well as free vibration test will be pursued. Ambient vibration frequencies and modes are also constantly monitored after erection of steel skelton, until end of the tests. Dynamic properties at moderate shaking will be also measured by operating two vibrating machines set on the building roof, during the break between the shake table tests.

The number of data channels will be about 1,350 which is the largest among all the tests performed previously at E-Defense. The quantities to be measured are as follows:

(1) Strains: columns strains, beam strains, damper

strains, anchor bolt strains, slab reinforcement strains, slab surface strains, panel zone strains, and cladding support member strains, etc.

(2) Deformations and Displacements: story drift and twisting deformation, damper deformation, foundation rocking, column rotation, beam rotation, ceiling displacement, stairway displacement, exterior panel in- and out-of-plane displacement, partition in-plane displacement, door shear deformation, and ceiling edge and partition relative displacement, etc.

(3) 3D-Accelerations: shake table accelerations, each floor accelerations, story drift support accelerations, ceiling accelerations, and cladding accelerations, etc.

(4) Others: pressure between ceiling edge and partitions, ceiling hanger reaction force, motion records outside and inside the building (including axial and transverse damper deformation) etc.

## 3. BUILDING FRAMES WITH DAMPERS

Japanese dampers can be categorized into five major types shown in Figure 3, and four types such as shown in Figure 4 are considered: they are steel, oil, viscous, and viscoelastic dampers. The building will have 12 dampers of the same type with three to four different sizes. For each type, full-scale dampers of three different sizes were dynamically tested at Tokyo Institute of Technology (Kasai et al. 2008). Detailed descriptions for the test results and analytical models are described elsewhere (Ooki et al. 2009).

As shown in Fig. 5, the building is 5-story with two bays in each direction. Due to the reduction in budget, the building is made smaller than originally planned and described elsewhere (Kasai et al. 2007, 2008). The plan dimension is 10m × 12m, and total height from the upper surface of a stiff foundation beam is 15.8 m (Fig. 5). Seismically active weight of the superstructure is 4,730 kN, including all structural/non-structural components and a portion of live load (Table 1).

The frame members of the superstructure consist of either rolled or built-up wide-flange beam sections of 400 mm deep (Table 2), and cold-formed square box column sections of 350 mm × 350 mm (Table 3). Note that each
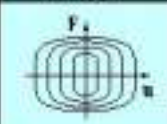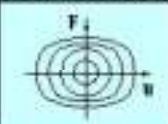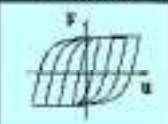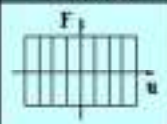

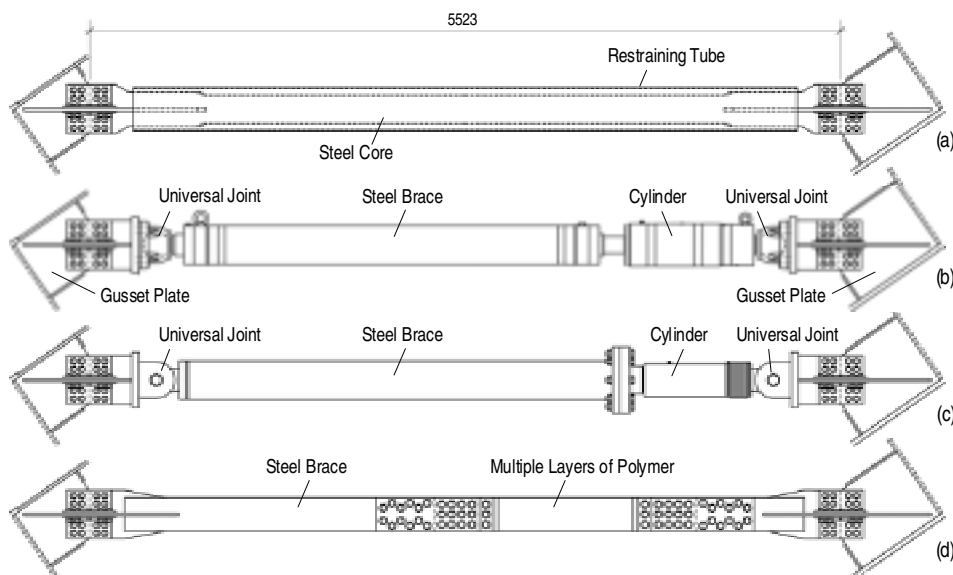
Figure 3. Five major types of dampers used in Japan.



Figure 4. Sizes and configurations of 4 types of dampers to be used (between 2nd and 3rd floors)
: (a) steel, (b) oil, (c) viscous, and (d) viscoelastic dampers

span consists of three beam portions, a center beam portion and two end beam portions that are bolt-connected through the splice plates on flanges and webs. For the beams of MRF bays (G2, G3, G12, and G13 in Fig. 5), the the center portion uses smaller cross section, as indicated by Table 2. And for the beams of damper bays (G1 and G11), the same sections are used for all the three portions in order to resist large axial forces transmitted from the damper. The coupon tests for all members indicate actual yield stresses of the columns and beams were in average about 1.2 times the nominal values (Table 4).

All the beam and column connections will be a fully-restrained type. For the beams of MRF bays, the flange is haunched to increase yield rotation and to delay onset of yielding. And for the beams of damper bays, haunch was considered unnecessary due to large sections created by the gusset plate (Kasai et al. 2009).

Figure 6 shows exterior views of the building. The

precast light-weight curtain walls and glass curtain walls are provided to the 1st and 2nd story levels only. The walls are not attached to the damper bays for ease of dismantling/mantling the dampers. Figure 7 shows four types of damper inserted in the building specimen.

The steel deck with normal concrete on top will be considered and fully composite beams will be created (Fig. 8). The concrete thickness is 80 mm above the corrugated metal deck of 75 mm high. The stairway (Fig. 5) is detailed to slide during shaking, thus, it does not produce significant twisting against building. At every story level above the 1st, partitions with doors are constructed. Two types of ceilings with sprinkler systems, as well as mechanical equipment are placed at some story levels.

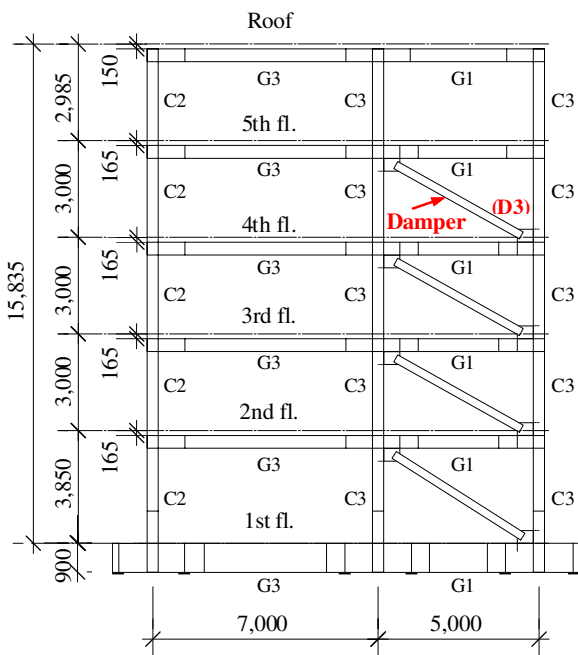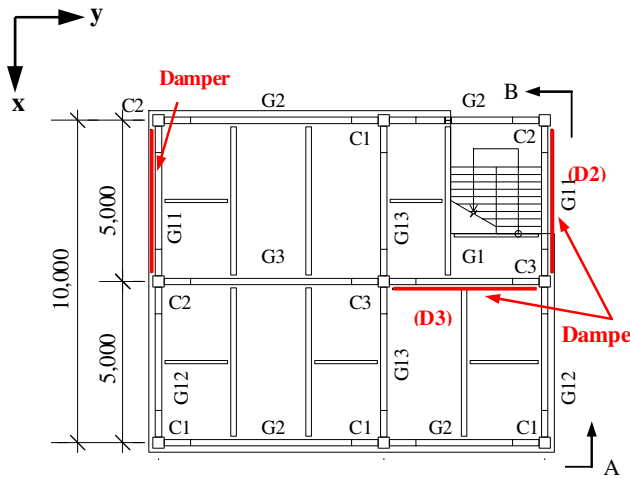Figure 9 shows measurement system of damper stroke and displacement of damper brace.

Table 1.  Breakdown of seismically active weight for the full-scale 5-story building specimen (Unit: kN)

| Floor | Floor | Steel frame | Exterior wall | Interior wall | Live load | Others | Total weight |
|---|---|---|---|---|---|---|---|
| RF | 963.4 | 111.6 | 127.8 | 20.3 | 81.7 | 150.0 | 1454.8 |
| 5F | 436.2 | 99.5 | 100.3 | 26.5 | 98.8 | 37.5 | 798.8 |
| 4F | 436.2 | 117.4 | 100.3 | 26.5 | 98.8 | 37.5 | 816.7 |
| 3F | 436.2 | 122.7 | 100.3 | 26.5 | 98.8 | 37.5 | 822.0 |
| 2F | 436.2 | 131.3 | 108.8 | 28.7 | 98.8 | 37.5 | 841.3 |
| Total | 2708.2 | 582.5 | 537.5 | 128.5 | 476.9 | 300.0 | 4733.6 |
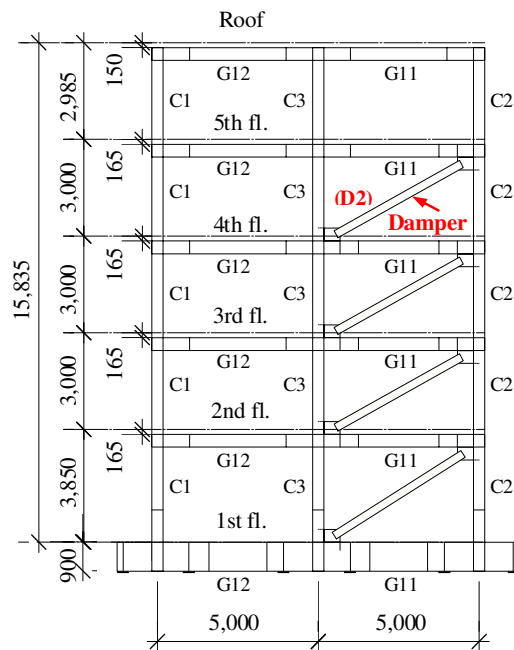


Figure 5.  Plan and elevations for full-scale 5-story building specimen

| Floor | G1(Full portion) | G2(End portion) | G2(Center portion) | G3(End portion) | G3(Center portion) |
|---|---|---|---|---|---|
| RF | H-400x200x9x12 | BH-400x200x9x12 | H-400x200x9x12 | BH-400x200x12x16 | H-400x200x9x12 |
| 5F | BH-400x200x12x16 | BH-400x200x12x16 | H-400x200x9x12 | BH-400x200x12x16 | H-400x200x9x12 |
| 4F | BH-400x200x12x19 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 4F | H-400x200x12x22 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 2F | H-400x200x12x22 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 1F | BH-900x500x16x28 | BH-900x500x16x28 | | BH-900x500x16x28 | |

| Floor | G11(Full portion) | G12(End portion) | G12(Center portion) | G13(End portion) | G13(Center portion) |
|---|---|---|---|---|---|
| RF | H-400x200x9x12 | BH-400x200x9x12 | H-400x200x9x12 | BH-400x200x9x12 | H-400x200x9x12 |
| 5F | BH-400x200x12x16 | BH-400x200x12x16 | H-400x200x9x12 | BH-400x200x12x16 | H-400x200x9x12 |
| 4F | BH-400x200x12x16 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 4F | BH-400x200x12x19 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 2F | H-400x200x12x22 | BH-400x200x12x19 | H-400x200x9x16 | BH-400x200x12x19 | H-400x200x9x16 |
| 1F | BH-900x500x16x28 | BH-900x500x16x28 | | BH-900x500x16x28 | |

Table 3. List of cross section sizes for all columns

| Story | C1 | C2 | C3 |
|---|---|---|---|
| 5 | □-350x350x12x12 | □-350x350x12x12 | □-350x350x12x12 |
| 4 | □-350x350x12x12 | □-350x350x12x12 | □-350x350x12x12 |
| 3 | □-350x350x16x16 | □-350x350x16x16 | □-350x350x19x19 |
| 2 | □-350x350x16x16 | □-350x350x19x19 | □-350x350x19x19 |
| 1 | □-350x350x19x19 | □-350x350x22x22 | □-350x350x22x22 |

Table 4. Yield and ultimate strengths of steel used (actual vs. nominal values)

| | $\sigma_y$(N/mm$^2$) | $\sigma_u$(N/mm$^2$) |
|---|---|---|
| Column | 346-398 | 430-470 |
| (BCR295) | 295 | 400 |
| Beam | 331-422 | 510-557 |
| (SN490B) | 325 | 490 |
| Gusset plate | 342-365 | 510-520 |
| (SN490B) | 325 | 490 |







Figure 6. Exterior views of the building specimen under construction
(Dec. 2008 and Jan. 2009)

Figure 7.    Four types of damper inserted in the building specimen (Feb. and Mar. 2009)



Figure 8.    Interior views of the building specimen under construction (Dec. 2008 and Jan. 2009)

Figure 9.　Measurement system of damper stroke and displacement of whole damper brace (Feb. and Mar. 2009)

## 4.　OBSERVED RESPONSES

Measured responses of main structural components such as dampers, frame and system during the shake table test are as follows.

Figure 10 shows comparison between the story shear based on inertia forces and the story shear based on member forces at 1st, 3rd, and 5th story in x-direction for the building with steel dampers under the 100% Takatori motion. As Figure 10 shows, both story shear based on inertia forces and story shear based on member forces match well. In addition, the former is about 10% larger than the latter probably because of the contribution from non-structural components.

Figure 11 shows relationship between damper forces and damper stroke of each of the four types of dampers at 1st story under the 15%, 50% and, 100% Takatori motions. As shown in Figure 11(a), steel dampers behave elastically under 15% Takatori motion, and elasto-plastically under 50% and 100% Takatori motions. As Figure 11(c) shows, oil dampers behave linearly under 15% and 50% Takatori motions, and non-linearly under 100% Takatori motion due to working of relief valve. As shown in Figure 11(d), viscoelastic dampers behave linearly regardless of shaking intensity.

Figure 12 shows relationship between story shear based on inertia force and story drift for the building with steel dampers. As Figure 12 shows, hysteresis curves at 1st and

3rd stories with dampers indicate significant energy dissipation by the steel damper. On the other hand, hysteresis curve at 5th story without damper indicates elastic behavior.

Figure 13 shows peak responses such as story shear, story drift angle and floor acceleration of building specimen with four types of dampers under 50% and 100% Takatori motion. In addition, peak responses of building specimen without damper are shown in Figure 13 for comparison. Note that, for the 100% Takatori motion, they are extrapolated as 2 times those for 50% Takatori, because shaking was limited up to the level of 70% Takatori motion for the safety reason. As shown in Fig. 13, story drift angle under 100% Takatori motion for the building with each type of damper is less than the design target value of 1% radian. As a whole, peak responses of the building having dampers are considerably less than those without dampers.

The recorded damper deformation is used as input for analytical prediction of damper force, and thus-obtained damper force appears to match well with that recorded during the test. Figure 14 shows comparison of these two damper forces for each damper type. Analysis gives good estimation for viscous, oil, and viscoelastic dampers except for the steel damper, that is not very accurately predicted by using the bilinear model. Analysis model including Bauschinger effect, strain hardening and dependency for velocity and frequency has been developed for improved accuracy.
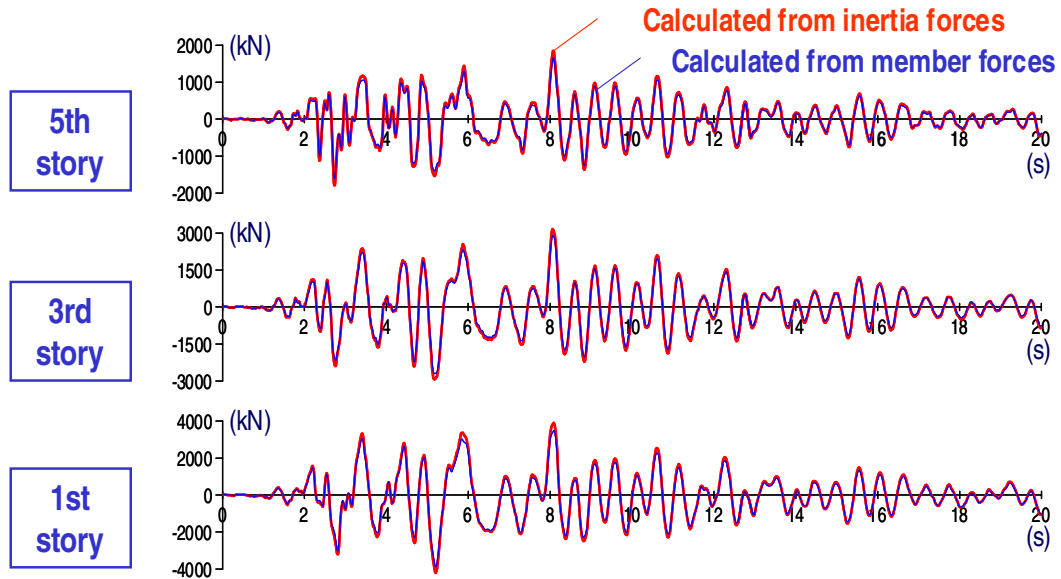
Figure 10.   Comparison between story shear based on inertia forces and story shear member forces
(with Steel dampers, Takatori 100%, X-Dir.)



(a) Steel damper



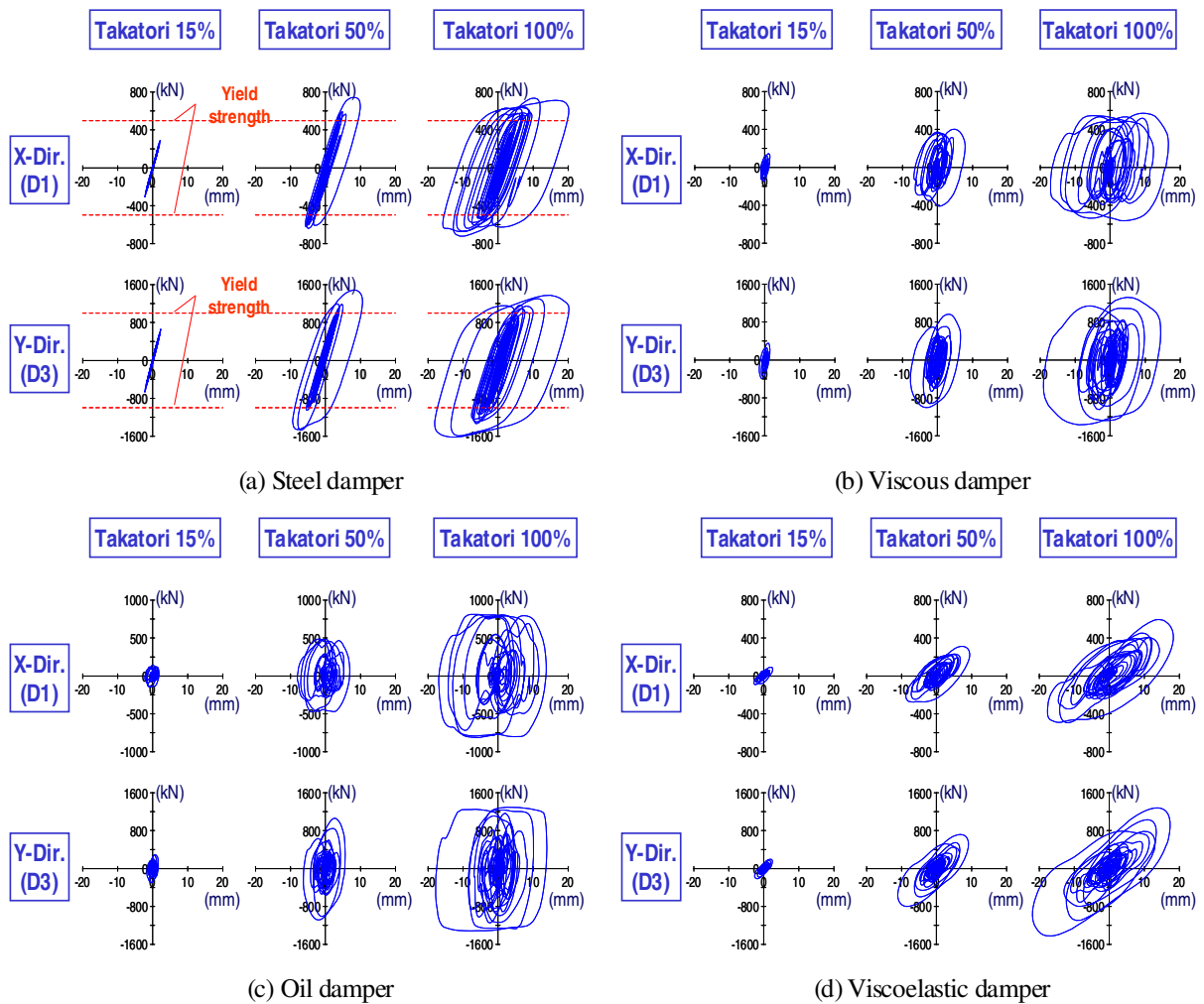(b) Viscous damper



(c) Oil damper



(d) Viscoelastic damper

Figure 11.   Relationship between axial damper forces and damper stroke of four types of dampers (1st story)

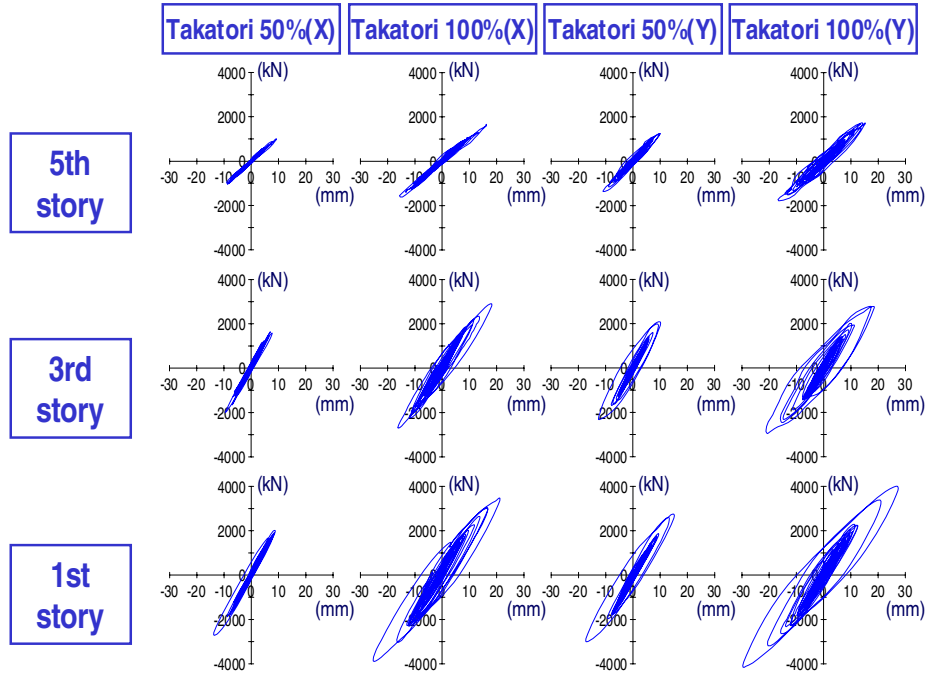Figure 12. Relationship between story shear based on inertia force and story drift (with steel dampers)



(a) Steel damper

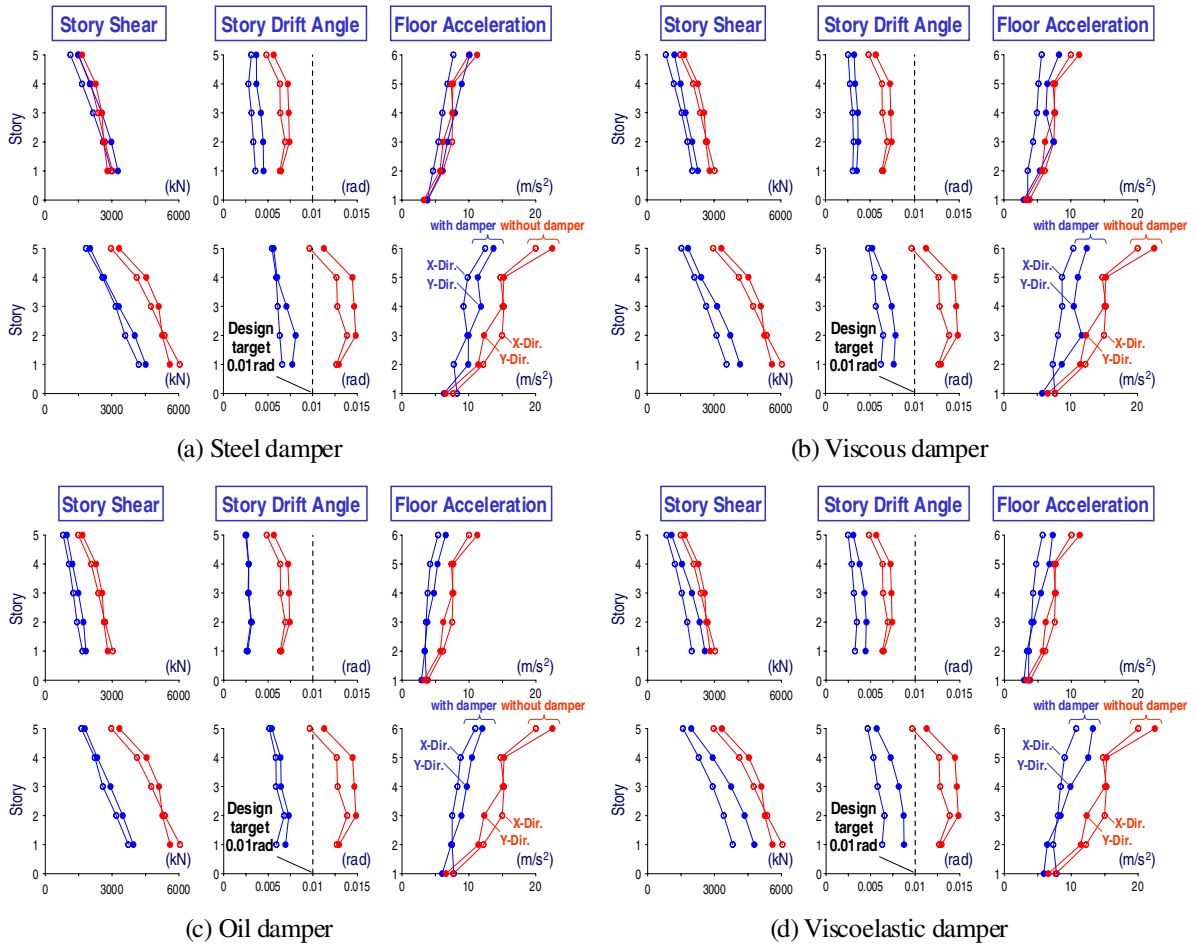(b) Viscous damper

(c) Oil damper

(d) Viscoelastic damper

Figure 13. Peak responses of building specimen with four types of dampers
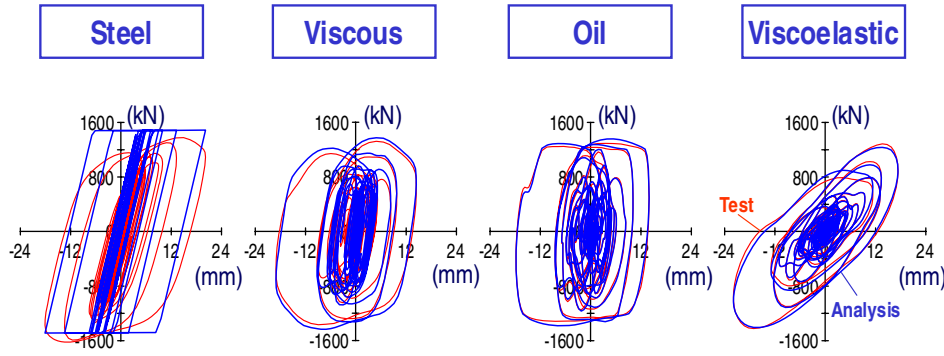
Figure 14.    Validation study for analysis model of four types of dampers (Takatori 100%)

## 5.  DYNAMIC PROPERTIES

As described above, the building was subjected many table motions, and changes in dynamic properties of the building were successfully monitored through the extensive measurement explained earlier.

Figure. 15 shows the cracks observed from the second floor concrete slab, prior to the tests, after the tests with steel dampers, viscous dampers, oil dampers, and viscoelastic dampers, and the tests without dampers, respectively. Cracks formed when the story drift reached approximately 0.5% rad. and significant increase in cracks was observed after the test with steel damper using 100% Takatori motion, as well as the test without dampers using 70% Takatori motion.
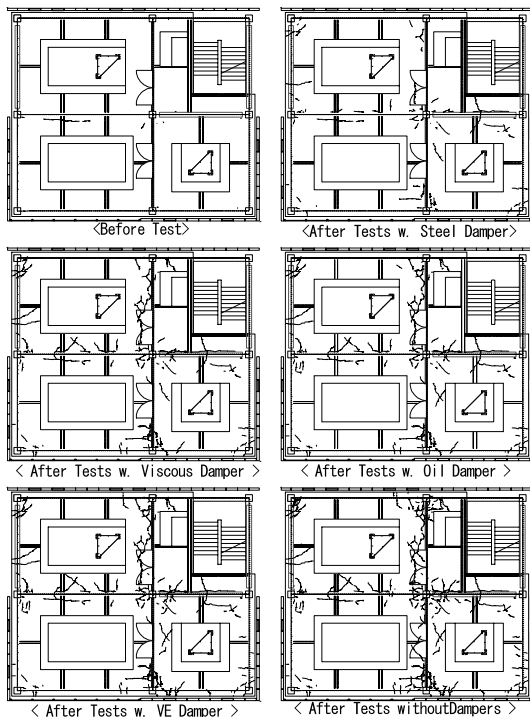


Figure 15    Cracks on second floor concrete slab

At each story level, story shear force is obtained from member forces (Fig. 10) excluding those of dampers, and it is compared with the story drift to obtain the story stiffness. Figure 16 shows change of thus-estimated story stiffness at each story level.   The frame had the largest stiffness before the test, and about 10 to 15% loss in stiffness occurred at the end of all the tests.   The cracks in the floor slabs noted above could be major source of such change.

Non-structural components are partially attached to the building.   As Figures 6 to 8 show, curtain walls and ALC (autoclaved light-weight concrete) panels are attached to the exterior surface of the buildings at the 1st and 2nd story level except where the dampers are installed.   The interior partitions with doors (e.g., Fig. 8) are also provided except for the 1st story level.   The stiffness contribution of these non-structural components were estimated by using the difference between the story shear forces obtained from inertia force and those from member forces (Fig. 10).

Figure 17 shows change of thus-estimated stiffness of the non-structural components at each story level.   The peak average story drift, the value obtained by averaging the drifts of all stories is also shown for every test conducted (Fig. 17 above), in order to indicate intensity of shaking.

It should be noted that the stiffness of nonstructural components is as much as 0.3 times that of the steel frame, at earlier tests.   Thereafter, the former deteriorates much more rapidly than the latter, and as much as 70% loss of the non-structural component is observed from Figure 17.   The loss is significant when large shaking took place.

Although not shown, energy dissipated by the non-structural components are also estimated.   As the smallest energy contribution, it was only 0.1 times the overall energy dissipation for the building with oil dampers that showed about 20% damping ratio.   However, the contribution increases for the building that indicated smaller damping ratio, using different dampers.   As the largest energy contribution, it was as much as 0.3 times the overall energy dissipation for the building without dampers.
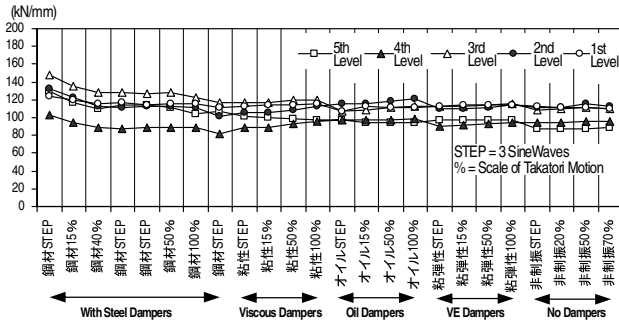
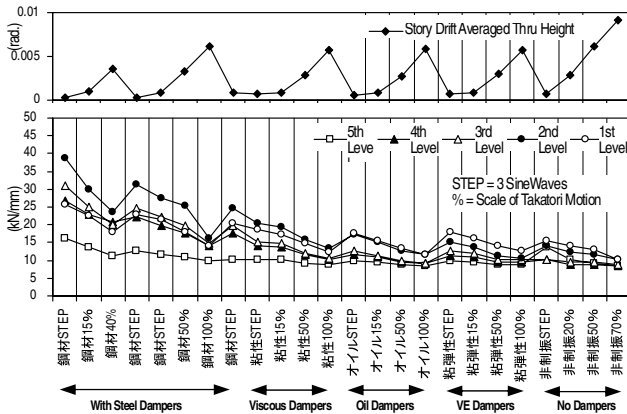Figure 16　Change in steel frame lateral stiffness
(x-direction)



Figure 17　Change in non-structural components'
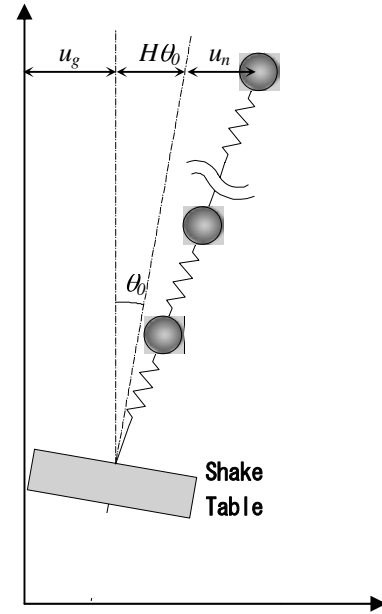lateral stiffness (x-direction)



Figure 18　Rocking of E-Defense shake table

the damper and the building, the linear identification method assuming real vibration mode still produces excellent results. Note however that the method showed less accuracy for the frame without dampers at the 70% Takatori motion, probably due to yielding of the members and connections of the building.

Vibration periods and damping ratios are also estimated from the responses recorded.   Several different tests and/or evaluation methods were employed.   In this paper, only the shake table tests will be referred to, and a basic system identification technique using real modes is used.   The control of the shake table considers interaction between the specimen and table, and it is used to calibrate shaking of the subsequent test.   The target translational acceleration histories along x-, y-, and z-axes are given, and target rotational acceleration with respect to the three axes are set zero.   However, due the shake table occasionally rocks with respect to the x- and y-axes (Fig. 18), especially at the earlier shaking where little calibration is done.

Significant error will develop when one considers the transfer function between only the shake table horizontal acceleration and building response acceleration.   It is necessary to decompose the response acceleration into the acceleration due to rigid-body rotation caused by table rocking and acceleration relative to the shake table surface. A technique to consider this was developed by the writers. It will be explained elsewhere and only the results will be shown in this paper.

Figure 19 shows the recorded roof displacement of the building with or without dampers.   Prediction using the writers' system identification technique is also shown. Remarkable correration is obtained between the prediction and test result.   In spite of different nonlinear behavior of
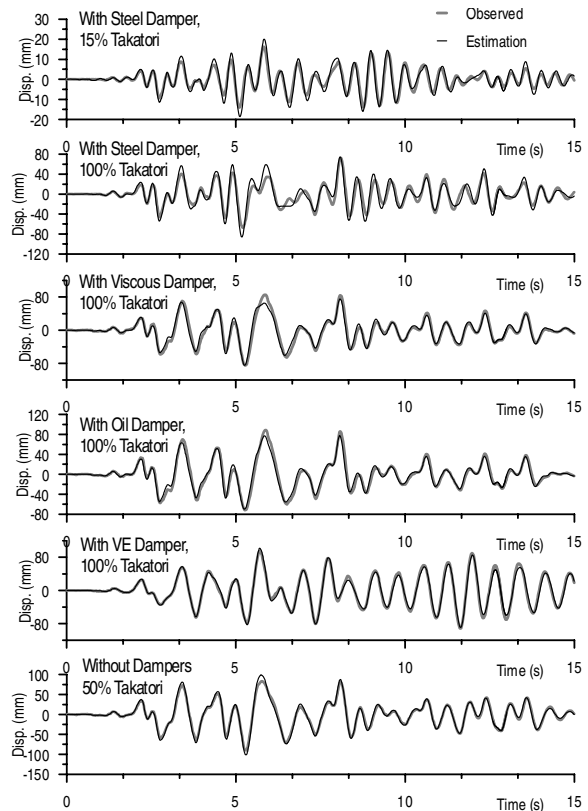


Figure 19　Roof displacements: test vs. prediction using
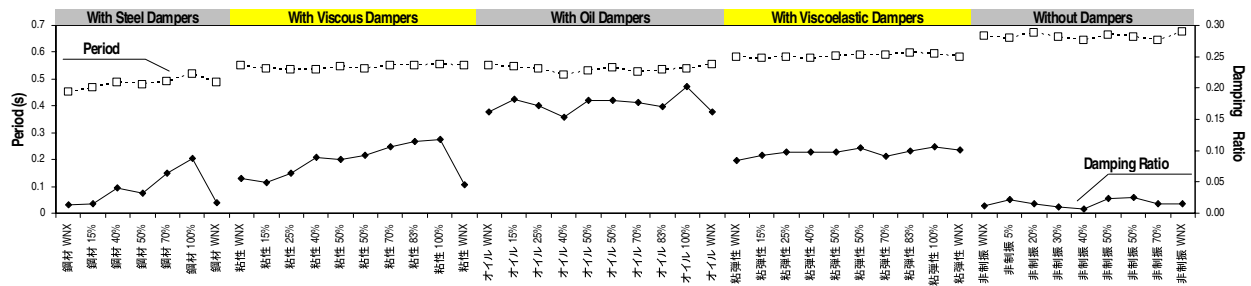system ID and modal superposition

Figure 20    Vibration periods and damping ratios in x-direction

Figure 20 shows the vibration periods and damping ratios obtained for all the tests, using the above-mentioned technique. The building with steel dampers shows shortest period and smallest damping ratio among those with dampers, and they become longer and larger, respectively, at larger shaking. The building with oil dampers shows the largest damping ratio of about 17%, and the smallest responses (Fig. 13). Note that the oil dampers were somewhat over-sized compared with other types dampers, which is one of the reasons for the above trend. The building with either viscous dampers or viscoelastic dampers shows damping ratios of about 10%, and performed well.

# 6.    SUMMARIES AND CONCLUSIONS

Realistic three-dimensional shaking table tests were conducted for full-scale 5-story building specimens with (March 2009) or without (April 2009) dampers. This paper has described the test concept, method and test results, as well as details of the 5-story building specimen.

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