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SEISMIC PERFORMANCE EVALUATION OF EXISTING HIGH-RISE STEEL BUILDING SUBJECTED TO LONG-PERIOD GROUND MOTION AND ASSESSMENT OF RETROFIT BY STEEL DAMPERS

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Abstract. *The main structural damage occurring in high-rise steel buildings subjected to long-period ground motions is cumulative inelastic deformation concentrated in the beam-column connections. The seismic performance of such buildings is thus significantly influenced by the type of beam-column connection detail. However, there is no current statistical information relating to beam-column connection details of existing high-rise steel buildings. Therefore, in order to evaluate seismic demands, a series of dynamic response analyses and long-period ground motions are conducted using two prototype building models. In addition, the effective distribution of dampers is considered in relation to retrofitting by steel dampers.*

1 INTRODUCTION

The design of high-rise buildings and the construction methods used have changed in relation to technology, society, and the economy. The safety of high-rise buildings designed 40 years ago, or longer, was previously examined using observed earthquake data such as El Centro; such studies were based on the maximum response. However, the main structural damage occurring in high-rise steel buildings subjected to long-period ground motions is cumulative inelastic deformation, which is concentrated on the beam-column connections [1]. In this respect, an evaluation of cumulative fatigue damage in the beam-column connections will not determine whether a building could withstand repeated ground motions throughout a large subduction-zone earthquake, such as the predicted Nankai Trough mega-earthquake. As many existing high-rise buildings were constructed in cities on the Pacific Ocean side of Japan, where considerable damage is predicted to occur with a massive Nankai Trough earthquake, there are misgivings about the safety of existing high-rise buildings in relation to cumulative damage occurring throughout long-period ground motion.

In this paper, we design 3D models that reflect the characteristics of the investigations made in relation to the existing 1970's high-rise steel buildings in Tokyo and 1980's buildings in Osaka. In addition, a time history response analysis is performed using predicted long-period ground motion based on recent research results. The seismic performance of the existing high-rise steel buildings subjected to long-period ground motions are thus evaluated based not only on the maximum response but also on the cumulative damage. Furthermore, the effect of retrofitting using steel dampers is assessed.

2 SEISMIC PERFORMANCE EVALUATION OF 1970’S HIGH-RISE BUILDINGS

2.1 Outline of 1970’s building models

The span of the column interval is one of the indices used to determine when a building was constructed. For example, this investigation verified that many existing high-rise steel buildings constructed in the 1970's have a short span column interval of approximately 3.2 m. In this respect, buildings with a short column interval have a small yield displacement of the beam material compared with buildings with long column intervals. There is therefore a possibility that cumulative damage will be increased at the beam-column connection in relation to long-period ground motion.

The existing high-rise building models used in this chapter are known as the "3.2-m model" and the "6.4-m model," and are respectively shown in Figures 1(a) and (b). These buildings have a height of 199.8 m (52 stories) and are of the center-core type. Figure 3 illustrate the results of static analyses. The natural period of the 1st mode of the 3.2-m model was found to be ${}_f T_1 = 6.10$ s, and the base shear coefficient at the elastic limit is $C_y = 0.048$ (${}_f T_1 C_y = 0.30$). However, for the 6.4-m model, the natural period of the 1st mode was ${}_f T_1 = 6.52$ s, and the base shear coefficient at the elastic limit was $C_y = 0.046$ (${}_f T_1 C_y = 0.30$). In addition, stiffness-proportional damping was used with a damping ratio of 2%.

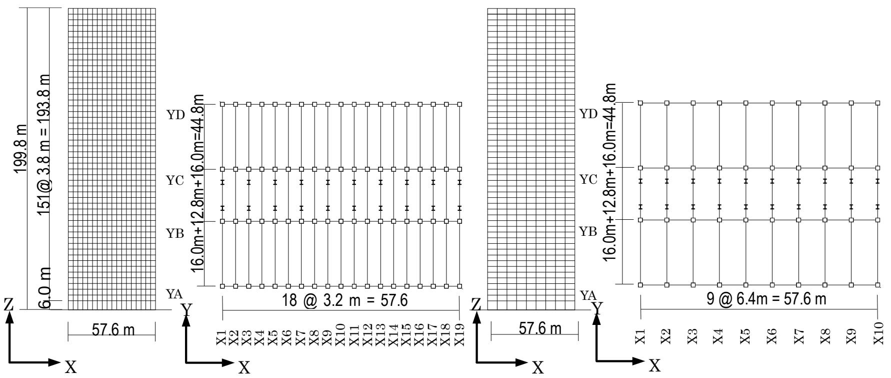
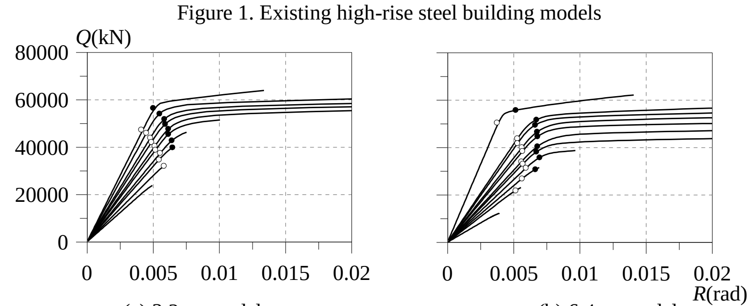


Figure 1. Existing high-rise steel building models



(a) 3.2-m model

(b) 6.4-m model

Figure 2. Static analysis results

2.2 Outline of input earthquakes

In this chapter, the following are used to evaluate the earthquake safety of existing high-rise steel buildings: El Centro 1940 NS (maximum velocity of 0.5 m/s); a simulated-seismic-wave with a phase of JMA KOBE 1995 NS (the response velocity spectrum S_v level when the damping ratio h = 0.05 is 0.8 m/s), known as ART-KOBE; and the estimated ground motion occurring in Tokyo during the Tokai-Nankai-Tonankai earthquake, known as KANTO. Figure 4 shows the time history of the input earthquakes used. Figures 5(a) and 5(b) illustrate the response velocity spectrum S_v (h = 0.05), and the energy spectrum V_E (h = 0.10) of the input earthquakes, respectively. As shown in Figure 3, KANTO has a long duration time compare with both El Centro and ART-KOBE. In addition, the energy spectrum (V_E) level of KANTO is four times that of El Centro, and three times that of ART-KOBE (see Figure 5(b)).

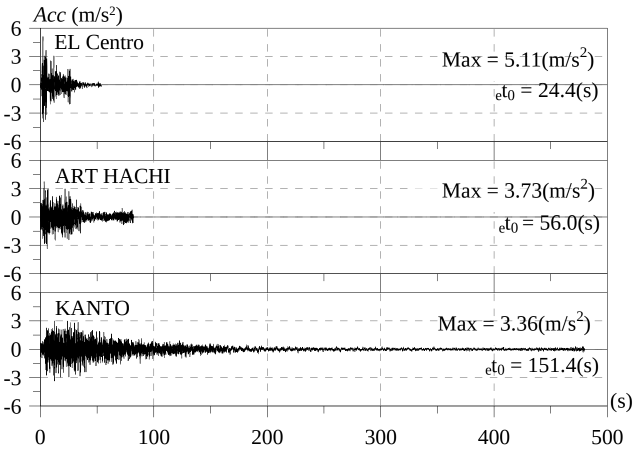
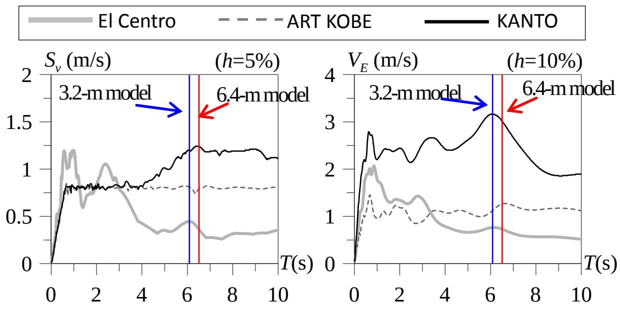


Figure 3. Time history of input earthquakes



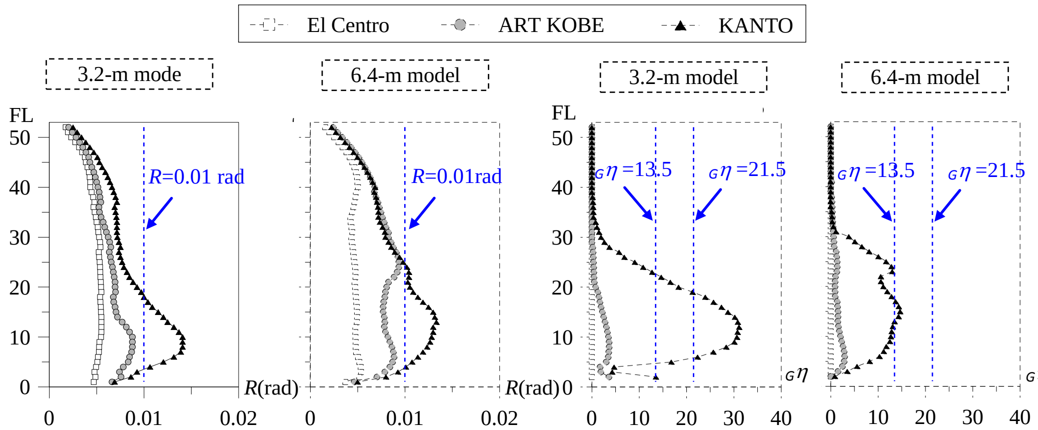
(a) Response velocity spectrum (b) Energy spectrum

Figure 4. Spectrum of input earthquakes

2.3 Time history response analysis results

In this study, the maximum story drift angle R, and the cumulative inelastic deformation ratio of the beam ${}_G\eta$, are used as indexes of earthquake safety evaluation, and were decided on based on a reference paper. In addition, the following are assumed: R = 0.01 rad, ${}_G\eta$ = 13.5 (field welding) and ${}_G\eta$ = 21.5 (shop welding)[2]. Figures 5(a) and 5(b) respectively show the time history response analysis results of the maximum story drift angle R, and the maximum cumulative inelastic deformation ratio of the beam ${}_G\eta$, at each story of the 3.2-m model and the 6.4-m model. As shown in Figure 5(a), R for both models were smaller than 0.01, when El Centro and ART-KOBE were used; however, when KANTO was input, the maximum story drift angle R reached approximately 0.014. In addition, ${}_G\eta$ for both models was 0 when El Centro was input, as shown in Figure 5(b); when ART-KOBE was used, there was a slight generation of damage to the models, although ${}_G\eta$ remained smaller than 13.5; and when KOBE was input, the ${}_G\eta$ of the 3.2-m model reached 30 and the 6.4-m model reached 15, thereby slightly exceeding 13.5.

These results therefore verify that although the 3.2-m model has an adequate seismic performance in relation to an earthquake such as El Centro or ART KOBE, it could not withstand the damage caused by KANTO, and therefore, it requires retrofitting.



(a) Maximum story drift angle

(b) Maximum cumulative inelastic deformation ratio of the beam

Figure 5. Time history response analysis result

2.4 Assessment of the effect of retrofitting with steel dampers

Figures 6(a) and 6(b) illustrate two types of arrangements of steel dampers used for retrofitting in the 3.2-m model. Figure 6(a) uses a method of installing steel dampers in all the layers of the building, and is known as a "consecutive layer placement" type of retrofitting. The yield shear coefficient of the damper installed in the 1st floor is 2%, and that of those installed on subsequent floors are decided based on the shear distribution. Using this method of retrofitting, as the damper axial tension is transmitted to the lower layer, the number of dampers in consecutive layers is either increased or decreased. In the "tree placement" type of method used for installing dampers, a large number of dampers are installed in the lower layer (in comparison with the consecutive layer placement type), as shown in Figure 6(b). The total number of dampers used is 1,248 in the consecutive layer placement type, and 1,184 in the tree placement type.

Figures 7(a)–(d) show the time history response analysis results of R, ${}_G\eta$, the effective damper deformation ratio α_e; and the maximum cumulative inelastic deformation ratio of the dampers, ${}_d\eta$, in each story of the 3.2-m building using both the consecutive layer placement type and the tree placement type. The effective damper deformation ratio α_e, is a ratio of the horizontal direction component of the maximum damper deformation and the maximum story-drift deformation. In addition, the analysis results of a 3.2-m building without dampers are shown in Figures 7(a) and (b).

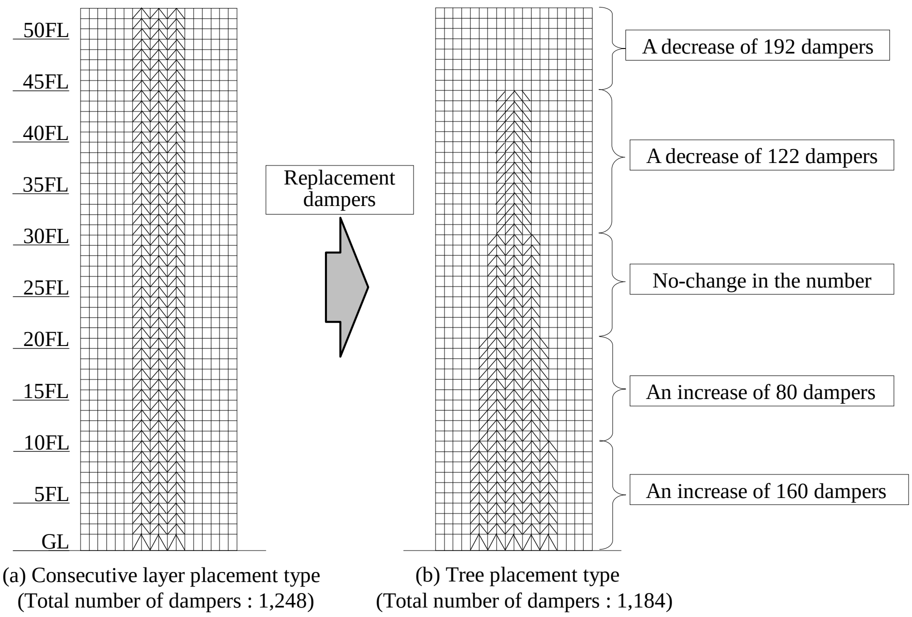


Figure 6. Arrangements of steel dampers in a retrofit of the 3.2-m model

In the lower layer, the maximum story drift angle of the building using the tree placement type of fitting dampers decreased greatly compared to the building without dampers, and the effect of the response reduction was larger than that of the consecutive layer placement type (see Figure 7(a)). In addition, in the lower layer, the cumulative inelastic deformation ratio of the beam of the building with the consecutive layer placement type decreased compared to that of the building without dampers; however, the maximum cumulative inelastic deformation ratio of the beam reached close to 13.5, as shown in Figure 7(b). In all layers of the tree placement type, the cumulative inelastic deformation ratio of the beam was greatly decreased, and no damage occurred to the main frame. The effective damper deformation ratio of the building with the tree placement type was larger than that of the consecutive layer placement type (see Figure 7(c)), and it was verified that the vibration control efficiency was improved by the dampers in the tree placement type. However, it is necessary to note that damper fatigue could occur in the tree placement type, because there may be an increase in the cumulative inelastic deformation ratio of the damper compared to in the consecutive layer placement type, as shown in Figure 7(d).

Figure 8 shows the absorbed energy by plasticity of the dampers, ${}_dW_p$; the absorbed energy by plasticity of the main frame, ${}_fW_p$; the absorbed energy by damping of the main frame, ${}_fW_h$; and the input energy, E. It can be verified that the energy absorbed by plasticity of the main frame of the building without dampers was larger than that of the building with dampers. In addition, the input energy of the tree placement type and that of the couple arrangement was almost the same. The absorbed energy by plasticity of the dampers in the tree placement type was the same as that in the consecutive layer placement type; however, the absorbed energy by plasticity of the main frame of the tree placement type was smaller than that of the consecutive layer placement type.

These results show that the vibration control performance changes in relation to the damper placement, and that the vibration control performance of the tree placement type is high compared with that of the consecutive layer placement type.

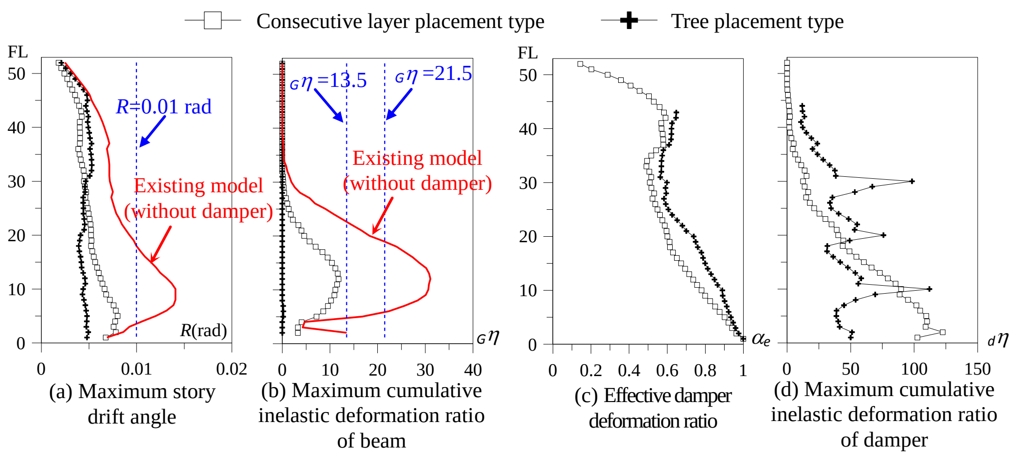


Figure 7. Time history response analysis result of 3.2 m-model

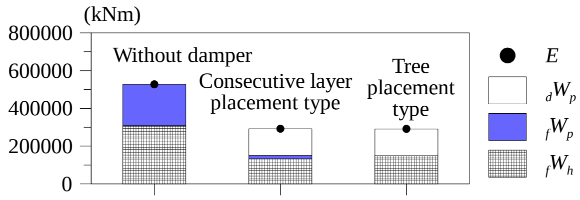


Figure 8. Energy distribution of 3.2-m model

3 SEISMIC PERFORMANCE EVALUATION OF 1980'S HIGH-RISE BUILDING

3.1 Outline of 1980's building model

Figures 9(a) and (b) show the 1980's building model, which is a side-core type with a height of 146 m height (36 stories). Figures 10(a) and (b), respectively, illustrate the static analysis results in an X and Y direction. This model's natural period of the 1st mode of the X direction is $_fT_{1x}$ = 4.80 s, and the base shear coefficient at the elastic limit of the X direction is C_{yx} = 0.075 ($_fT_{1x}C_{yx}$ = 0.36). In addition, the natural period of the 1st mode of the Y direction = $_fT_{1y}$ = 4.90 s, and the base shear coefficient at the elastic limit is C_{yy} = 0.082 ($_fT_{1y}C_{yy}$ = 0.40). Stiffness-proportional damping is used, and the damping ratio is 2%.

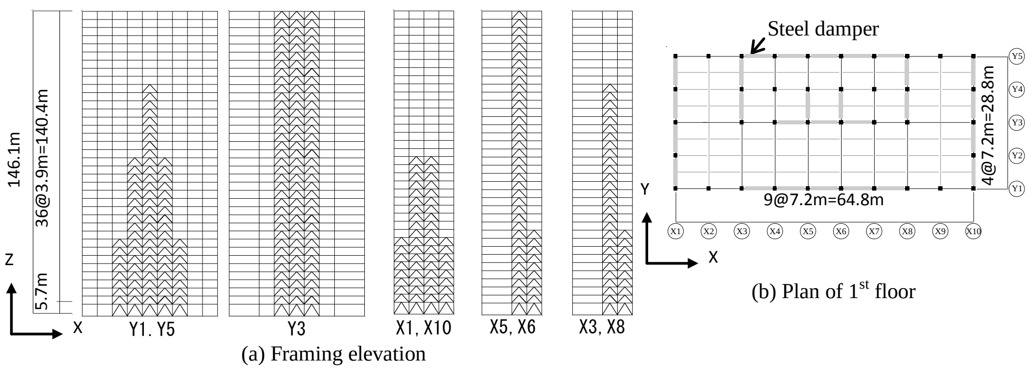
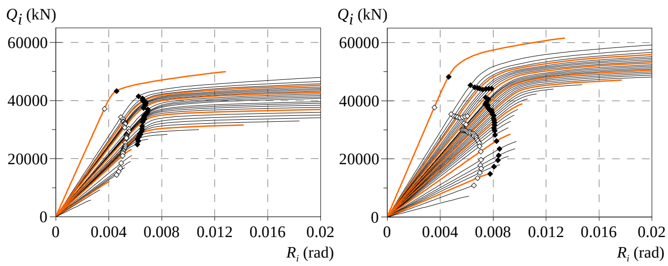


Figure 9. Existing steel 1980's high-rise building model

Steel dampers are installed in the building, as shown in Figure 10(b). It was decided that the yield shear coefficient of the damper installed in the 1st floor would be 0.3 times the shear coefficient of the frame, and that the yield shear coefficient of the damper at each floor would be based on the shear distribution.



(a) X direction (b) Y direction

Figure 10. Static analysis results of 1980's high rise building model

3.2 Outline of earthquakes input

In this chapter, two kinds of simulated long period earthquake ground motions, known as OSAKA1 and OSAKA2, were used to estimate ground motion of the Tokai-Tonankai-Nankai coupled earthquake. Figure 11 shows the time history for OSAKA1 and OSAKA2. Figure 12(a) illustrates the response velocity spectrum S_v (h = 0.05), and Figure 12(b) shows the energy spectrum V_E (h = 0.10) of the input earthquakes. As shown in Figure 11, OSAKA1 and OSAKA2 have a long duration time compare to El Centro and ART-KOBE (see Figure 3), and furthermore the energy spectrum (V_E) level of OSAKA1 and OSAKA2 are three or four times that of El Centro (see Figure 4(b)).

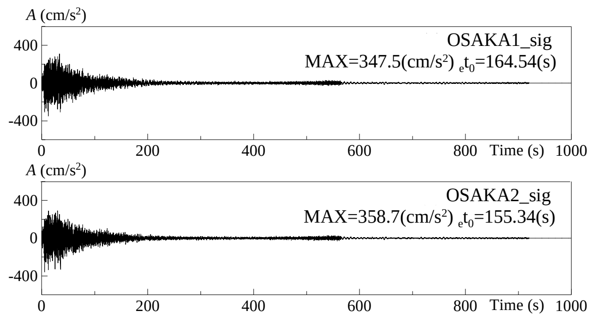
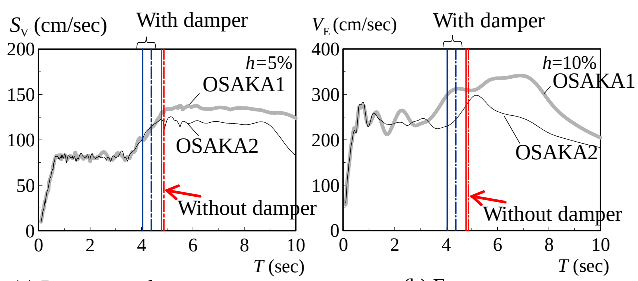


Figure 11. Time history of input earthquakes



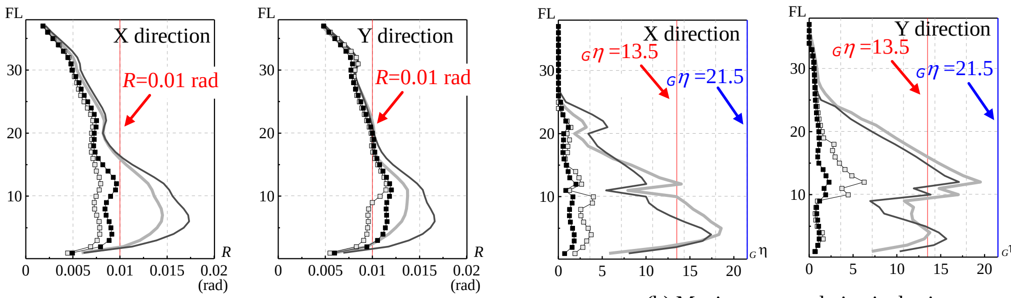
(a) Response velocity spectrum (b) Energy spectrum

Figure 12. Spectrum of input earthquakes

3.3 Results of time history response analysis on a building with and without dampers

The maximum story drift angle, R, and the cumulative inelastic deformation ratio of the beam, ${}_G\eta$, are used as indexes of earthquake safety evaluations. Figures 14 and 15, respectively, show the results of a time history response analysis relating to the maximum story drift angle, R, and the maximum cumulative inelastic deformation ratio of the beam, ${}_G\eta$, on each story in an X and Y direction. As shown in Figure 14, the maximum story drift angle, R, in the building without dampers was about 0.017 rad–exceeding 0.01 rad in an X and Y direction when OSAKA2 was input. In addition, the maximum cumulative inelastic deformation ratio, ${}_G\eta$, in an X and Y direction was about ${}_G\eta$ = 18, illustrating that the building without dampers would suffer damage in both directions, and therefore acknowledging the need for retrofitting with dampers against the simulated long period earthquake ground motions.

The maximum story drift angle of the building with dampers decreased greatly in both directions in comparison with the building without dampers, and the response in the Y direction was slightly bigger than 0.01 rad. However, the cumulative inelastic deformation ratio on all floors of the building with dampers decreased in comparison with the building without dampers, and therefore there was minimal damage to the building.



(a) Maximum story drift angle

(b) Maximum cumulative inelastic deformation ratio of beam

Figure 13. Result of time history response analysis

4 CONCLUSION

In this paper, the seismic performance of existing high-rise steel buildings subjected to long-period ground motions was evaluated based not only on the maximum response but also in relation to cumulative damage. Furthermore, the effect of retrofitting using steel dampers was assessed. The maximum response and cumulative damage on all floors of the building with dampers decreased in comparison with that of the building without dampers, and there was subsequently minimal damage to that building.

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