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Damage Prediction and Retrofit Plan Using Dampers for a High-Rise Steel Building in Tokyo Based on Its Response Records

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Abstract

The Great Tohoku-oki earthquake of March 11, 2011 (the '311' earthquake) showed that the intensity of ground shaking for some site location has been as large or larger than the maximum levels (450 years reoccurrence period) considered in the current design of typical structures in Japan. In addition, the ground motion in the Tokyo Metropolitan showed extremely long durations (around 10 min.). Structural engineers and researchers have consequently recognized that the structural performance should be improved to match the potential of more intensive shaking with long durations in the future. Seismologists expected that the Tokyo metropolitan area will experience as large as three to four times the shaking of the 311 earthquake in near future, such as M7-class earthquakes under the City of Tokyo (the 'Shuto-Chokka' earthquake) and/or a M8/M9-class earthquake along the Nankai-trough (the 'Nankai-Trough' eathquake). In such cases, high-rise buildings, particularly those without protective systems, are supposed to suffer significant structural damage. Accordingly, the comprehensive seismic performance reevaluation and corresponding retrofitting on existing high-rise buildings have become the most critical issues in current structural engineering in Japan.

In this study, a 29-story steel building is analyzed, which is a university campus building, and experienced great shaking during the 311 earthquake. The building structure is a conventional moment resisting frame, and characterized by long-span with many braces of non-ductile performance. First, a detailed member-to-member frame model of the building is constructed, and the validity of the model is checked by comparing the simulated results with the observed records of the 311 earthquake. Next, the nonlinear structural analysis is conducted and its structural damage is estimated using large input ground motions for the hypothetical Shuto-Chokka and Nanka-Trough earthquakes. Finally, a new retrofit design methodology to optimally determine the location of dampers will be presented, and the effect and efficiency is confirmed.

Keywords: High-Rise Building Retrofit; Damper; the 2016 Great Tohoku-oki earthquake; nonlinear structural analysis

1. Introduction

Japan was hit by the great Tohoku-oki earthquake of magnitude 9.0 on March 11, 2011 (the '311' earthquake), and many tall buildings constructed in the last 40 years in Tokyo metropolitan experienced shaking much stronger than those in the past. High-rise buildings typically suffered severe damage on nonstructural components and facilities including ceilings, copy machines and elevator cables etc., but only very slight damage of structural components was observed [1]. Owing to the monitoring system instrumented in high-rise buildings, the performance of buildings are recorded during the shaking, which has been advantageously used for reevaluation of structural seismic performance afterward.

The ground motion records in the Tokyo metropolitan area during the earthquake showed extremely long durations (about 10 min), because of the excitation of the long-period surface waves in the Kanto sedimentary basin. It is expected that Tokyo metropolitan area will experience as large as three to four times the shaking of the 311 earthquake in near future, because of M7-class earthquakes under the City of Tokyo (the 'Shuto-Chokka' earthquake) and/or a M8/M9-class earthquake along the Nankai-trough (the 'Nankai-Trough' eathquake). In such cases, high-rise buildings are probably supposed to suffer significant structural damage.

In this study, we evaluate the structural damage of a 29-story steel building for large earthquakes, and propose retrofit plans using dampers. The building is the campus building of Kogakuin University in Shinjuku (the downtown Tokyo), and experienced great shaking during the 311 earthquake. First, we construct a detailed member-to-member frame model of the building, and check the validity of the model by comparing the simulated results with the observed records of the 311 earthquake. Next, we estimate the building damage for the large input ground motions from the Shuto-Chokka and Nankai-Trough earthquakes, by conducting nonlinear structural analysis. Finally, we propose a new retrofit design methodology to optimally determine the location of dampers, and discuss the effect and efficiency will be discussed in detail.

2. Structural Model and of the Kogakuin University Building and Input Ground Motions

We show the structural model of the high-rise building of the Kogakuin University, compare the simulation results of seismic response with the observed records for the '311' earthquake. And, then, we show the input ground motions of the Shuto-Chokka and Nankai-Trough earthquakes, which we use for the damage estimation and a corresponding effective retrofit plan.

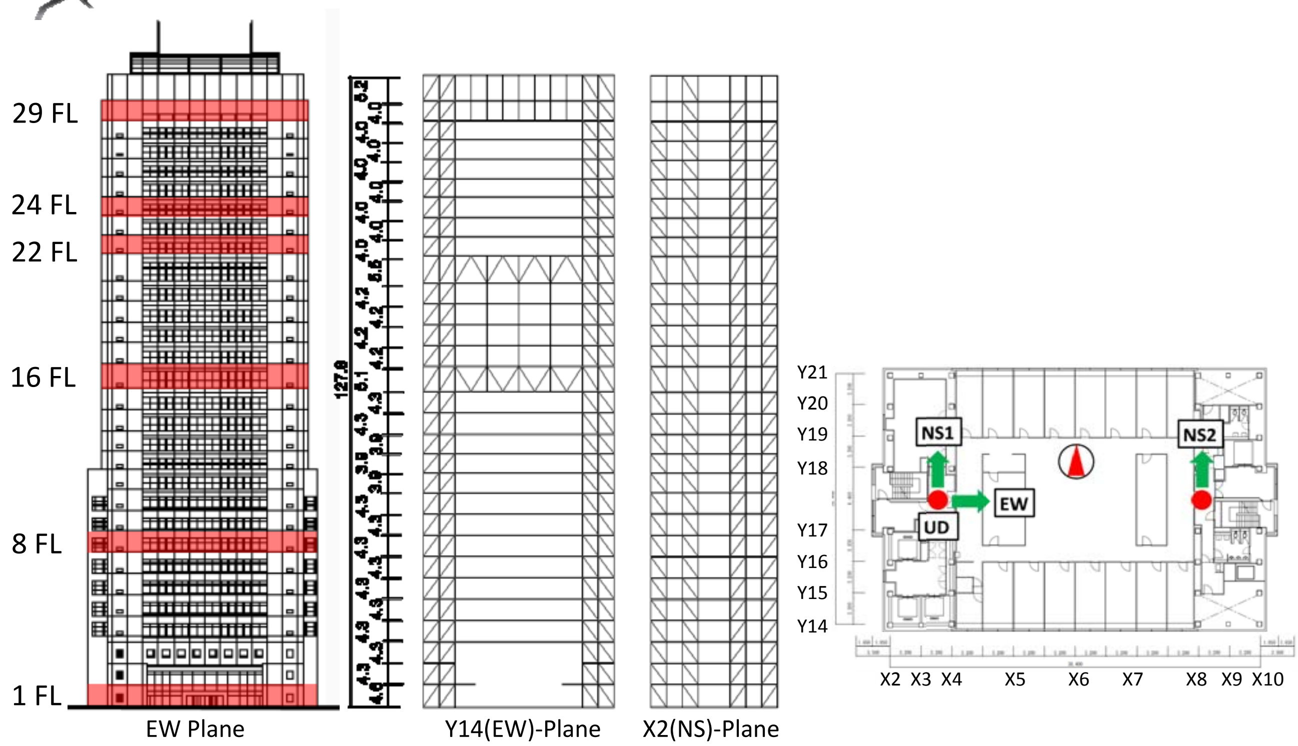
2.1 Structural model of the Shinjuku Campus Building of Kogakuin University

Fig.1 shows a) the elevation with the locations of the acceleration sensors, b) the framing elevations, and the plan of a typical upper floor. The building consists of the double cores at both ends of the longitudinal (EW) direction, which consists of short beams and braces, and their span between the cores reach 25.6 m. The cores are connected by the truss beams at the 16th and 21th floors, whose heights are 5.1 and 5.5 m, respectively, and are higher than those of regular floors (about 4 m). The underground structures (down to 6th floors) are very rigid RC and steal frame structures, and thus, we can assume the building stands above the rigid basement.

Table 1 shows natural periods and damping ratios from the 1st modes to the 3rd modes, which were identified using the observed records during the '311' earthquake, and those of the 3D analysis model using a structural software (SNAP of Kozo System Co.). The analytical results show good agreements with those of the observations. In our analytical model, we adopt the Rayleigh damping ratios based on the observed results of the 1st and 3rd modes [2].

2.2 Seismic Response Simulation for the 2011 Great Tohoku-oki Earthquake (the '311' Earthquake)

Fig.2 shows the comparisons of accelerations (upper) and displacements (lower) between observations of the 311 earthquake (black lines) and simulations (red lines) at the 29th floor. We can confirm that the simulations reproduce well the observed seismic response, and thus, the validity of the 3D analysis model.



a) Elevation with the locations of the acceleration sensors　　b) Framing elevations　　c) Typical upper floor plan

Fig. 1 – Elevation, framing elevations and a plan of the campus building of Kogakuin University

Table 1 – Natural periods and damping ratios using observed records and 3D analysis model

NS	Observation Records		3D Analysis Model	
Mode	Natural Period (s)	Damping Ratio	Natural Period (s)	Rayleigh Damping Ratio*
1	3.080	0.019	3.080	0.019
2	0.950	0.012	0.979	–
3	0.470	0.030	0.502	0.030
Note*	Coefficients for [K] and [M] are 0.0042 and 0.0602			

EW	Observation Records		3D Analysis Model	
Mode	Natural Period (s)	Damping Ratio	Natural Period (s)	Rayleigh Damping Ratio**
1	2.960	0.012	2.994	0.012
2	0.990	0.012	1.035	–
3	0.502	0.030	0.551	0.030
Note**	Coefficients for [K] and [M] are 0.0046 and 0.0302			

2.3 Input Ground Motions for Damage Prediction and Retrofit Plan

Figs.3 and 4 show the input ground acceleration and the corresponding pseudo velocity response spectra, respectively, for estimating the building damage and the following retrofit plans. We use the four sets of the ground accelerations. The first is the El Centro NS record ('EL' for brevity) as a standard input acceleration for seismic design. The seconds is the simulated ground acceleration for a hypothetical M7.3 Earthquake under Tokyo ('Shuto-chokka': 'SHU' for brevity), where we used the hybrid method of the wavenumber integration method at longer periods and the empirical Green function method at shorter periods [3].The third and fourth are the synthetic accelerations for considering Nankai-Trough earthquakes. We compute them for fitting the target response spectra of the Japanese building code of Level 2, whose phase spectra are those of the observed records at the 1st floor of the Kogakuin University during the 311 earthquake (M9.0) and the 2004 Kii-Peninsula-oki earthquake (M7.6), respectively. We will cite the last two input motions as '311', and 'KII', respectively, for brevity. By considering the observed horizontal/vertical ratios, we set the UD components of the 311 and KII accelerations to be 1/2 and 1/3 of the horizontal components, respectively.

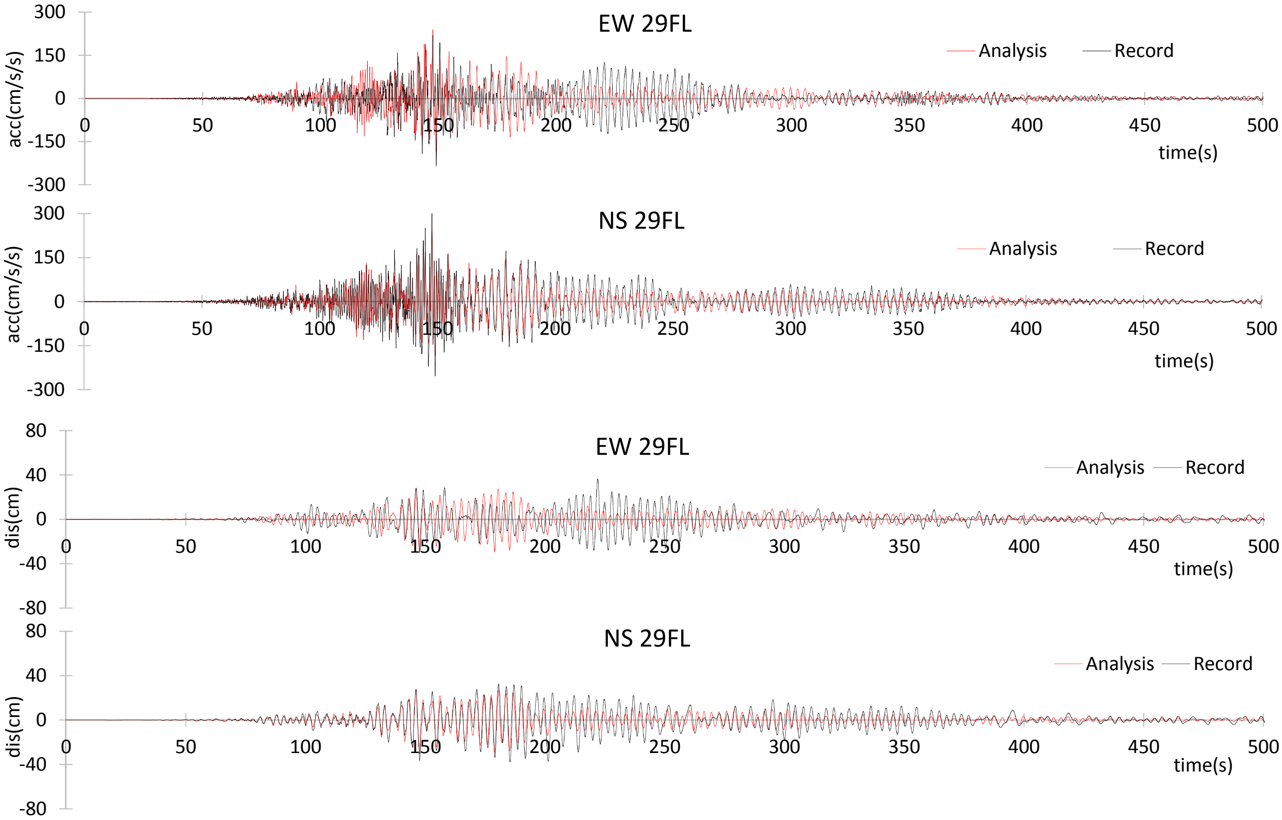


Fig. 2 – Comparisons of accelerations (upper) and displacements (lower) between observations and simulations

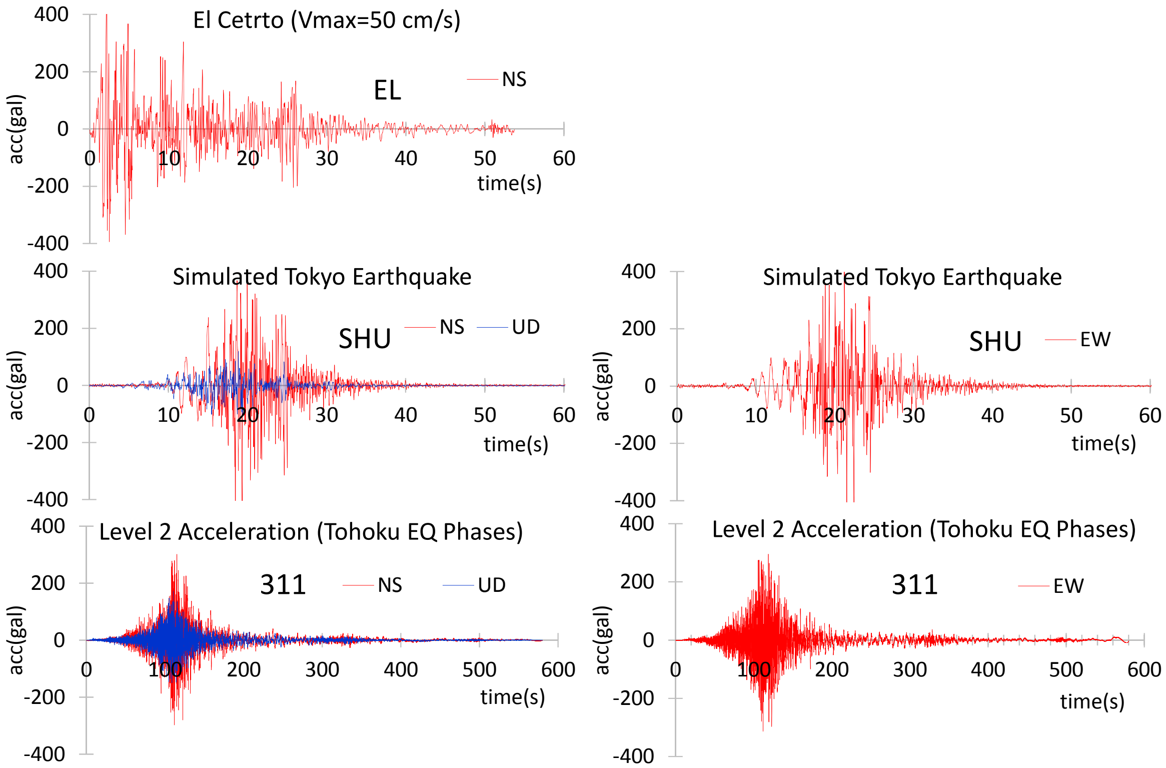


Fig. 3– Input ground accelerations for damage estimations and retrofit plans

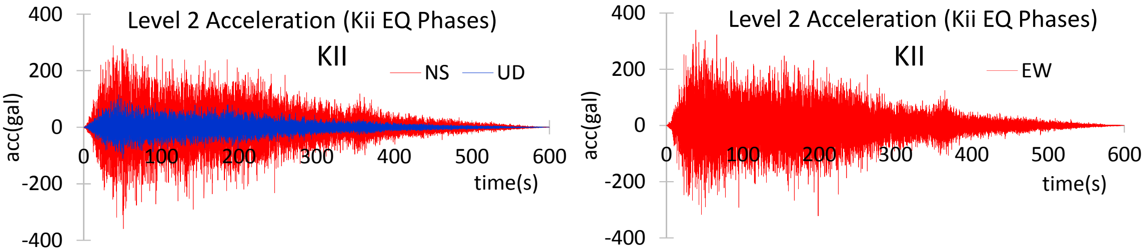


Fig. 3– Input ground accelerations for damage estimations and retrofit plans (continued)

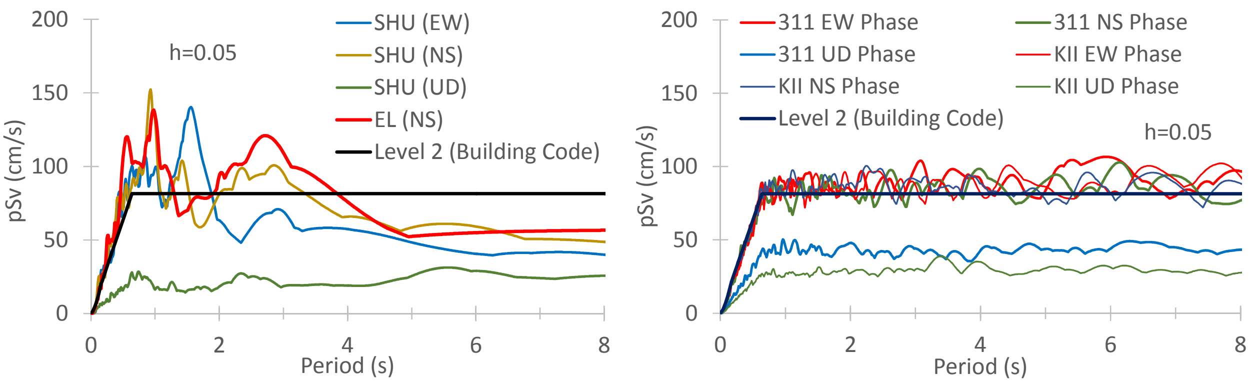


Fig. 4– Pseudo velocity response spectra using the accelerations of Fig.3

3. Retrofit Design Using Minimum Amount of Dampers

Many super-tall buildings constructed earlier employ conventional structural systems, and some of them have been retrofitted by using dampers in Japan. We also have been investigating the use of oil dampers for the 29-story steel building. We use the member-to-member analytical model of the building validated in Sec. 2.2 to examine effects of dampers. We discuss the yielding and damage potential of the frame based on its static push-over analysis in Sec. 3.1, and explains the retrofit design by using relatively small number of dampers considering budgetary restraints in Sec. 3.2.

3.1 Inelastic Characteristics and Potential Damage Distributions of the Frame

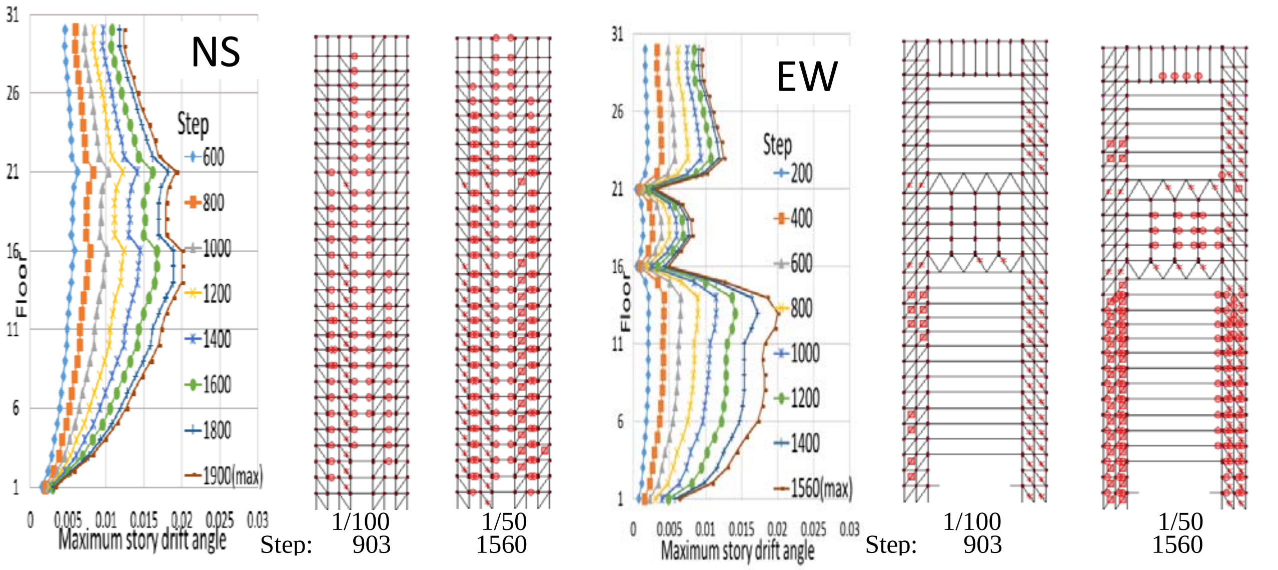
The building consists of steel moment resisting frames combined with many short braces providing seismic resistance. The slenderness of the typical brace is about 50, and buckling capacity is relatively large. However, since the braces yield at small story drift, their maximum deformation and cumulated plastic deformations must be controlled in order to avoid significant deteriorations of strength and stiffness.

The extent of yielding with respect to story drift magnitude is therefore examined by conducting static push-over analysis. Two types of lateral forces are examined: one follows the seismic design force based on the story shear distribution called *Ai*-distribution in Japanese code, and another is the inertia force estimated from the 1st mode displacement of the frame considering that the long period ground motion typically excites the longest vibration period of the building. Although the latter gave somewhat larger story shear at the lower story levels, they did not lead to very different design of dampers, thus, we will use the seismic design force based on the *Ai*-distribution throughout this paper.

Fig.5 shows the maximum story drift ratios and the corresponding extent of yielding/buckling of elements using the *Ai*-distribution loading. For the NS(X2) frame, when the maximum value of the story drift ratio throughout the building height reaches about 1/140, the first yielding occurs, and the yielding considerable begins spreading thereafter. Fig.5a shows story drift ratios and spread of yielding at the maximum story drift ratio of 1/100, where many beams connecting between the braced bays of the NS frame yield, due to the large vertical displacement and rotation of their ends caused by the cantilever actions of the brace bays.

In contrast, the first damage of the EW(Y14) frame is the compression buckling at the brace, when the drift ratio exceeded 1/200. The brace compression buckling and tension yielding as well as yielding of the beams at the braced frames spread at the lower stories for the story drift ratio 1/100.

In both frame, the damage seems to abruptly increase when story drift ratio exceeds 1/120 and 150, respectively, if the static force distributions well approximate dynamic inertia force distributions. Thus, we consider that these ratios as the approximate limits for the level 2 seismic design.



(a) NS (X2) frame (b) EW (Y14) frame

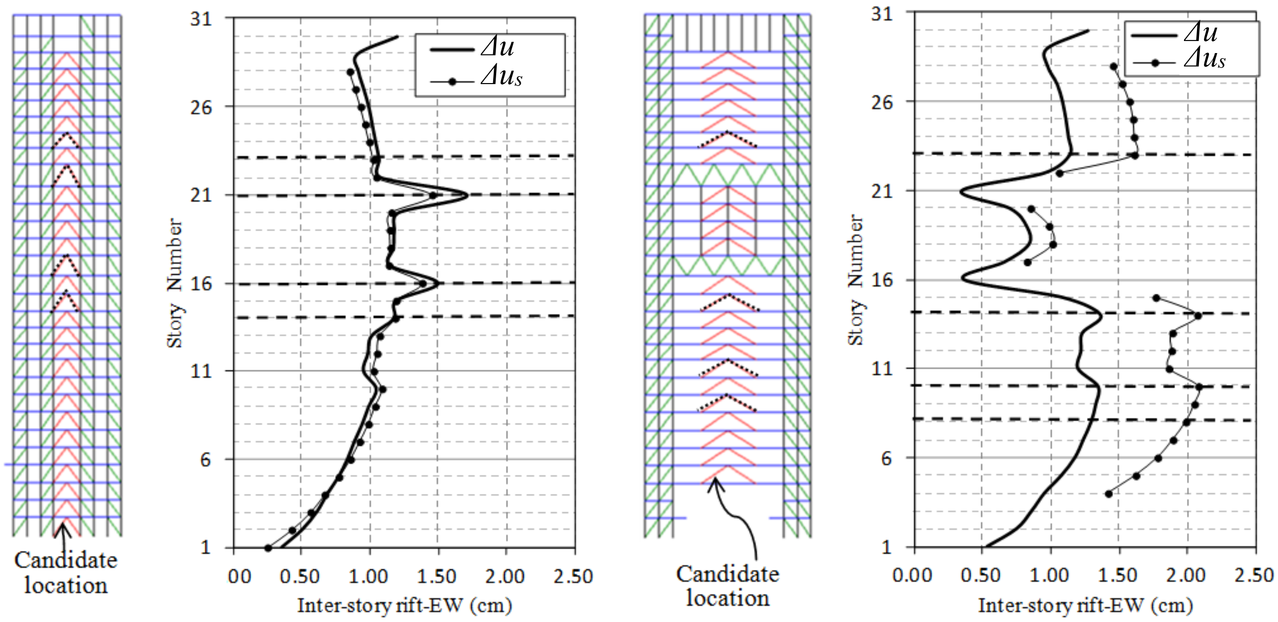
Fig. 5 – Various maximum story drift ratios and extent of yielding/buckling of elements using *Ai*-distribution loading (circle: bending yielding, star: compressive buckling, square: tensile yielding)

3.2 Selection of Damper Locations

Even though Fig.4 shows the pseudo-velocity velocity spectra with initial damping ratio of 5%, the actual measured damping ratios are very small as shown in Table 1: 1.9% and 1.2% for NS and EW directions, respectively. We try to increase them up to 6 % using a new retrofit design method to optimally determine the location of dampers. As will be noted, this depended on the feasible number of the oil dampers up to 64 and their selected locations to maximize the damping ratio. Note that the damper price is 20% of the total retrofit cost of about $100,000 per location, including structural, and largely non-structural and cosmetic works. Thus, limiting the number of dampers and locations is a key to control the cost.

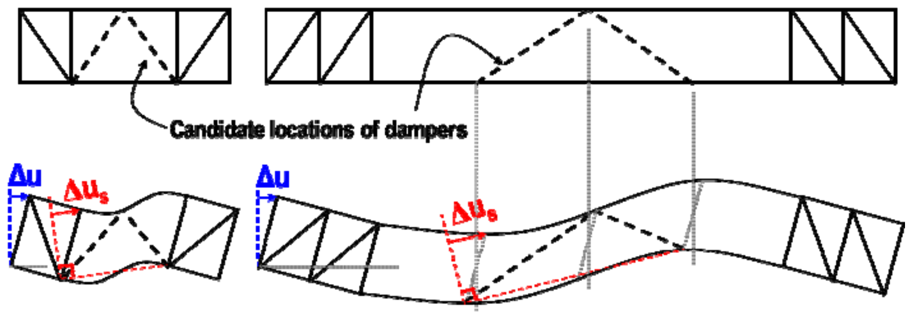
In a building with dampers, the frame must have the ability to transmit its deformation and inertia force to the damper as much as possible in order to maximize energy dissipation by the damper. The appropriate damper locations are found by measuring the shear drifts of the candidate frame units selected. The lateral force distributions follows the *Ai*-distributions as mentioned.

The above calculation is performed easily, by inserting zero stiffness braces into the candidate frame units of the 3D member-to-member building model, and obtaining their axial deformations and horizontal components (i.e., shear drift) as shown by Fig.7. For the NS direction (Figs 6a, 7a), the frame units in the central bay are considered, since their shear drifts would be amplified by the significant chord (bending) drifts of existing brace bays at both sides. As Fig.7a shows, shear drift of the central bay unit is larger than the story drift of the building, suggesting possibility of large damper stroke and energy dissipation. Note that the 2nd and 6th bays of Fig.6a are alternatively checked and their shear drifts are found to be much smaller. As for the EW direction (Fig.7b), the dampers are attached at the one-quarter span of the 25.6m long beam of 1m depth. Due to the eccentric attachment of the damper, large shear deformation develops (Fig.7b), where triangle is formed by the two dampers in the frame unit not containing actual columns.



(a) NS direction (X2 and X10) (b) EW direction (Y14 and Y21)

Fig. 6 –Static analysis to find deformation mode: Comparison between building story drift and shear drift
Note: horizontal broken lines indicate the selected locations of the dampers for D32 design.

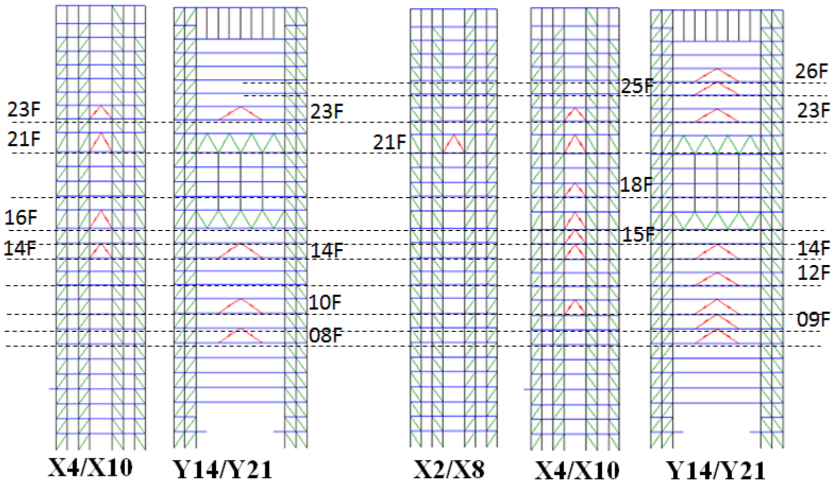


(a) NS direction (b)EW direction

Fig. 7 –Static analysis to find deformation mode: Comparison between building story drift and shear drift
Note: horizontal broken lines indicate the selected locations of the dampers.

Fig.8 shows two retrofit designs: D32 design uses 16 dampers for NS and EW directions, respectively. D64 design uses 32 dampers for NS and EW directions, respectively. Considering smaller shear drift (≃ damper stroke horizontal component) compared with the EW direction (Fig.6), the dampers may be increased in the NS direction, which was not pursued here. We select the story for the damper installation in the order from the largest shear drift of the frame unit, but we avoid the selection of consecutive story for D32 design.

The example selections for the D32 design are shown by the horizontal broken lines drawn in earlier Fig.6 and corresponding shear drifts of the frame units. For D64 design in NS direction (Fig.8), since the 21st level tends to show the largest frame unit shear drift without dampers mainly due to the largest story height of 5.5m (Sec. 2.1), 8 dampers instead of 4 dampers are considered. Thus, D32 design in average considers only 0.55 dampers per story in each of the NS and EW directions. Whereas, D64 design considers only 1.1 dampers per story in each direction.



(a) Scheme 1:D32 (D 16 EW+D16NS) (b) Scheme 2:D64 (D 32EW+D32NS)

Fig. 8 –Two kinds of damper arrangement scheme

3.3 Calculation of supplemental damping ratio

In order to control vibration for smaller earthquake and to limit the damper forces for larger earthquake, the oil damper with relief mechanism [4] is used. All the oil dampers have common sizes and properties of relief load $\hat{F}_{dy}$ = 1,600 kN, viscous coefficient $\hat{C}_d$ = 375 kN/(cm/s), post-relief viscous coefficient $p\hat{C}_d$ = 37.3 kN/(cm/s), and damper stiffness $\hat{K}_d$ = 5,400 kN/cm., where the " ^ " refers to the properties in axial direction (Fig.9). The damper size is the largest among those with reasonable constructability. The damper is connected in series with the brace, forming the "added component" (Fig.9) [4].

In order to judge whether the number and locations of dampers are satisfactory, the damping ratio of the vibration controlled systems is estimated using three methods as follows:

Method 1: A static analysis modeling the added component as the elastic spring is conducted. The stiffness is derived [4], based on the phase difference between the viscous element and elastic component consisting of the stiffness of damper and brace (Fig.10a). From obtained global displacement and local deformation of the added component, the strain energy of the total frame system and energy dissipated by the viscous element are estimated assuming the cyclic loading with the same displacement and deformations, which lead to approximate estimation for the first mode damping ratio [5].

Method 2: From analysis simulating free vibration of the system, maximum displacement as well as viscous element deformation and force are obtained. The strain energy of the system is evaluated as explained in method 1. As for the energy dissipation by the viscous element, obtained hysteresis is used to estimate it numerically.

Method 3: From the decay of the system displacement during the free vibration analyzed in method 2, well-known formula to estimate damping ratio can be used.

In all the above methods, we use the displacement that do not cause the frame yielding. Note that, we can include the bilinear characteristics of the damper (Fig.10b) in the formulation. However, the displacement where relief occurs appears to be similar the yield displacement of the frame. Thus, we set the frame elastic, and used the three methods as well. If the damper is relieved, the damping ratio will decreases with the increase of the strain energy of the structure, which will be discussed elsewhere. Table 2 shows the estimated damping ratios of the D32 and D64 designs using the three methods. The deformation and the energy dissipation of the damper calculated by two methods are very close, which means the static calculation method can accurately predict the dynamic displacement response of the damper. In spite of these small numbers of dampers, the D32 design gives damping ratios of 3.53% and 3.96% in the NS and EW directions, respectively, and the D64 design gives 5.06% and 6.15%, respectively. The small numbers of retrofit locations and yet relatively large damping ratio indicates the design is very economical and efficient.

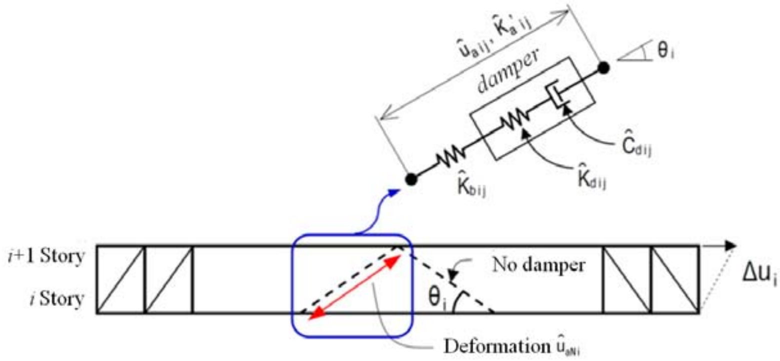


Fig. 9 – Candidate location of the dampers in the EW frame

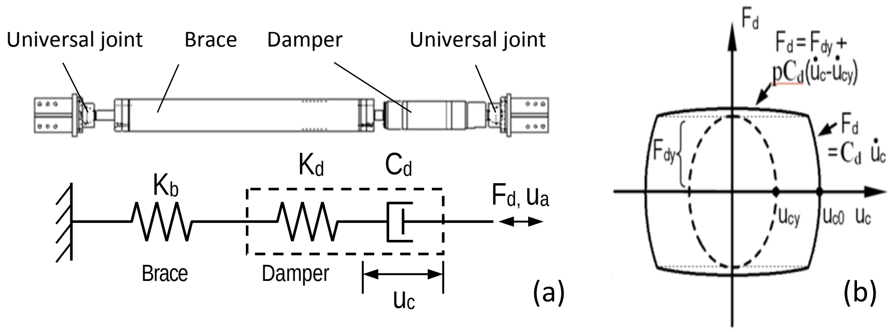


Fig. 10 – (a) Oil damper analysis model; and (b) hysteresis curves under smaller and larger strokes.

Table 2 –damping ratio of structure with the damper by the three methods

Dir.	D32 (%)			D64 (%)		
	method 1	method 2	method 3	method 1	method 2	method 3
NS	1.91+***2.46***=4.37	1.91+***2.31***=4.22	3.53	1.91+***3.93***=5.84	1.91+***3.74***=5.65	5.06
EW	1.24+***2.97***=4,21	1.24+***2.63***=3.87	3.96	1.24+***4.79***=6.03	1.24+***4.53***=5.77	6.15

Note：1.24% and 1.91% are the initial damping ratios of the frame in EW and NS directions, respectively.

4. Dynamic analyses with or without dampers

Dynamic analyses are conducted to evaluate the effectiveness of the retrofit with dampers. As explained in Sec. 3.2 (Fig. 8), D32 design uses 16 dampers in each direction, and D64 design 32 dampers in each direction, respectively (Fig.8). The oil damper elements simulating the bilinear viscous behavior (Fig. 10) are inserted into the 3D member-to-member frame model.

Fig.11 shows absolute peak values of story drift angles and accelerations due to the “EL”, “SHU”, “311”, and “KII” earthquakes (Sec. 2.3, see Figs.3 and 4). By virtue of increased damping ratios, D32 and D64 designs show good vibration control. In the NS direction (Fig. 11(a)), larger drift angle distributions at upper stories due to the EL and SHU earthquakes differ from those due to the 311 and KIT long-period earthquakes as well as the prior estimates (Figs. 5 and 6), because of the higher mode excitations. The drift angle of the 21st story is much larger than other stories, but is controlled by the dampers. In the ES direction (Fig. 11(b)), similar trend occurs due to the EL earthquake, but distributions due to the other earthquakes tend to be more uniform in the stories except the 16th to 21st stories that are originally stiff. In contrast, Figs. 11(c) and (d) show insignificant reduction of the peak floor accelerations. For the entire duration, however, accelerations at most cycles during the entire duration are significantly reduced (see the similar effect for displacement, Fig. 13).

Fig.12 shows the axial force and deformation curves for the oil damper in the 21st story of NS direction, where force cut-off by the relief mechanism is set to occur at the reasonable stroke, assuring good energy dissipation. These validate the design methods to determine the damper locations. Note also that the hysteresis curves due to the KII and 311 earthquakes contain many cycles due to the long-duration shaking of about 400 sec.

Fig.13 shows example displacement time histories at the top story of the structure using the D64 model under the 311 earthquake. In addition to the reduction of peak cycle responses mentioned above, the decay of the vibration is quick and causes even more significant reduction of other cycles during the entire duration (Fig. 13(c) to (d)), thereby reducing number of large cycles considerably. These are very important to avoid fatigue of both structural and non-structural elements, as well as fear of the occupants. It is noteworthy that in each direction, the above-mentioned effects are made possible by using, in average, about one damper or less per floor area of 983 m^2 (= 25.6 m×38.4 m).

5. Summaries and Conclusions

We evaluated the structural damage of an existing 29-story steel building for several hypothetical large earthquakes including the M7.3 earthquakes near Tokyo and a M8/M9-class earthquake along the Nankai-trough, and proposed effective retrofit plans using dampers. First, we constructed a detailed member-to-member frame model of the building, and compared the simulated results with the observed records of the 311 earthquake. Next, we estimated the building damage for the large earthquakes by conducting nonlinear structural analysis, and proposed a new retrofit design methodology to optimally determine the location of dampers. Even though we used small numbers of dampers (32 and 64), the estimated damping ratios were drastically increased from 1-2 % up to 6%, and the seismic response of the building were effectively reduced.

6. Acknowledgements

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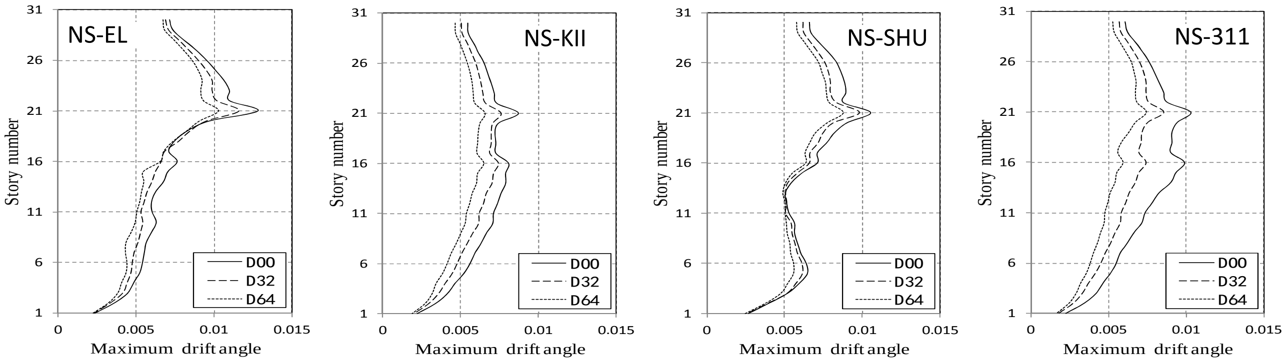
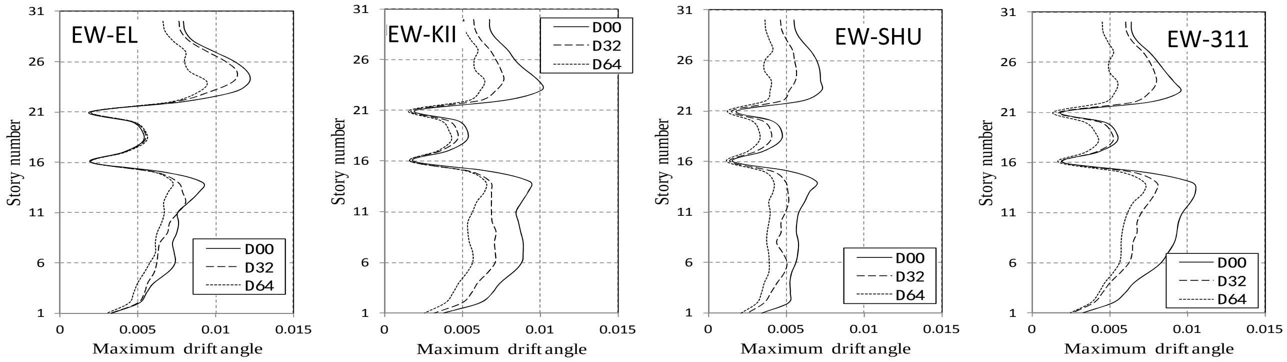
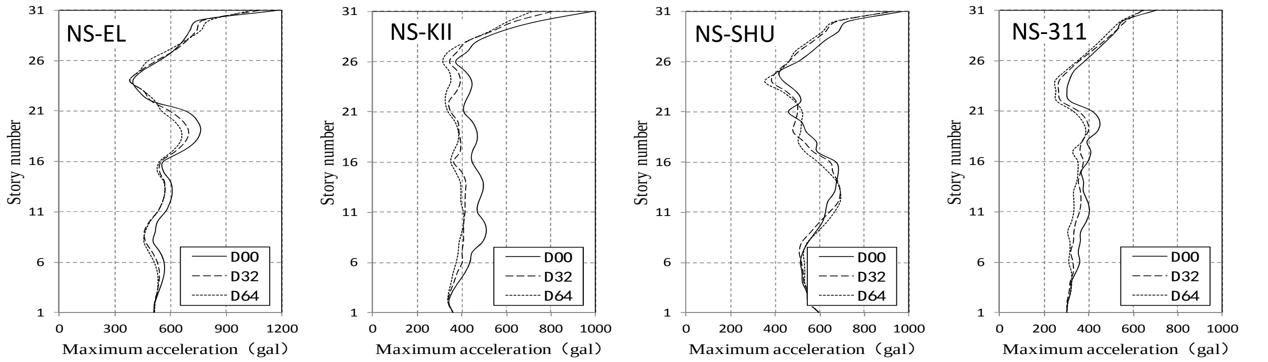


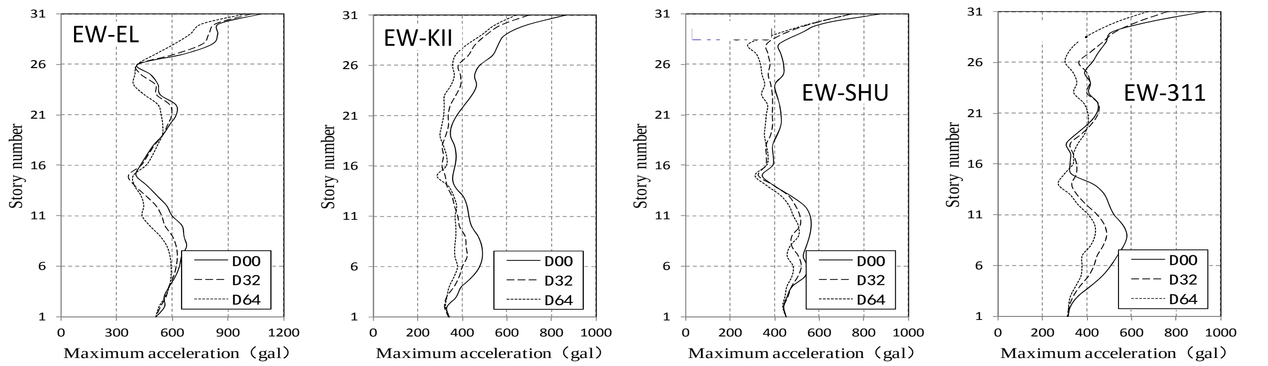
(a) Maximum inter-story drift angel in NS direction



(b) Maximum inter-story drift angel in EW direction



(c) Maximum acceleration in NS direction



(d) Maximum acceleration in EW direction

Fig. 11 –Story drifts angle and acceleration with/without damper

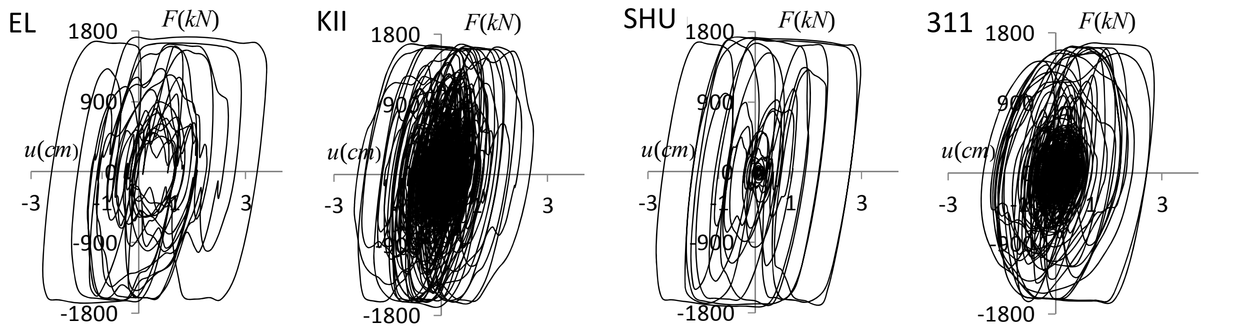


Fig. 12 – Hysteretic curve of the damper under four seismic waves of the 21 layer damper in the NS direction

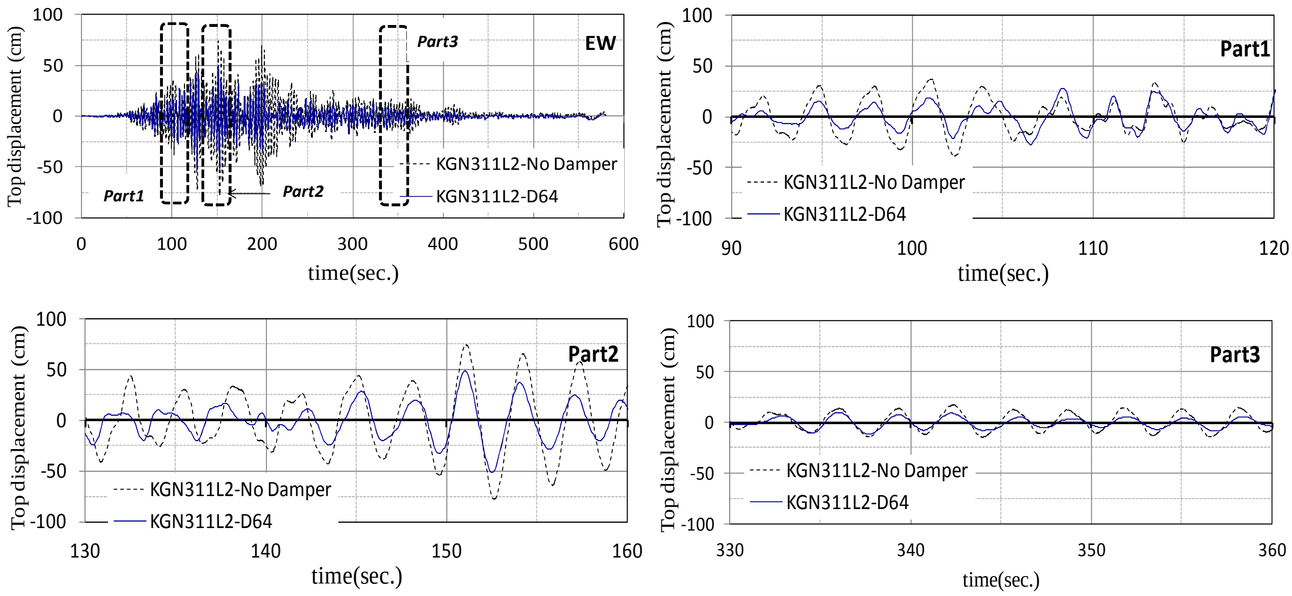


Fig. 13 – The displacement time history curve in the top story using the D64 dampers

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