The Structure Health Monitoring for Identifying the Damping Performance of Building Structures under Strong Ground Motion on March 11 in 2011

I. Nishimura
Dep. of Architecture, Tokyo City University, Japan

SUMMARY:
This paper reports the recorded data obtained from one of the facilities on the campus of Tokyo City University, on March 11 in 2011. The author started monitoring the response of the target building structure with a purpose of identifying the damping performance since 2005. There have been accumulated enough data to estimate the damping factor and natural frequency of the building structure with nonlinear damping devices installed. When Tohoku district experienced the major earthquake M9.0 on March 11 in 2011, the monitoring system started recording the response of the structure as well as the ground motion. We have several other data obtained on the occasion of post quakes ever since then. These data enabled us to compare the damping performance of the structure before the event with what was observed after the event. The damping system was proved to be effective during the major quake by comparing those data. In the end we have certified the building structure was kept intact when it experienced the major quake on March 11, 2011.

Keywords: Damage detection, Nonlinear damping, Identification, Earthquake response, Structure health monitoring

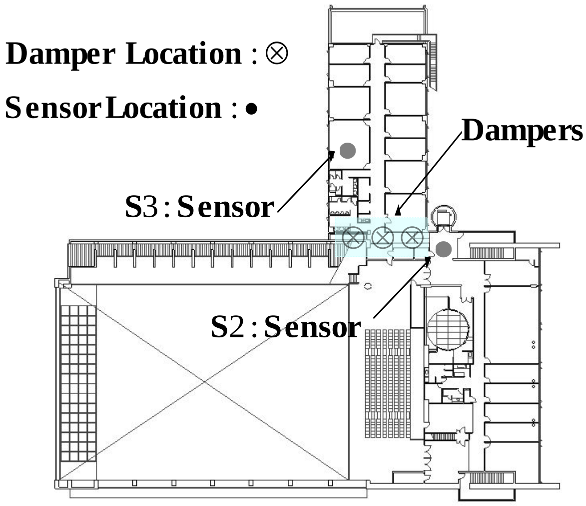
1. THE BASIC DATA OF THE APPLICATION PROJECT (REVIEW OF THE PAST STUDY)

1.1. The building structure property

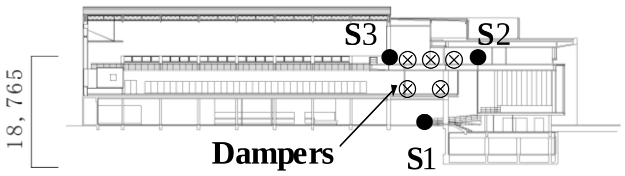
The author's research team installed the structure monitoring system into the complex building on the campus of Tokyo City University in April, 2005. It is basically composed of two different building structures: the student gymnasium and the annex office building structure. We designed the nonlinear damping devices installed between the two buildings so that the damping performance of the student gymnasium is highly increased. The general information of the project was published (Nishimura, 4th WCSC, 2006) and briefly reviewed in this section.

There are shown the ground, second and third plans of the complex structure as well as the north and south elevations in Figure.1.1. There are also indicated the locations of the damper devices in the FEM model. The area and weight of the gymnasium and the annex office building are shown in Table 1.1.

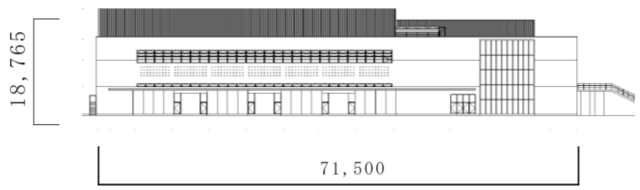
The first floor is used for student dining hall and the second floor is for gymnasium. As a result of FEM frame analysis, we came to know that the annex structure is much stiffer than the gymnasium entrance frame in EW direction. The mathematical model for dynamic study is based on those data. There are enough shear walls in the NS direction in the main building structure. On the other hand the third floor is supported by rather slender columns without major shear walls in EW direction. The horizontal forces in EW direction should be transferred to the annex office building when earthquakes happen. We decided to use oil dampers to connect two building structures in EW direction because of this reason. The simple mathematical model representing the dynamic behaviour in EW direction is shown in Figure 1.2. The effective mass and stiffness of the gymnasium and annex office building are evaluated with respect to the connection point (Table 1.1.) and given in Table 1.1, and the identified equivalent single degree of freedom model is given in Figure 1.2.



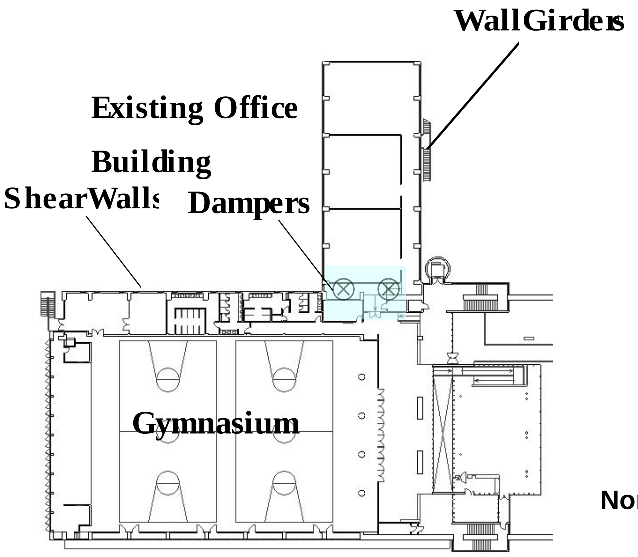
3rd Floor Plan



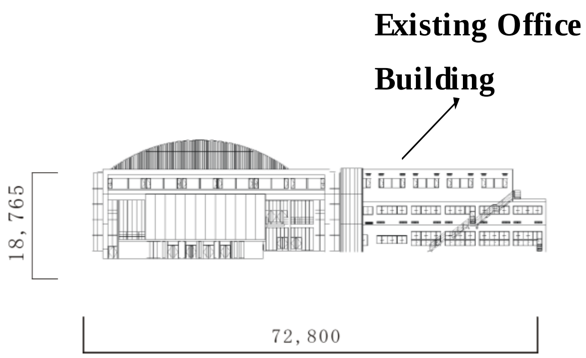
North-South Section



East Side Elevation

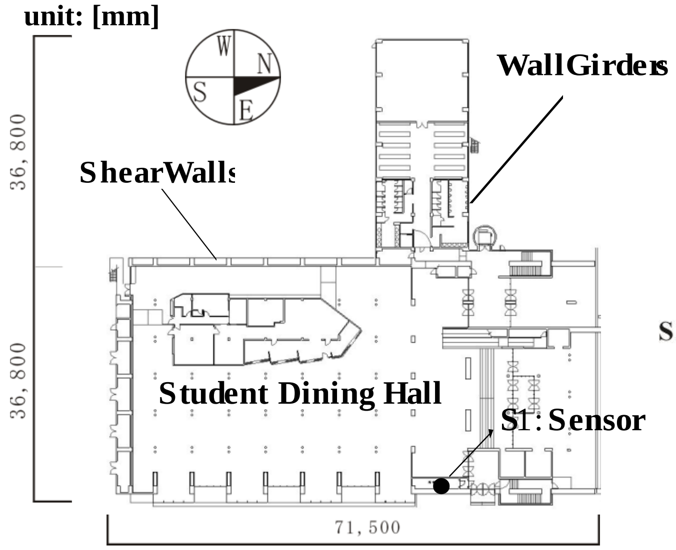


2nd Floor Plan

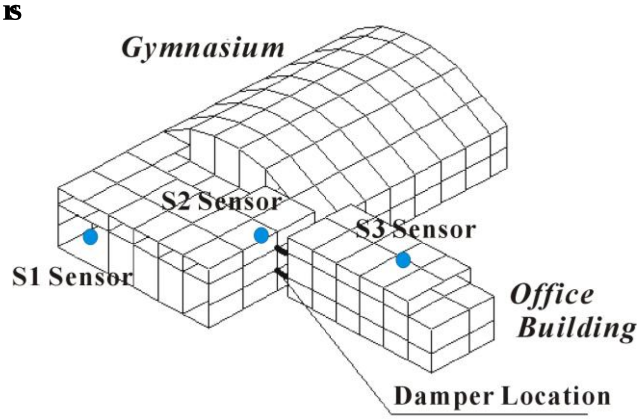


North Side Elevation (Entrance Side View)

unit: [mm]



1st Floor Plan



FEM Model for Eigen Value Analysis

Skeleton View of the Gymnasium Frames

Figure 1.1 Sensor Locations and Damper Locations

Table 1.1 Parameters of Linear Connection Damper and Compressive Damper

System 1	$\boldsymbol{\omega}_1 = 25.2$ rad/sec; $\boldsymbol{k}_1 = 3000$ KN/mm; $\boldsymbol{m}_1 = 4.7 \times 10^6$ kg
System 2	$\boldsymbol{\omega}_2 = 70.0$ rad/sec; $\boldsymbol{k}_2 = 3000$ KN/mm; $\boldsymbol{m}_2 = 6.0 \times 10^5$ kg
Damper parameters	$\boldsymbol{k_d} = 800$ KN/mm; $\boldsymbol{\omega}_\infty = 28.2$rad/s; $\boldsymbol{\omega}_{opt} = 26.7$ rad/s; $\boldsymbol{k}_{opt} = 400$KN/mm
Linear Damper	$\boldsymbol{c}_{opt} = 32.0$ KN sec/mm; $\eta_{eq} = 0.055$
Compressive Damper	$\boldsymbol{c}_N = 60$ KN sec/mm (For compression only.); $\boldsymbol{k}_d = 800$ KN/mm

Table 1.2 Basic Dynamic Properties of Gymnasium and Office Building

Gymnasium Properties (Preliminary Study)	4th FL. 104.9m^2, 3rd FL. 1732.9 m^2 2nd FL. 2725.8m^2, 1st FL. 2633.3 m^2	Effective mass at 3rd FL. $\boldsymbol{m}_1 = 4.7 \times 10^6$ kg Frequency $\boldsymbol{\omega}_1 = 25.2$ rad/sec
Office Properties (Preliminary Study)	3rd FL. 430.9 m^2, 2nd FL. 518.3 m^2 1st FL. 518.3 m^2	Effective mass at 3rd FL. $\boldsymbol{m}_2 = 6.0 \times 10^5$ kg Frequency $\omega_2 = 70.0$ rad/sec
Damper Specification	Damper stiffness 200KN/mm per one damper. Damping coefficient 15 KN sec/mm per one damper.	

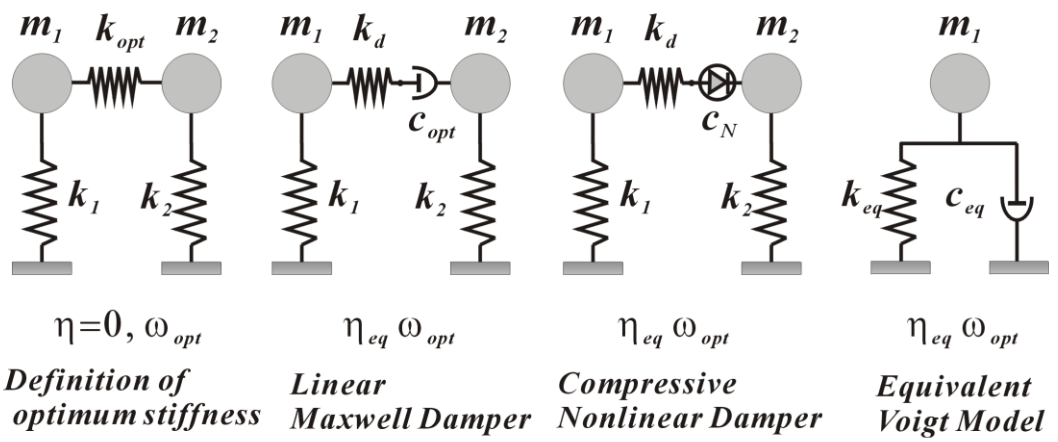


Figure 1.2 Mathematical model for the nonlinear damper devices in EW direction

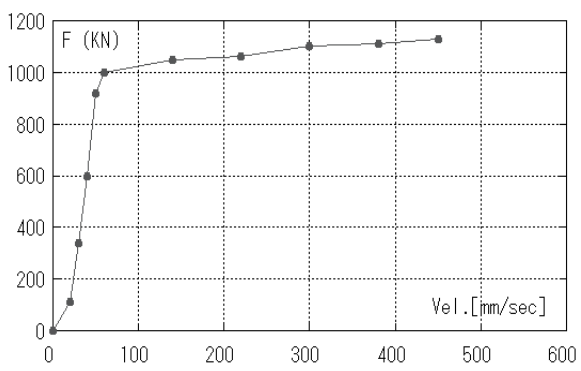


Figure 1.3 Test results of damper coefficient

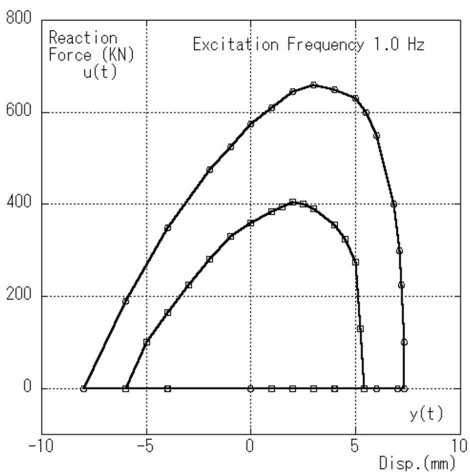


Figure 1.4 Cyclic loading test results

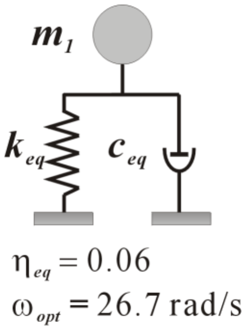


Figure 1.5 Equivalent Voigt Model **Photo 1.1 Accelerometers** **Photo 1.2 Network data logger**

1.2. The dynamic property of the nonlinear dampers

We used a simple model in Figure 1.2 for designing the damper specification. Even though a better performance is roughly expected, yet careful selection of damping coefficient is still the key factor for achieving the best performance of the devices. According to the preliminary study, the appropriate compressive damping coefficient for one device should be around 15KN sec/mm, which is certified in Figure 1.3 that shows the dynamic test result before shipping and installation. The compressive damper characteristic is clearly seen in Figure 1.4.

1.3. Data acquisition system

The purpose of the structure health monitoring that the author adopted for this project is to evaluate the performance of building structures with compressive dampers. Special attention is paid to nonlinearity of the damping devices, because preliminary study showed that local nonlinearity would disappear naturally but change the global dynamics in an average sense. It is extremely difficult even impossible to identify the mathematical model that could explain cause and effect, but it would be much easier to create a mathematical model that could explain the observed phenomena. Both of them have clearly different objectives and purposes. In this project, the author started observing the earthquake response dynamics of the structure to create a model that could explain the phenomena not the cause-and-effect. The local area network sensor system is shown in photo 1.1 and Photo 1.2. It is composed of two accelerometers, 16-bit A/D converter with data logger PC, and another PC that works as a gateway computer with global IP address. (See Figure 1.6 and 1.7) There are three Local Area Network sensor systems (LAN) working individually so that none of them are synchronized and each one of them starts data acquisition according to its own trigger level.

1.4. Earthquake record of July 23 in 2005 and August 18 in 2005.

There were two noticeable events shortly after the author research team started observation using the network sensors. One of them occurred on 23 July 2005, whose epicenter is located about 50km east of Tokyo and recorded magnitude 6.0 and intensity 5 according to JMA scale. Another one occurred about 300km away from Tokyo on 16 August 2005 with magnitude 7.2 and JMA intensity 4. The observed acceleration data on the ground floor are shown in Figure 1.12, and Figure 1.13, respectively. The acquired response acceleration histories on the 3rd floor are shown in Figure 1.8 and Figure 1.9, respectively. There are also shown transfer functions of 3rd floor from ground floor for each event. Even though the intensity levels are different, there is noticed little difference between the two transfer functions. Using the observed ground data and a simple Voigt Model shown in Figure 1.5, we can simulate the 3rd floor acceleration responses shown in Figure 1.10 and Figure 1.11. The participation coefficient factor at the 3rd floor with respect to the first mode is 1.10 according to the FME analysis. The damping factor in Figure 1.5 is determined by considering the participation coefficient 1.10.

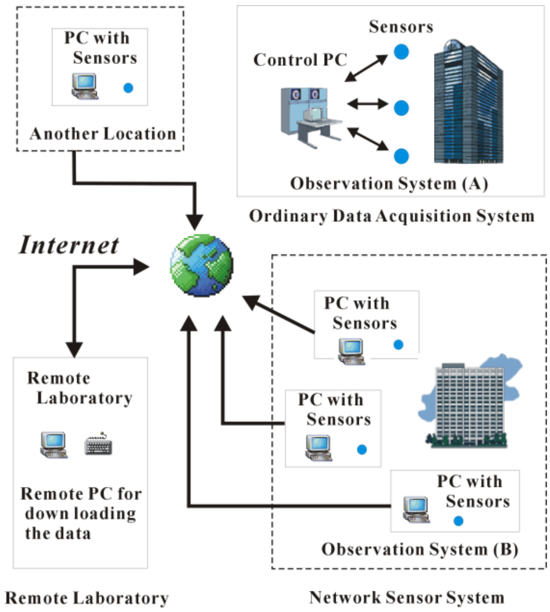


Figure 1.6 Global network sensor system

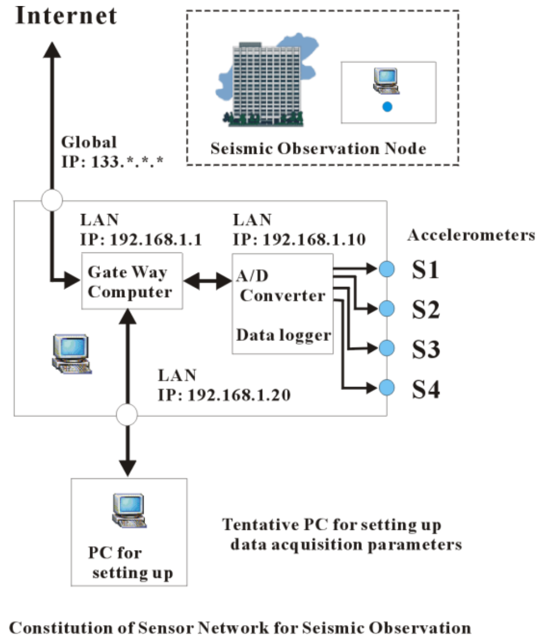


Figure 1.7 Local network sensor system

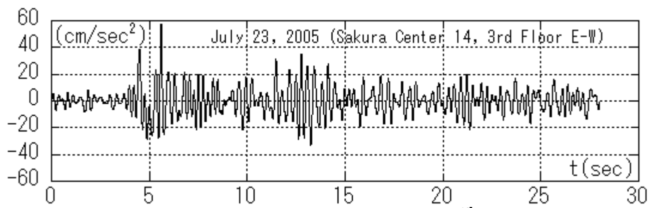


Figure 1.8 Observed Acc. on the 3^{rd} FL

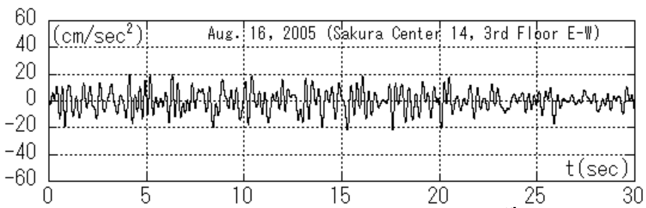


Figure 1.9 Observed Acc. on the 3^{rd} FL

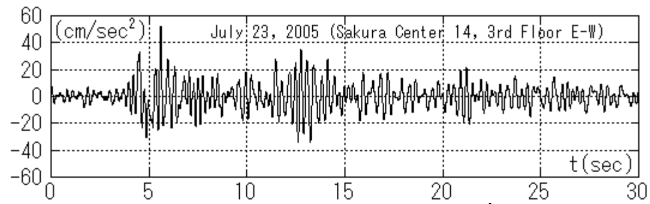


Figure 1.10 Simulated Acc. on the 3^{rd} FL

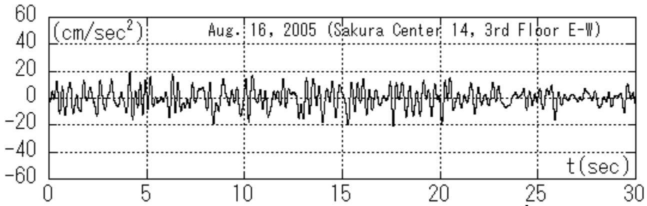


Figure 1.11 Simulated Acc. on the 3^{rd} FL

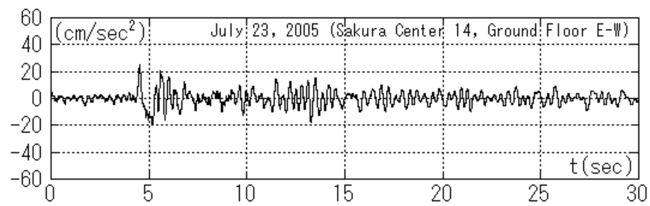


Figure 1.12 Observed Acc. on the Ground FL

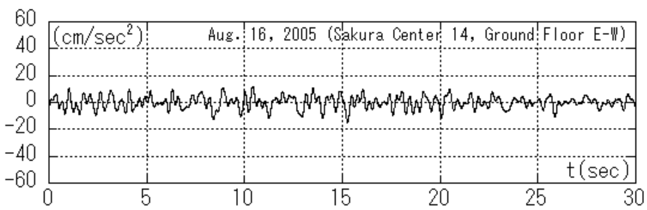


Figure 1.13 Observed Acc. on the Ground FL

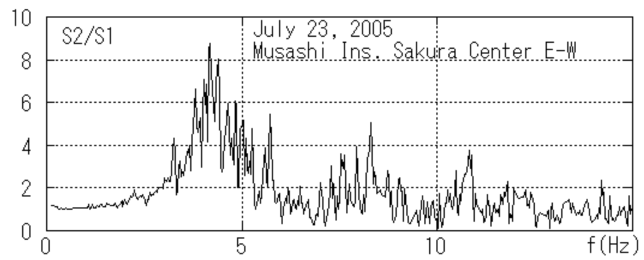


Figure 1.14 Transfer Function (3FL/GL)

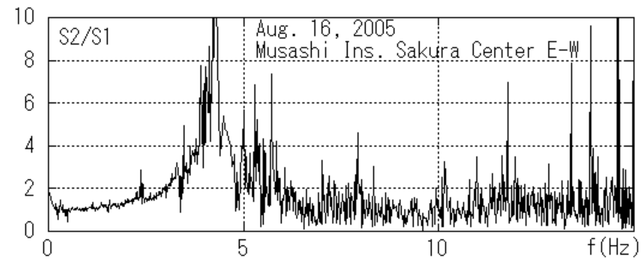


Figure 1.15 Transfer Function (3FL/GL)

Data recorded on July 23, 2005 (E-W)

Data recorded on August 16, 2005 (E-W)

2. DATA IN THE EVENT OF THE TOHOKU EARTHQUAKE IN 2011

2.1. General information of the Tohoku Earthquake on the March 11 in 2011

The epicentre of the event is about 125km offshore of the coastline of Tohoku district in northern part of Japan. The location of the epicentre is 38.1N and 142.9E. The campus of Tokyo City University is located at 35.60N and 139.65E. They are roughly 400km away from each other. The recorded acceleration history on the ground floor of the gymnasium in EW and NS direction is shown in Figure 2.1 and Figure 2.2, respectively. The response acceleration spectrum of those data is shown in Figure 2.3 and Figure 2.4. The velocity spectrum shows a flat spectrum over a wide range of period in Figure 2.5 and Figure 2.6. It is quite interesting that the EW ground motion seems to be attenuated in the high frequency range as compared with the NS ground motion. The damping system in the structure seems to reduce the intensity over the high frequency zone in EW direction, when we compare the acceleration spectrum of EW direction and with the acceleration spectrum in NS direction.

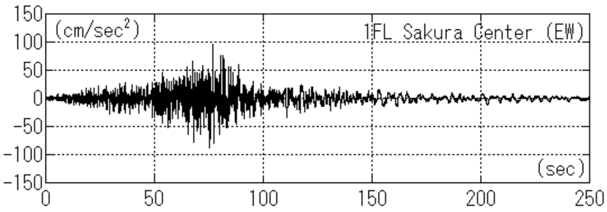


Figure 2.1 Observed Acc. on the Ground FL (EW)

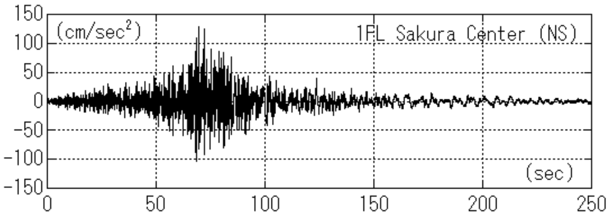


Figure 2.2 Observed Acc. on the Ground FL (NS)

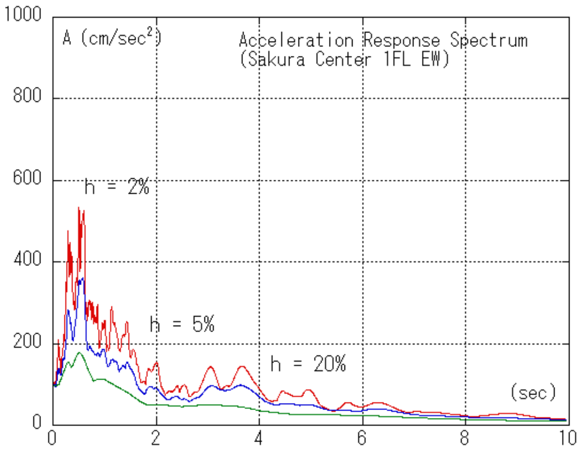


Figure 2.3 Acceleration Response Spectrum (EW)

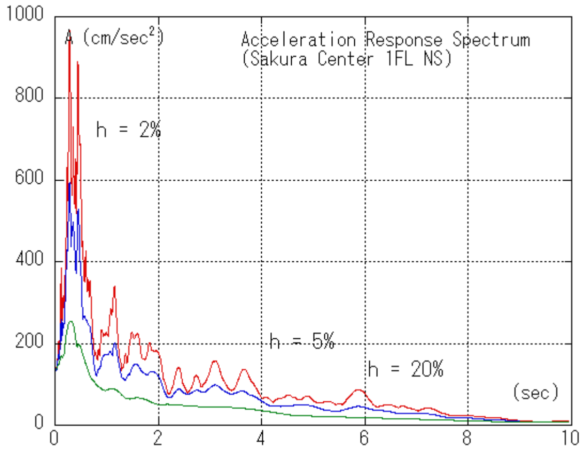


Figure 2.4 Acceleration Response Spectrum (NS)

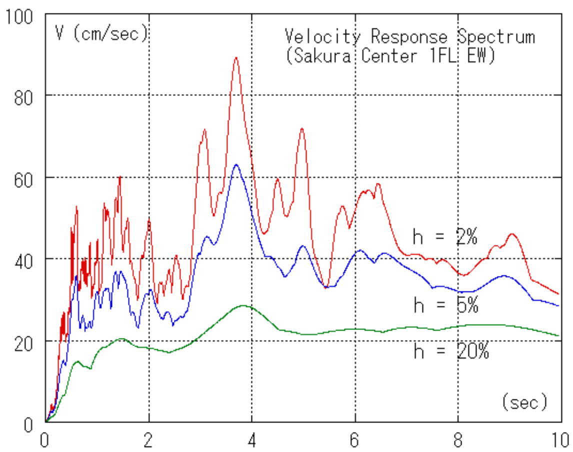


Figure 2.5 Velocity Response Spectrum (EW)

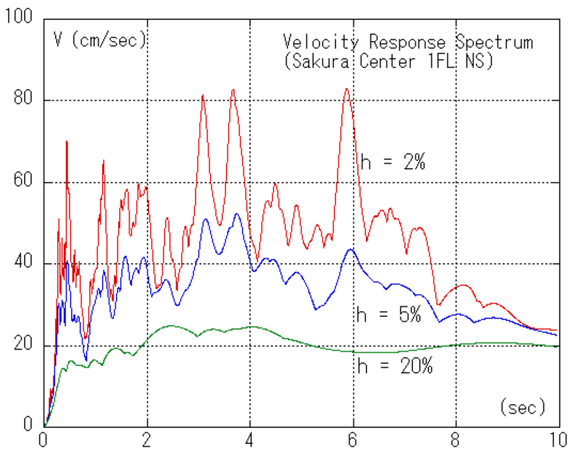


Figure 2.6 Velocity Response Spectrum (NS)

2.2. Response of the structure on March 11, 2011

The response data on the third floor of the structure are shown in Figure 2.7 and Figure 2.8. The transfer functions of the third floor from the ground floor are shown in Figure 2.9 and Figure 2.10 for east-west direction and north-south direction, respectively. According to the previous study the participation coefficient is 1.10 so that the damping factor judging from Figure 2.9 is 6.0 % and the first modal frequency is 3.9 Hz, where the phase lag between the ground motion and the 3rd floor response is 90 degree in Figure 2.11, which is equal to the dynamic properties identified in the previous events. There is no significant difference between Figure 1.14 and Figure 2.9 so that the damping performance of the system in EW direction was achieved as much as before. The damping factor identified in the previous study was 6.0 %, which is actually certified during the event on March 11, 2011.

The transfer function in NS direction in Figure 2.10 does not show any noticeable peak that can be identified as the first modal frequency. As a matter of fact, the complex structure seems to be moving as a rigid body in NS direction and response of the 3rd floor in Figure 2.8 is even smaller than the input ground motion in Figure 2.2. The author could not create a simple model for explaining the observed data yet, however, the monitoring system can be used for judging whether the system experienced severe damage or not. Because there is no clear amplification over the frequency range less than 5Hz in Figure 2.10. In this case, the input energy from the ground to the structure in NS direction is extremely small because there is small phase lag between the ground motion and the floor response in Figure 2.12.

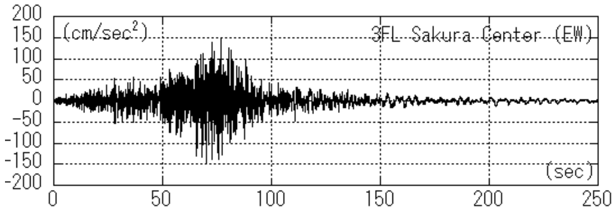


Figure 2.7 Observed Acc. On the 3rd FL (EW)

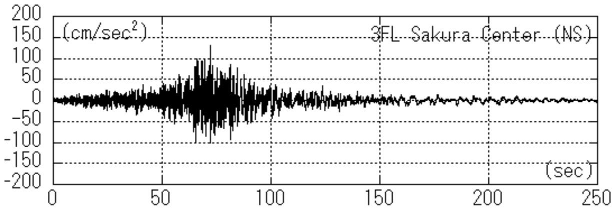


Figure 2.8 Observed Acc. On the 3rd FL (NS)

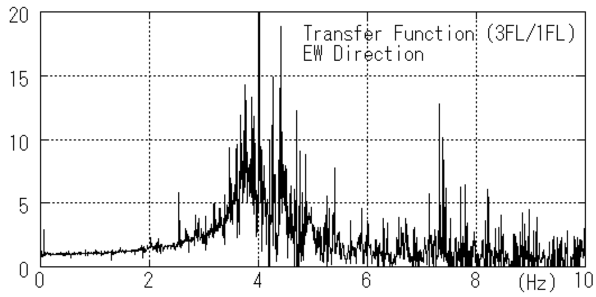


Figure 2.9 Amplitude transfer function of the 3rd FL from Ground FL (EW)

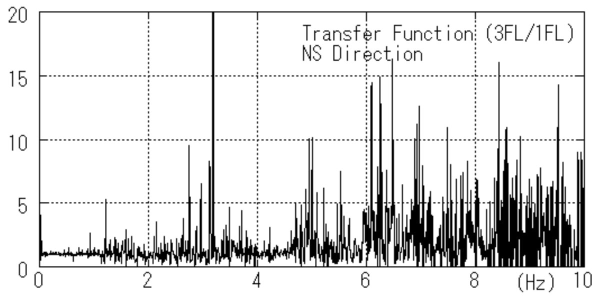


Figure 2.10 Amplitude transfer function of the 3rd FL from Ground FL (EW)

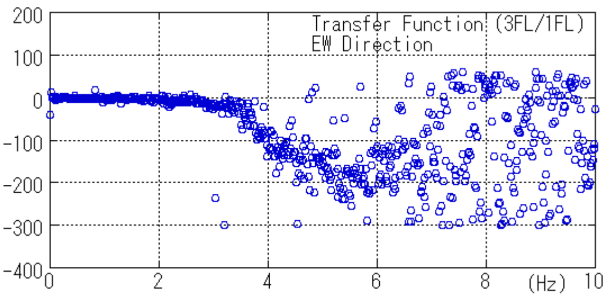


Figure 2.11 Phase transfer function of the 3rd FL from Ground FL (EW)

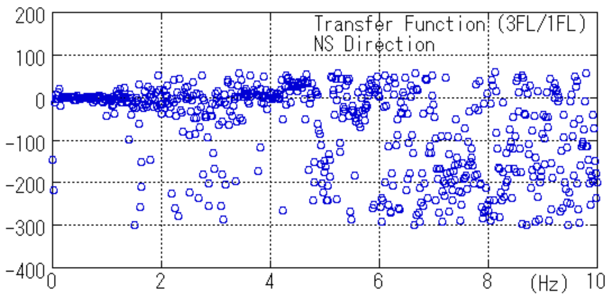
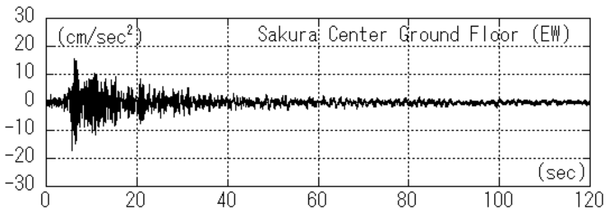


Figure 2.12 Phase transfer function of the 3rd FL from Ground FL (EW)

3. DATA OBTAINED AFTER MARCH 11 IN 2011

3.1 Data of the event on April 16 in 2011

One of the many events after March 11 is picked up and shown in Figure 2.13 and Figure 2.14. The epicentre of this event is 36.4N 140.0E and is 300km away from the facility on the university campus. The Acceleration spectrum and Velocity response spectrum are shown in Figure 2.15 to Figure 2.18. The magnitude of the event is M 5.9 and intensity in the vicinity area is 4 according to JMA scale. It is quite interesting that the response spectrum in EW direction is relatively smaller than the counterpart in NS direction. These phenomena are commonly observed in most cases. We must admit this is not a particular characteristic of the event but a general tendency associated with the system dynamics. As a result we must admit that this phenomenon is associated with the structure dynamics rather than earthquake characteristics.



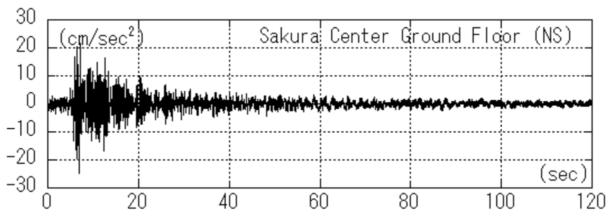
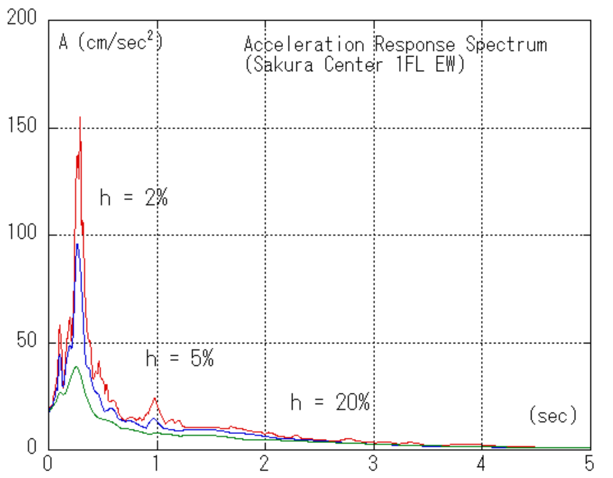


Figure 2.13 Observed Acc. on the Ground FL (EW) **Figure 2.14 Observed Acc. on the Ground FL (NS)**



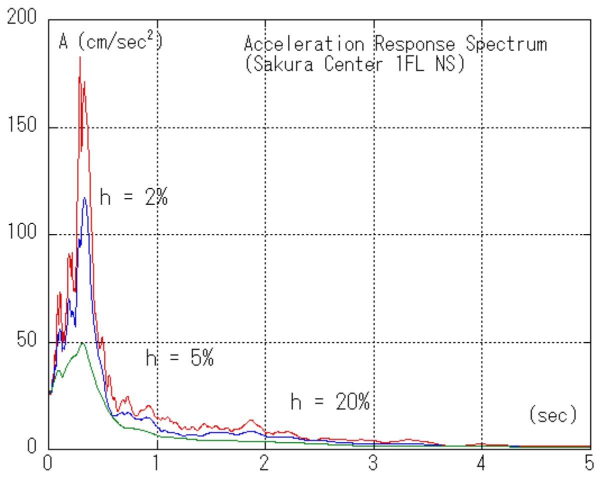
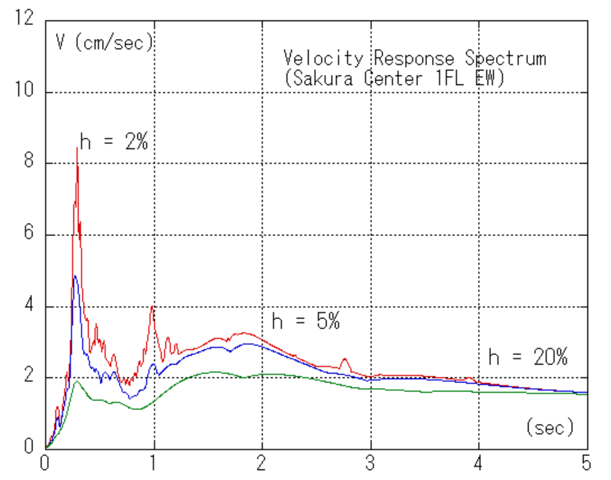


Figure 2.15 Acceleration Response Spectrum (EW) **Figure 2.16 Acceleration Response Spectrum (NS)**



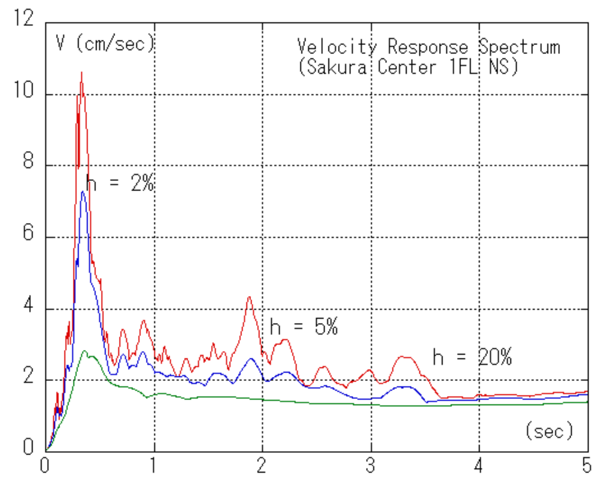


Figure 2.17 Velocity Response Spectrum (EW) **Figure 2.18 Velocity Response Spectrum (NS)**

3.2 Response of the structure on April 16 in 2011

The acceleration responses on the third floor in EW direction and NS direction are shown in Figure 2.19 and Figure 2.20, respectively. The transfer function of floor response from ground motion is shown in Figure 2.21 and Figure 2.22. There is no clear difference between the previously identified system dynamic in 2005 and the other one in April 16 in 2011. It is true that the first modal frequency seems to have shifted from 4.2 Hz to 3.9 Hz but the damping factor is kept 6.0%. Judging from the transfer functions we concluded there is no noticeable damage caused by the event in March 11 in 2011. Long term observation enabled us to certify the procedure for damage detection based on system identification method. As is suggested by many researchers so far, the transfer function can be used as a good index to certify whether structure system is kept in good condition, even if it does not exactly tell us where the damage occurred.

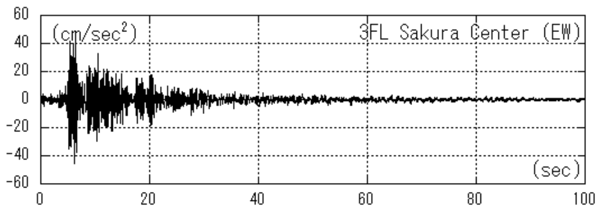


Figure 2.19 Observed Acc. on the 3rd FL (EW)

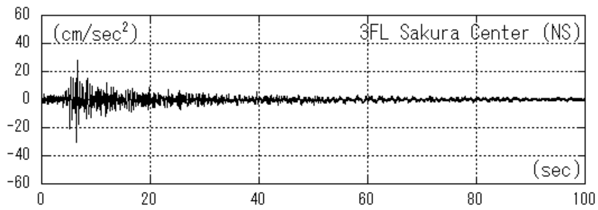


Figure 2.20 Observed Acc. on the 3rd FL (NS)

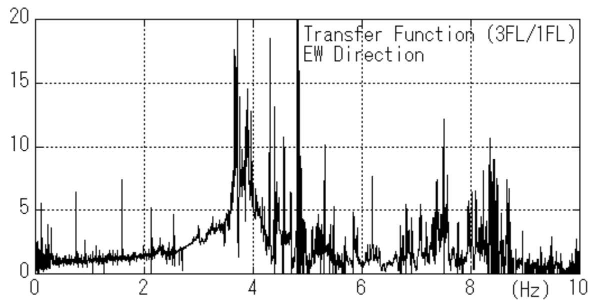


Figure 2.21 Amplitude Transfer function of 3rd FL/GL (EW)

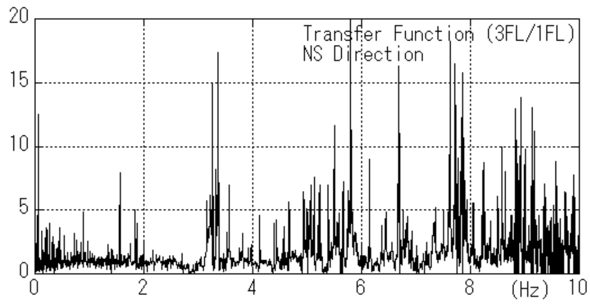


Figure 2.22 Amplitude Transfer function of 3rd FL/GL (NS)

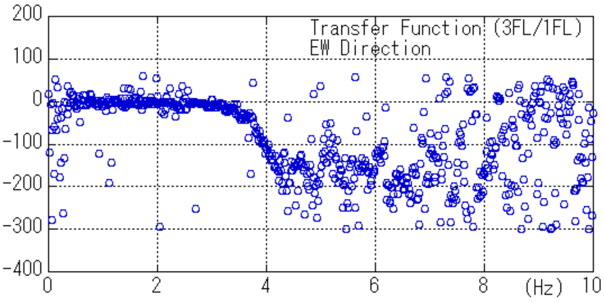


Figure 2.23 Phase transfer function of 3rd FL/GL (EW)

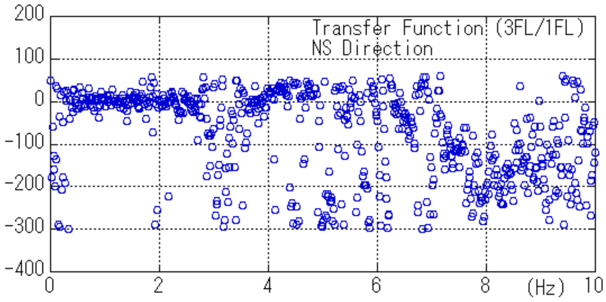


Figure 2.24 Phase transfer function of 3rd FL/GL (NS)

3.3 Comparison between EW ground motion and NS ground motion

The acceleration transfer function in EW direction is smaller than the counterpart in NS direction. There are many observation data after March 11 in 2011, but there is no exception for this phenomenon. The author believes that the damping augmentation of the structure system dynamics in EW direction attenuated the ground motion in the same direction. If this assumption is true, the damping device installation not only reduces the structure response but also attenuates the input disturbances. Further study is necessary for qualitative conclusion, but long term observation can only reveal the unexpected effect of the damping augmentation of superstructure on the soil condition.

CONCLUSIONS

The nonlinear compressive damping devices was invented and applied to a complex building structure, which is used as a student dining hall and gymnasium. The structure health monitoring system or network sensor system was implemented with the facility and successfully recorded the structure response vibrations on occasion of earthquake events since 2005. The author's laboratory identified the system dynamics of the structure prior to the major event of earthquake in March 11 in 2011. The health monitoring system succeeded in recording the response vibration on the third floor level and ground floor level during the event. We compared those data obtained before the major event and after the event and came to conclusion that the installed damping devices worked as much as expected and kept the structure intact in case of a major earthquake with intensity 5 according to JMS. This whole project took about 7 years starting from design in 2004 to post analysis in 2012. Although the installation of compressive damping devices has been proved effective as much as ordinary linear dampers and the stiffness associated with damper installation rather than damper connection stiffness is of more importance, yet it still is difficult to evaluate the attenuation effect of structural damping factor on the reduction of ground motion in case of major earthquake. The author wishes to share this valuable data and experiences with other engineers in the same field of science to mitigate the earthquake disasters.

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