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SEISMIC RETROFITTING EFFECT OF 10-STORY RESIDENTIAL BUILDING RETROFITTED USING PASSIVE SEISMIC CONTROL SYSTEM WITH AMPLIFIER MECHANISM

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Abstract. *In Japan many buildings that were designed by the old code have been retrofitted structurally because lots of buildings designed by the old code were collapsed in the Southern Hyogo prefecture earthquake in 1995. Moreover many retrofitted structures were subject to strong seismic motion in the 2011 off the Pacific coast of Tohoku Earthquake. It was found that seismic retrofitting was very useful for seismically weak structures. In Sendai city, there is the 10-story residential building that was retrofitted using a passive seismic control system with an amplifier mechanism. The nonstructural members (partial walls) of the building had been damaged when Tohoku Earthquake, though their structural members had not been damaged. In order to evaluate the effect of the seismic retrofitting, we investigated the damage of the building after the 2011 off the Pacific coast of Tohoku earthquake. In addition we calculated the story drift angle by seismic response analysis to evaluate from the analytical viewpoint. As a result of the damage investigation of this building, it was obvious that the building was safe structurally. Furthermore, we verified when nonstructural members had been in failure using the modified compression field theory(Vecchio,F.J. and Collins,M.P, 1986). The nonstructural members were in failure before the peak acceleration. Thus, we concluded that the effect of the partial walls on structural performance was very small.*

1 INTRODUCTION

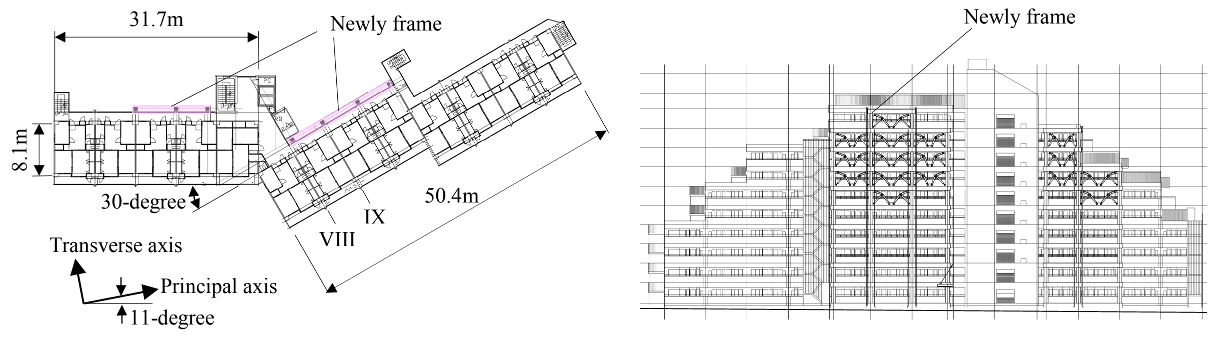
At the time of the 2011 off the Pacific coast of Tohoku Earthquake (the Earthquake below), the City of Sendai near the earthquake source subjected to very strong ground motion. Numerous buildings with poor earthquake resistance had been retrofitted against the Miyagi-ken-oki Earthquake and other active-fault-induced inland earthquakes that were highly likely to occur. The state of the retrofitted buildings were reported[1] after the Earthquake. The effects of and problems of seismic retrofit are gradually being known.

The authors investigated the structural performance and the effects of seismic retrofit in a ten-storied high-rise residential apartments (the Building below) that had been retrofitted using a seismic control systems equipped with amplification mechanism. Damage to nonstructural members was confirmed in numerous buildings including the Building in the wake of the Earthquake. In this study, therefore, the shear stress - deformation relationship in nonstructural members of the building was estimated using an existing theoretical model, and the effects of nonstructural members on the structural performance of the Building were examined based on the shear strength and the time when the shear strength was reached.

2 OUTLINE OF THE BUILDING AND SEISMIC RETROFIT

2.1 Outline of the Building

Figure 1 shows a plan view of the second floor of the retrofitted Building and a northern elevation. The Building was constructed in 1979. At present, buildings are designed to the earthquake resistance standards came into forced in 1981. The Building does not therefore comply with the existing standards. The Building is of a complex shape. In plan, the Building bends sharply at the elevator hall and is of an echelon structure in eastern buildings. In elevation, the Building has setbacks on both sides on the fifth and higher floors. From a structural viewpoint, the Building is made of steel encased reinforced concrete (SRC) on the first to sixth floors and of reinforced concrete (RC) on the seventh to tenth floors. The Building is of a frame structure longitudinally and of a frame structure combined with shear walls in the span direction.



(a) The floor plan (b) The nothern elevation

Figure 1: Views of the building

2.2 Outline of retrofit design

As a result of the seismic diagnosis earthquake resistance diagnosis of the Building, the Is value exceeded 0.6, the level specified in the new earthquake resistance standards, on all floors in the span direction, but was lower than 0.6 longitudinally except on some floors. It was then decided to apply seismic retrofit longitudinally. For retrofit, a seismic retrofit method using the seismic control systems with amplification mechanism was adopted in view of the post-retrofit usability of habitable rooms, usability of the Building during retrofit work and cost performance. Seismic control systems with amplification mechanism were installed on the side of the side-corridor on the north face. An exterior retrofit method was adopted in which a retrofitted frame composed of steel columns and SRC beams was connected to the existing frame through additional slabs. The additional slabs were connected to the existing frame using post-installed anchors. The objective of retrofit was to prevent the Building from collapsing. The main design criterion was to hold the story drift angle below 1/125 in the SRC structure and below 1/150 in the RC structure against the wave defined by the earthquake motion in notification with a spectrum intensity of an extremely rare earthquake motion and the typical recoded wave. Shear failure of columns was not allowed before the story drift angle reached the angles of the design criteria. Structural slits were made in brittle columns to ensure ductility before retrofit.

2.3 Seismic control system with amplification mechanism

The method of retrofit using seismic control systems equipped with amplification mechanism [5] amplifies the story displacement in the frame and transfers the amplified displacement to the hydraulic damper and thereby doubles the absorption of seismic energy, applying the principle of leverage. In the seismic retrofit, hydraulic dampers with a design marginal damping force of 43 tf were used. The amplification ratio was set at 1.7 to 2.3 times. A total of 32 dampers were installed, four on the sixth floor, ten on the seventh floor, eight on the eighth floor, eight on the ninth floor and two on the tenth floor. The seismic control systems with amplification mechanism were placed only on the sixth through tenth floor. Only retrofitted frames were placed on the first through fifth floors.

3 RESULTS OF POST-EARTHQUAKE FIELD SURVEYS OF THE BUILDING

3.1 Outline of field surveys

Surveys were conducted on April 6 through 8, 2011. Relatively large aftershocks occurred on March 11 when the Earthquake occurred through April 8, the last day of surveys. The results of the surveys therefore include also the results of aftershocks. Investigations were made of (i) existing structural members, (ii) retrofitted frame (newly applied structural members), (iii) nonstructural members and (iv) seismic control systems with amplification mechanism. Only the sections where confirmation was possible from the side corridor were examined. An aftershock with a seismic intensity of 6 upper occurred on April 7. Only item (iv) was investigated after the aftershock and the other items before the aftershock.

3.2 State of existing structural members

Photographs 1 and 2 show an existing column with cracking and a wall with slits on three sides, respectively. Small cracks were confirmed in some of the existing columns and beams, most of which had a width of 0.2 mm or smaller. The damage was classified by the Japanese guidelines for evaluating seismic performance of reinforced concrete structures as the “one that enables continued use of the structure without repair”. Photograph 2 shows that shear cracks occurred in the wall with three slits but that none progressed to existing columns. It was thus determined that the slits worked effectively

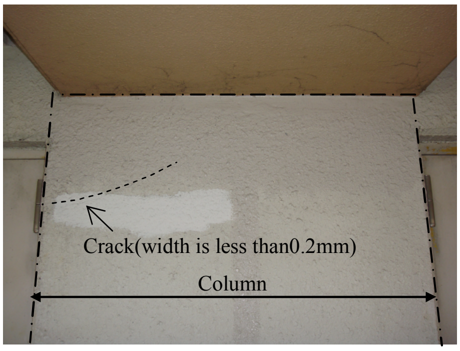


Photo. 1: Example of Columns with cracking

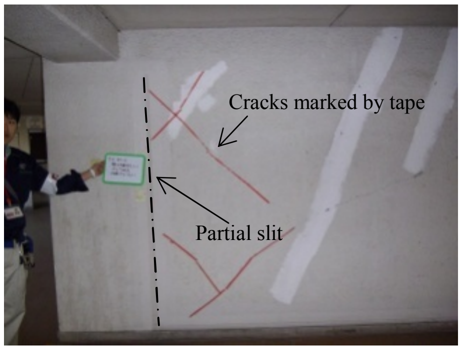


Photo. 2: Shear wall with slit

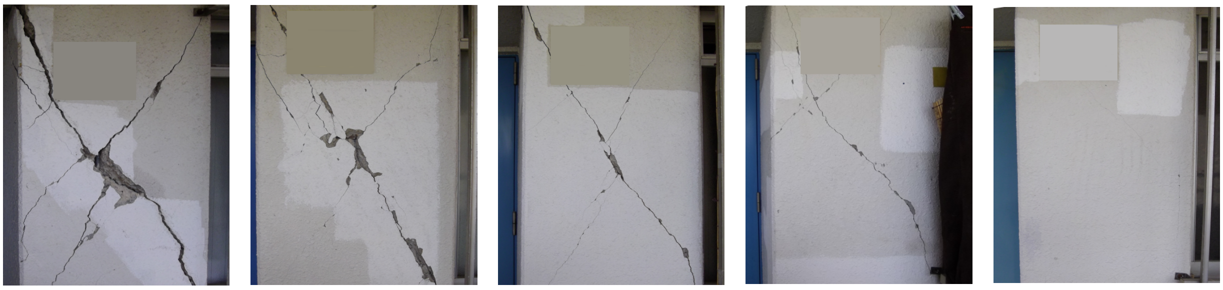
3.3 State of newly applied structural members

Newly applied steel columns were coated with autoclaved lightweight concrete (ALC) plates for fireproofing. No direct observation of the frame was therefore possible. No damage was, however, found in ALC plates. If steel members had yielded and been deformed greatly, ALC place might also suffered damage. It was therefore assumed that there was no serious damage to newly applied columns despite local damage. No damage was confirmed in newly applied beams. No trace was recognized of cracking or slip at the plane of connection at the bond between newly applied and existing frames. It was thus assumed that shear force was appropriately transferred through post-installed anchors, and both frames behaved as one during the Earthquake. The seismic control systems with amplification mechanism that was responsible for absorbing seismic energy, therefore, worked effectively.

No damage was confirmed that required repair in the structural members of the Building as well as in the existing structural members. The Building was found to be fit for continuous use.

3.4 State of nonstructural members

No measures were considered in the retrofit design to avoid or reduce damage to nonstructural members. Shear cracks were confirmed in the partial walls on the side of the side corridor. In the 120-mm-thick partial wall, ϕ =10mm reinforcing bars were arranged at a pitch of 250 mm. Photograph 3 shows the damage to the partial walls between lines (viii) and (ix) on the first through tenth floors. Only moderate shear cracks with a width of approximately 0.2 mm (concrete crack width except spalling) were found in the partial walls on the first and tenth floors. Damage was larger on the middle floors. The spalling of concrete cover was confirmed on the third through fifth floors. In the surveys, the residual crack width was measured with a crack scale in partial walls on each floor. The results are shown in Section 3.5.



(f) 6F (g) 7F (h) 8F (i) 9F (j) 10F

(a) 1F (b) 2F (c) 3F (d) 4F (e) 5F

Photo. 3: Failing of partial walls

3.5 State of seismic control systems with amplification mechanism

In the surveys, total traces of lubricant left each time the hydraulic damper worked during an earthquake were verified. Hydraulic displacement was measured based on the trace. If it was assumed that the hydraulic damper returned to the center after the earthquake, the measurement would have indicated the unidirectional hydraulic displacement (in the direction in which the damper shrank). The maximum hydraulic damper displacements on respective floors are shown in Figure 2 together with the residual crack widths in the partial walls. The figure shows that the hydraulic displacement of damper was largest on the sixth floor and smaller on the upper floors. The tendency was similar to the state of residual crack widths in the partial walls. In the partial walls on the third through fifth floors, no residual crack widths could be measured because of the spalling of concrete.

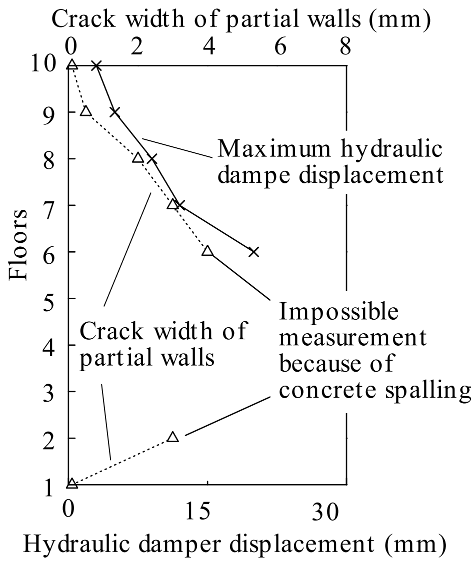


Figure 2: Results of the suvey

4 ESTIMATION OF SEISMIC DEFORMATION BEHAVIOR BASED ON SEISMIC RESPONSE ANALYSIS

4.1 Outline of seismic response analysis

The seismic deformation behavior of the Building was evaluated by nonlinear 3D-frame analysis that considered vertical springs of piles and the seismic control system with amplification mechanism. As analysis models, the Takeda model [3] was used for representing force-displacement characteristics of structural members, Maxwell model for hydraulic damper and a slip model considering the displacement loss at the point of attachment for the brackets of the seismic control system with amplification mechanism. Seismic response analysis was made in cases with and without seismic retrofit. The deformation of the Building during the Earthquake was grasped and the effect of seismic retrofit was verified.

4.2 Estimation of input motion

No seismic monitoring were conducted in the Building. The ground motion records obtained at Sendai city hall, approximately 2.6 km south of the study site, were modified for use. To modify, a relatively simple method was used that enabled the estimation of ground motion at the study site with adequate accuracy from an engineering viewpoint. The method uses the H/V spectral ratios at the study site and the site of seismic records based on the microtremer measurement. Specifically, it was assumed that equation (1) could be established based on the works of Seo et al. [2] and the frequency amplitude of observed wave was corrected using the H/V spectral ratios at the two sites. Figure 3 shows *H/V* spectra at the two sites.

$$\frac{A(f)}{B(f)} \approx \frac{A_{H/V}(f)}{B_{H/V}(f)} \tag{1}$$

where, $A(f)$ and $B(f)$ are the Fourier amplitudes of ground motions at the study site and the site of seismological observation, respectively; and $A_{H/V}(f)$ and $B_{H/V}(f)$ are *H/V* spectra at the study site and the site of the Sendai city hall.

In the seismic response analysis, ground motions were in two directions, along and transverse to the principal axis. The ground motions modified using *H/V* spectra were subjected to orientation transform. Figure 4 shows the acceleration time history of input ground motion. Figure 5 shows the pseudo velocity spectrum (h = 5%). The figure also shows the notification of the Japanese ministry spectrum used as level-2 ground motion used in retrofit design.

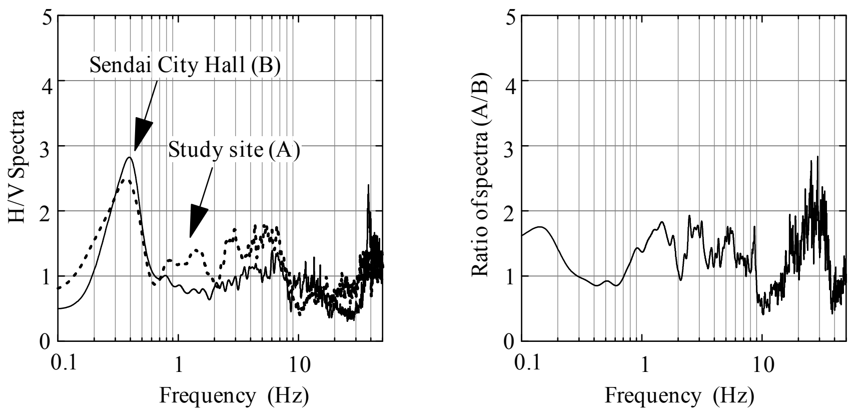


Figure 3: H/V spectra at the two sites

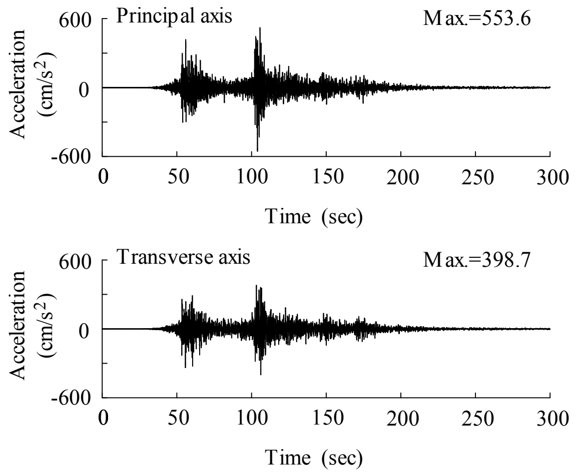


Figure 4: Input ground motions

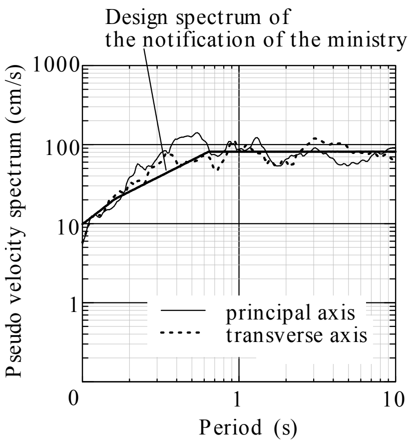


Figure 5: Pseudo velocity spectra

4.3 Behavior in the case with no seismic retrofit

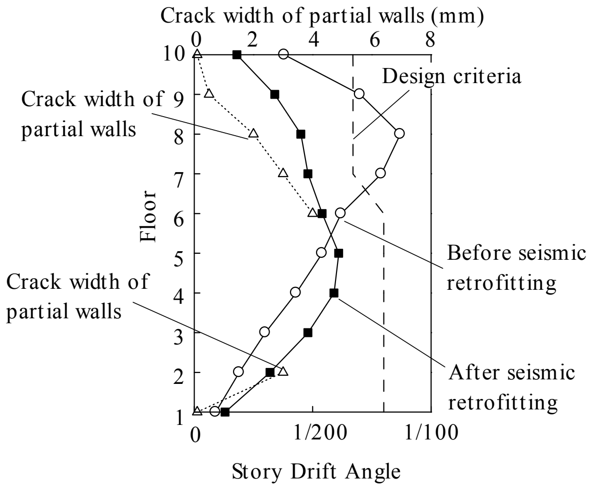


Figure 6: Story Drift Angle

Figure 6 shows the results of analysis of the maximum story drift angle in the longitudinal direction in the case with or without seismic retrofit for comparison. Story drift angle before seismic retrofit exceeded the design criteria (reinforced concrete structure: 1/150, SRC structure: 1/125) on the seventh through ninth floors and increased to 1/115 on the eighth floor. In the case without seismic retrofit, therefore, some damage may have occurred such as the shear failure of structural members on upper floors of the Building. The distribution of story drift angles in the direction of height in the case without seismic retrofit shows that the story drift angle was largest on the eighth floor. The tendency was different from the state of damage to partial walls confirmed by field surveys and described in the previous chapter in which damage was moderate on the lowest and highest floors and damage increased on middle floors.

4.4 Behavior in the case with seismic retrofit (actual behavior)

The maximum story drift angle after seismic retrofit was 1/164, approximately 70% of a maximum value of 1/115 in the case without seismic retrofit. Story drift angle after seismic retrofit was smaller than the design criteria on all floors, which was in agreement with the results of field surveys in which no damage was found in structural members.

The distribution of story drift angles in the direction of height after seismic retrofit shows that story drift angle was largest on the fifth floor. The deformation was in agreement with the damage to the partial walls found in the building surveys. Figure 6 also shows the width of the residual crack in the partial wall between lines (viii) and (ix) for comparison with story drift angle. No measurement could be made on the third through fifth floors. Small cracks on the upper and lower floors and large cracks on the middle floors were in agreement with the distribution of story drift angles. The tendencies of hydraulic displacement and residual crack width (Figure 2) were also similar. Thus, the results of seismic response analysis for evaluating post-seismic-retrofit conditions were also in agreement with hydraulic displacement.

As a result of above discussions, it was assumed that the results of seismic response analysis conducted in this study generally reproduced the seismic behavior of the Building during the Earthquake. Retrofitting the Building using seismic control systems with amplification mechanism greatly reduced story drift angle. Then, the design criteria were satisfied. The vertical distribution of story drift angles was altered owing to seismic retrofit because seismic control systems with amplification mechanism were installed only on upper floors.

5 CONSIDERATIONS CONCERNING THE DAMAGE TO PARTIAL WALLS

5.1 Outline

One of the characteristics of damage to the Building was the confirmation of shear cracking in partial walls despite the soundness of structural members owing to seismic retrofit. Partial walls were nonstructural members. The damage to partial walls was allowable according to the seismic design policy of the Building. Knowing the degree of influence of partial walls on the seismic behavior of the Building provides valuable data for future seismic retrofit design. In this chapter, the maximum shear on the partial wall, shear deformation angle in shear failure is reached and the time of partial wall failure are estimated.

5.2 Method for estimating shear strength of partial walls and deformation

Studies have been made mostly on structural members. Few studies have been conducted concerning partial walls, as nonstructural member. Most of the previous studies were experimental verifications. No works are available that analytically evaluate shear stress - deformation angle relationships in partial walls. In this study, it was assumed that the stress-deformation relationship in partial walls could be estimated using the modified compression field theory (MCFT) [4], one of the reinforced concrete plate constitutive laws because the partial walls in the Building were shaped like a square. Figures 7 shows the stress and displacement fields in the MCFT model, respectively. The MCFT model expresses the stress-deformation relationship in reinforced concrete plane element based on the condition of compatibility of deformation, condition of equilibrium of stress, and material constitutive laws of rebars and concrete.

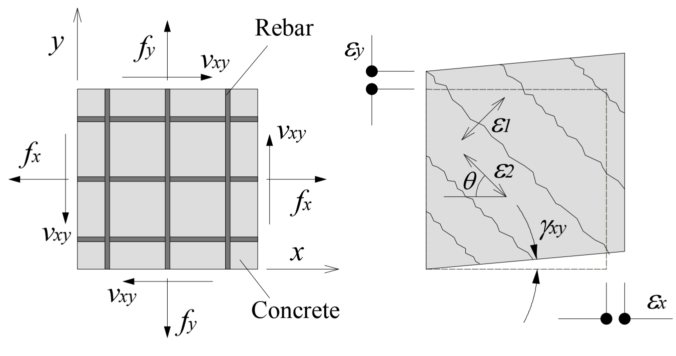


Figure 7: Example of the construction of one table.

5.3 Estimation of shear strength of partial walls and deformation angle at shear strength

Table 1 lists the shear strengths and deformation angle in shear failure on respective floors of the Building calculated using the MCFT model. The shear strength of the partial wall ranged from 111 to 132 kN. Story drift angle when shear strength was reached was approximately 1/330 on each floor. Judging from the fact that the shear strength of a main reinforced column on the seventh floor of the Building was 700 kN to nearly 1000 kN, the shear strength of the partial wall was approximately 10 to 15% of the shear strength of existing columns.

Story	f_c (N/mm^2)	Q_{pw} (kN)	R_{dd} (mm/mm)	R_{cd} (mm/mm)
10	22.50	132	1/333	1/610
9	22.50	132	1/333	1/610
8	11.45	111	1/331	1/608
7	22.50	132	1/333	1/610
6	12.48	113	1/331	1/608
5	15.04	118	1/332	1/610
4	13.47	116	1/331	1/608
3	13.13	114	1/331	1/608
2	15.07	123	1/332	1/610
1	22.50	132	1/333	1/610

f_c: Concrete compressive strength, Q_{pw}: Shear strength of partial wall,
R_{dd}: Dd-Angle(distributed drift angle), R_{cd}: Cd-Angle(concentrated drift angle)

Table 1: Example of the construction of one table.

5.4 Time of shear failure of partial walls during the Earthquake

Figure 8 shows the detail of the partial walls of the Building. In structures of that partial wall is integrated with a hanging wall and a spandrel wall, like the Building, deformation concentrates locally as shown in figure 8. Then, story drift angle is not identical to the deformation angle of the partial wall. No shear cracking was confirmed in hanging walls and spandrel walls of the Building. It is therefore reasonably assumed that no deformation occurred in these sections but that deformation concentrated at the center of the partial wall. In other words, it was assumed that the partial wall suffered shear failure before story drift angle reached the deformation angle at the shear strength of the partial wall. It was assumed that the hanging wall and spandrel wall were subjected to rigid deformation. The story drift angle when the partial wall collapsed was referred to as the Cd-angle (concentrated drift angle), and listed in Table 1. The Cd-angle is the story drift angle at which the partial wall suffered shear failure at the earliest time. The story drift angle that was the same as the deformation angle at the shear strength of the partial wall was referred to as the Dd-angle (distributed drift angle). At which story drift angle the partial wall suffered shear failure was estimated based on the results of time history response analysis and of the surveys of the Building.

Figure 9 shows the time history waveforms of story drift angle on the first, second, third, fifth and tenth floors after seismic retrofit based on the results of seismic response analysis. The time when story drift angle reached the Cd-angle (Cd-angle point) and the time when story drift angle reached the Dd-angle (Dd-angle point) were plotted on the output waveform on the positive and negative sides, respectively. The time history output waveforms of story drift angle were obtained as a result of calculation for the column member (line (viii)) on the west side of the partial wall shown in Photograph 3.

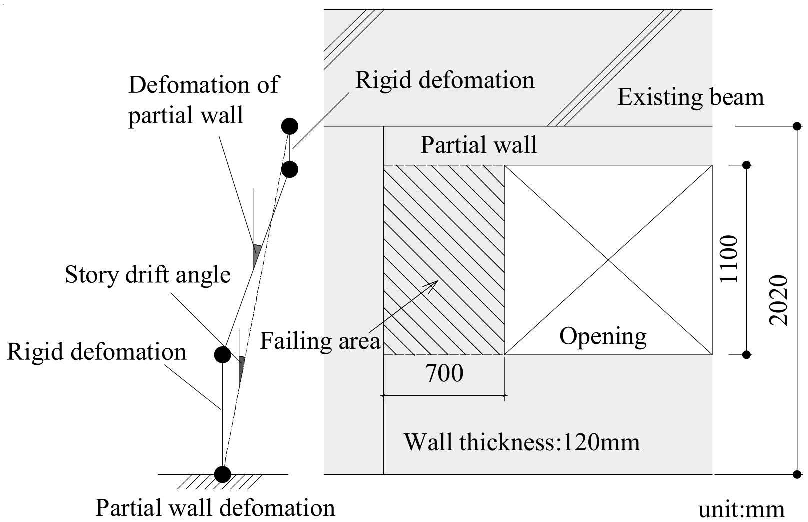


Figure 8: Partial wall derogation when the earthquake.

Figure 9 shows that story drift angle did not reach the Cd-angle during the Earthquake on the first and tenth floors, which confirms that only moderate damage occurred in the partial walls on the first and tenth floors.

Focus is now placed on the second, third and fifth floors in Figure 9. One of the characteristics of the input ground motion is two peaks in the acceleration waveform. On the third and fifth floors, the Cd-angle and Dd-angle points existed before the first peak was reached. On the second floor, the Cd-angle point existed but no Dd-angle point existed before the first peak was reached, which indicates that the Dd-angle was not reached. It was, however, determined that partial walls suffered shear failure on the second floor based on the condition of partial walls on the second floor (Photograph 4 (b)) and 3-mm residual crack width. It is therefore highly likely that partial walls suffer shear failure when the story drift angle reaches the Cd-angle.

The Cd-angle point in the partial wall suggests that the partial wall suffered shear failure in the early stages of the Earthquake because the story drift angle reached the Cd-angle point before the first peak was reached on the second, third and fifth floors. The tendency was similar on the fourth, sixth, seventh and eighth floors. In seismic retrofit design, no seismic energy absorption by partial walls was considered. The partial walls suffered shear failure in the early stages of the Earthquake and it was concluded that seismic energy absorption of partial walls made little contributions to the structural performance of the Building.

6 CONCLUSIONS

The following conclusions were obtained.

- As a result of damage surveys of the Building, it was found that the structural frame of the Building suffered no serious damage and that the Building could be used continuously without any repair.

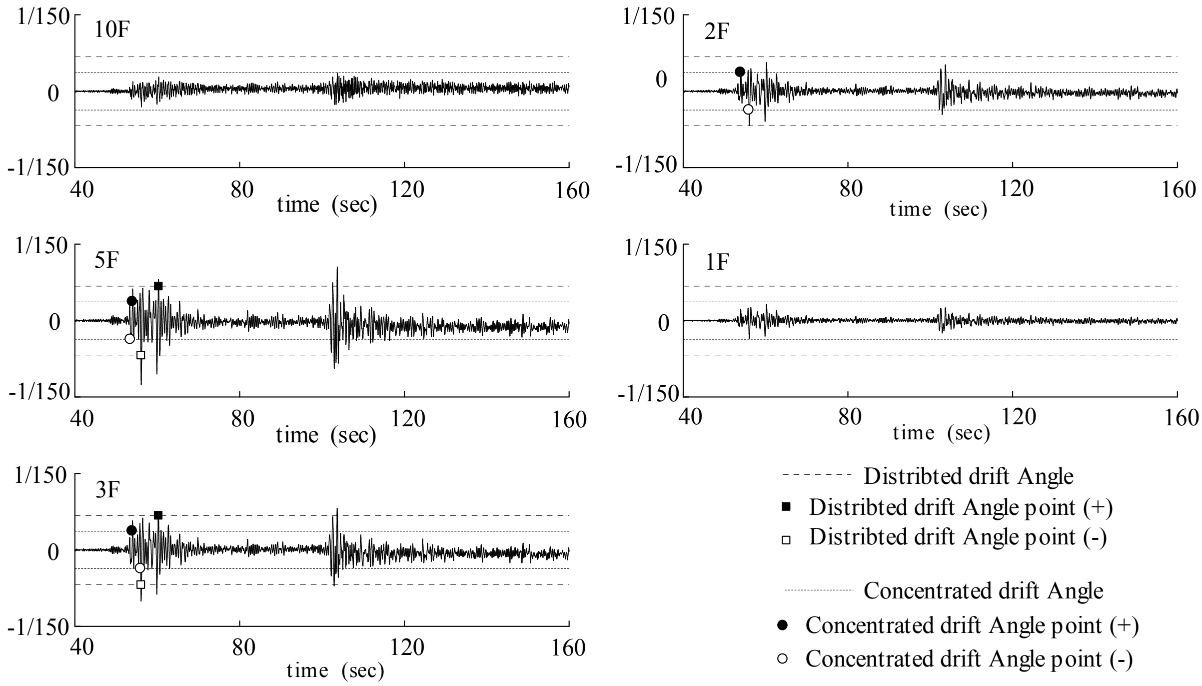


Figure 9: The time history output waveforms of story drift angle after seismic retrofit

- The results of estimation of the maximum story drift angle after seismic retrofit based on the results of seismic response analysis were in agreement with the damage to partial walls and with the action of hydraulic dampers. It was considered based on the analysis results that seismic control systems with amplification mechanism reduced the story drift angle of the Building to approximately 70%, the angle in the case without seismic retrofit, and the design criteria were satisfied. Then, no damage was observed in structural members during the Earthquake.
- It was shown that shear strength and story drift angle when shear strength was reached could be estimated properly using the MCFT model. As a result of calculation for partial walls of the Building, shear strength was 111 to 132 kN and deformation angle when shear strength was reached was approximately 1/330.
- As a result of the time history waveforms of story drift angle during the Earthquake based on seismic response analysis, it was assumed that partial walls collapsed in relatively early stages of earthquake. It was thus concluded that partial walls had little influence on the structural performance of the Building.

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REFERENCES

[1] Sasaki, K., Tanaka, R. and Takase: *Seismic Retrofitting Using Vibration Contorol System with Ampifer Mechanism and Effect of Sendai holl, No. 750,* The Kenchiku Gijutu(Architectural technology), July 2012.(in Japanese)

[2] Seo K.: *A Joint Work for Measurements of Microtremors in the Ashigara Valley*, International Symposium the Effect of Surface Geology on Seismic Mortion, ESG1992, pp.43-52, 1992.

[3] Takeda, T.: *Study on Reinforced Concrete Members Subjected to the Alternate Plastic Deformation - Characteristics of Deflection of Reinforced Concrete Frames-*, Anual conference of Architectual Institute of Japan ,pp153, 1964.

[4] Vecchio. F. J.and Collins M. P.: *The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear*, ACI Struct. J., pp.219~231, Mar.-Apr. 1986.