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DAMAGE TO EIGHT-STORY RC BUILDING RETROFITTED TO HAVE BASE SHEAR COEFFICIENT OF 0.65

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ABSTRACT: An eight-story reinforced concrete building retrofitted according to the 1990 Japanese Standard was severely damaged during the 2011 Tohoku earthquake. The building was constructed in 1966 at Tohoku University. In 1996 the building was retrofitted using structural walls and carbon fiber. The base shear coefficients of the building in the longitudinal and transverse directions after the retrofit were estimated to be 0.67 and 0.65, respectively. This paper discusses the reasons for the severe damage that took place despite the retrofit.

Key words: RC structure, structural walls, carbon fiber, seismic retrofit, penthouse, base shear coefficient

INTRODUCTION

This paper deals with a reinforced concrete building in Tohoku University built in 1966 and retrofitted in 1996.The retrofit was expected to bring the building the capacity of withstand a big earthquake. However, it experienced great damage in some of its members.

The observed failure modes in this building were different from the predicted ones. Structural walls, which were expected to fail in shear, seemed to have yielded in flexure. Compressive failure associated with bending was also observed in a wall without boundary column. In addition, the carbon fiber used to prevent shear failure of coupling beams did not work properly because of anchorage failures. The penthouse, was also greatly damaged, though the areas of the walls in X and Y directions were 2.8% and 1.3%, respectively, of the total floor area. The penthouse was tentatively strengthened after the earthquake as shown in Photo 1.

Photo 1 General view of the building after tentative strengthening of the damaged penthouse

This paper discusses hypotheses about the discrepancies between the predicted and observed failure modes, as well as the reasons why the building retrofitted to have such high base shear coefficient was so damaged during the 2011 Tohoku Earthquake. Data from the buildings have been drawn based on the original and reformation plans. Observed damage patterns, analysis models and results are also presented in this paper.

CONFIGURATION OF THE BUILDING

The building under analysis was constructed in 1966 in Tohoku University, Sendai, located approximately 130 km from the epicenter and it is dedicated to Electrical Engineering. It is a reinforced concrete building with eight stories and a two-story penthouse. The first story is 4.3 m, each upper story is 3.6 m and the two stories of the penthouse are 5.4 m and 3 m high respectively. It has seven spans in the East-West direction of 8m and three spans in the North-South direction of 6.6 m, 6 m and 6.6 m respectively, defining the structural grid of the building (Fig. 1). The upper stories have the same distribution in plan. Storage, baths and vertical communication nucleus are in the middle longitudinal axis. The rooms are around this nucleus, giving a configuration quite symmetrical to the building. The configuration of the building becomes different when reaching the penthouse level. It is characterized by having three bays of 8.0 m x 6.0 m in the western part of the roof (Fig. 2). Also, from third to eighth stories the West and East facades have cantilevers of 3 m (Fig. 3).

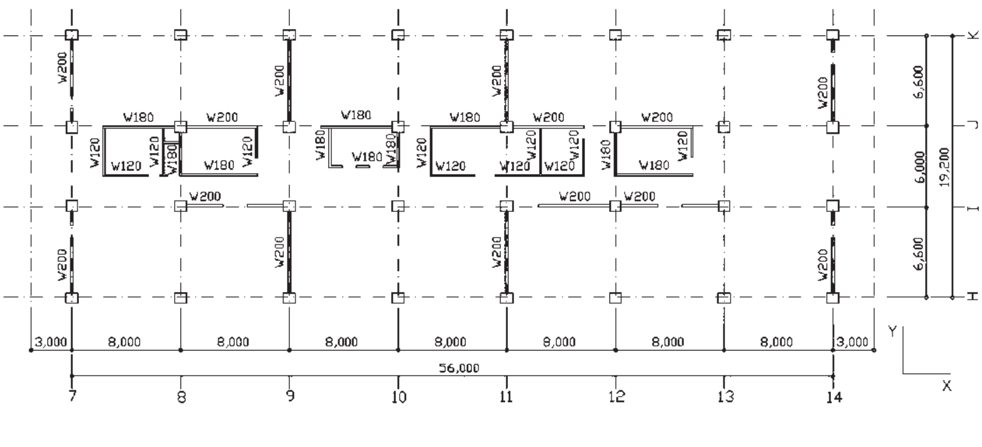

Fig. 1 Sixth story plan

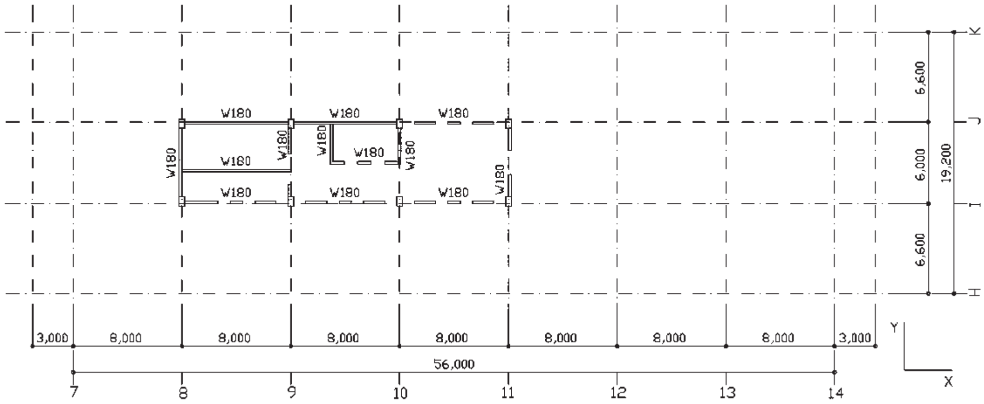

Fig. 2 First story of the penthouse plan

Due to this configuration the walls are concentrated in the middle bay, having totally free frames in both south and north facade. The building has also suffered several modifications. Some of the holes in some structural walls have been filled after the first completion of the building but before the retrofit, like in the penthouse or in the first story, giving continuity to the structural system.

The building was retrofitted in 1996 according to the 1990 Japanese Standard (Building Research Institute 2001). The main changes during the retrofit were addition of new walls, change concrete walls for new and thicker ones and reinforcing some of the beams of the building with carbon fiber. According to the documents for the retrofit design, the base shear coefficients of the building in the longitudinal and transverse directions after the retrofit were estimated to be 0.67 and 0.65 respectively. While the wall changes were concentrated in lower stories, from first to fifth stories, the carbon fiber reinforcement has been located mostly in the upper ones. About the location of the carbon fiber reinforcement, since there was no documentation of where was applied, a deep field investigation was needed. From the total of 159 short beams in the building only 109 were visible. The fiber was used in 24 of them, specifically in those in the third to eight stories. However, the fiber was not only used in short beams, but this reinforcement was also found in 7 additional beams. There were three kinds of fiber reinforcement: the first one is tape-shaped (Photo 2), the second one was a continuous fiber wrapping all beams (Photo 3) and the third one is similar to the previous one but with anchorages inside the concrete (Photos 4). Regarding to the penthouse no retrofit was done.

Photo 2 Stuck tape-shaped carbon fiber

Photo 3 Stuck carbon fiber without anchors

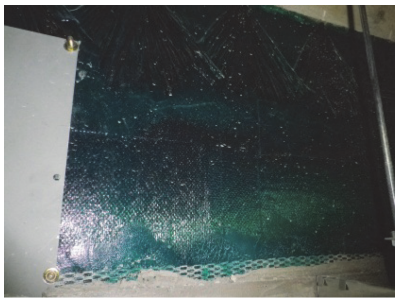

Photo 4 Stuck carbon fiber reinforcement with anchors

Referring to the constructive aspect, the building has slabs of 120 mm thickness in all stories. They have also a thickening of 60 mm near the beams (Figs. 8 and 10). The size of the beams varies from 350 mm x 800 mm to 300 mm x 1400 mm, but the most common section is 400 mm x 1000 mm. The size of the columns decreases around every 2 stories about 100 mm each side, varying the size of columns from 1100 mm x 1000 mm in the first story to 400 mm x 700 mm in the penthouse. About the foundation, the building has reinforced concrete piles. They are 400 Ø, 10 m long and they have a resistance of 392 kN per pile.

The used stirrups are plain and 9 Ø or 13 Ø, distributed a distance between 150-250 mm in beams and every 150 mm in columns; satisfying the limit of 300 mm of the Building Standard Law Enforcement Order of 1950 (Architectural Institute of Japan 1951). The used materials are concrete of 18 MPa, longitudinal bars of 324 MPa (SD35) for girders and columns, and 235 MPa (SR24) for other elements. These values were also adopted in the retrofit design.

The ratio of the area of longitudinal reinforcement to the concrete section area is between 0.86% - 0.45% in columns and between 0.57% - 1.60% in beams. The transverse reinforcement ratio is between 0.09% - 0.88% in beams and between 0.08% - 0.21% in columns. The ratio of 0.88% is used in the beams connecting walls. The reinforcement ratio of wall panel is between 0.23% - 1.09%.

OBSERVED DAMAGES AND ANTICIPATED CAUSES

The heaviest damages in the building are located mainly in beams and in the penthouse. Damages in the first story were minor. Figures 3-6 shows the observed cracks in selected frames. The grey shade shows the parts that were not visible, the green one represents carbon fiber reinforcement, the brown one is tape-shaped reinforcement, the red one is detached concrete and the blue one is completely destroyed concrete. Also the thickness of cracks represents cracks between 0.2-1.00 mm, 1.10-2.00 mm and above 2.00 mm. The blue cracks were not measured. The most damaged stories in decreasing order were third, fourth and fifth stories. Regarding to the frames in X direction, damages were more prominent in frames I and J with walls (Fig. 3) than open frames H and K. In Y direction, damages in the penthouse were large as shown in Figs. 4 and 5. Damages in the beams with openings were also prominent as shown in Figs. 5 and 6.

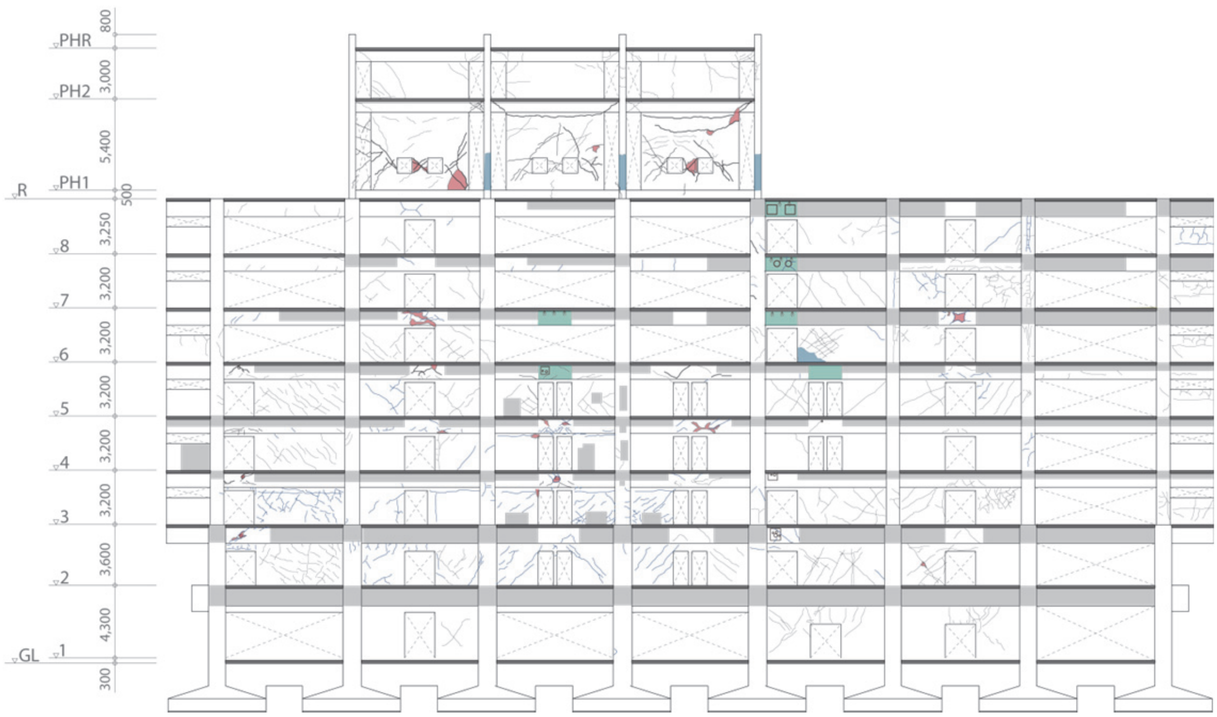

Fig. 3 Frame I cracks schedule

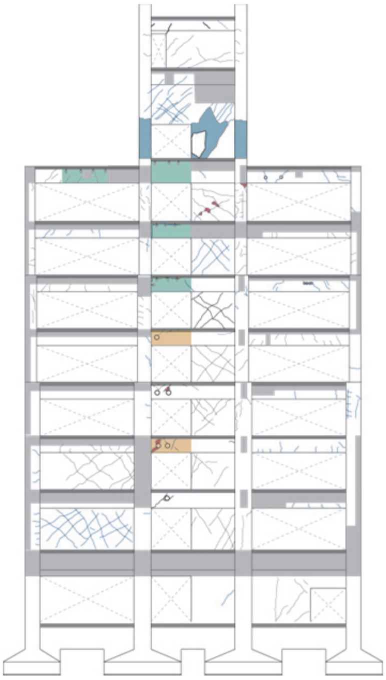

Fig. 4 Frame 10 cracks schedule

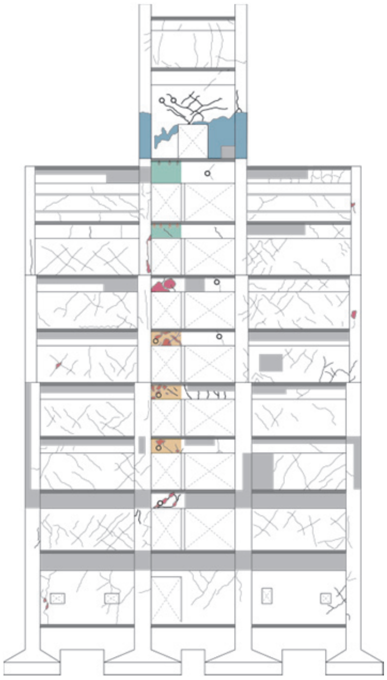

Fig. 5 Frame 11 cracks schedule

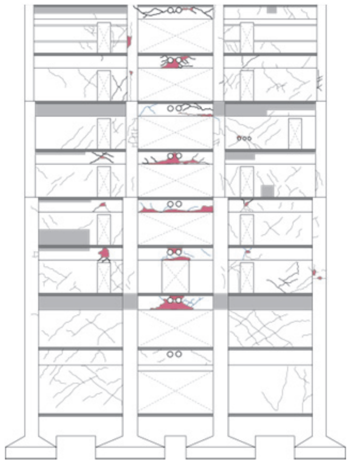

Fig. 6 Frame 14 cracks schedule

Beams

Carbon fiber reinforcement has been used in short and normal beams. The short beams took a lot of damage and failed in shear, while the other beams, even the not reinforced ones, had less damage. Different failures in the carbon fiber of the short beams can be identified such as unstuck tapes (Photo 5), peeled-off tapes (Photo 6), broken fibers (Photo 7), torn fibers (Photo 8) and failure of anchorages (Photos 9 and 10).

Photo 5 Unstuck tapes
Floor 5 Frame 12 Beam I-J

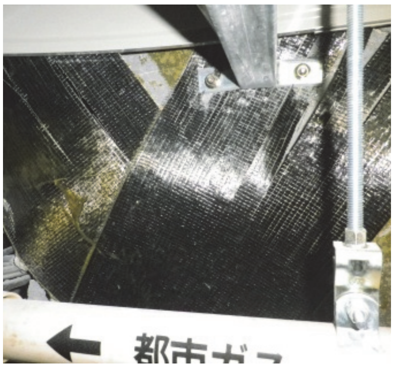

Photo 6 Peeled-off tapes
Floor 5 Frame 11 Beam I-J

Photo 7 Broken fibers
Floor 7 Frame 11 Beam I-J

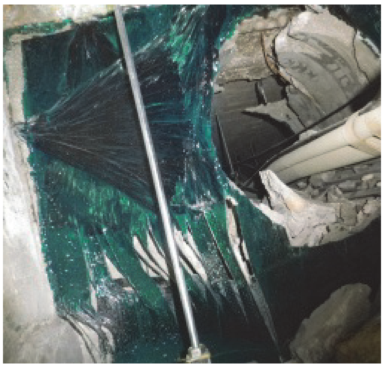

Photo 8 Torn with anchors
Floor 7 Frame 11 Beam I-J

Photo 9 Anchorage failed
Floor 6 Frame 12 Beam I-J

Photo 10 Anchorage failed
Floor 7 Frame 11 Beam I-J

The most severe damage in this group is the failure of the anchorage of two of the carbon fiber reinforcements that were visible. In the last photo the damage was so great that a rebar was visible. For clarifying the photo of the failure a zoom and a drawing are provided in Photo 11 and Fig. 7:

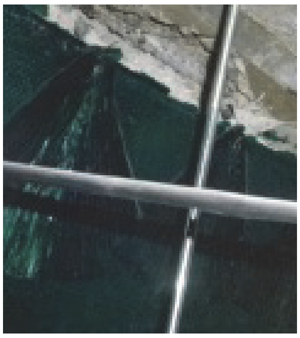

Photo 11 Zoom in failed anchorage
Floor 7 Frame 11 Beam I-J

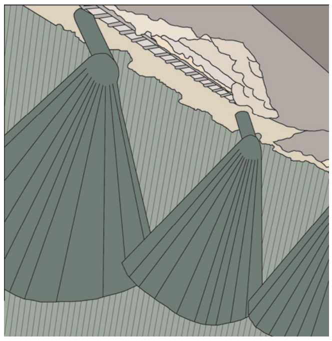

Fig. 7 Drawing of the failure in Photo 10

Figures 8, 9 and 10 show the reinforcement detail of the 400 mm x 1000 mm beam in Photos 10 and 11. It is between a column on the left side and a wall in the right one. Its longitudinal reinforcement is 25 Ø, the diagonal reinforcement is 16 Ø and the stirrups are 13Ø. The only data obtained about the anchorage is that, according to photo measurement, it is around 12 Ø.

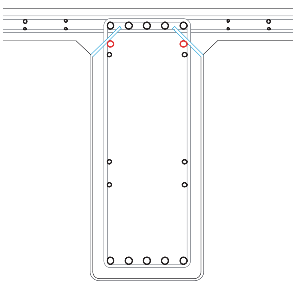

Fig. 8 Section near column

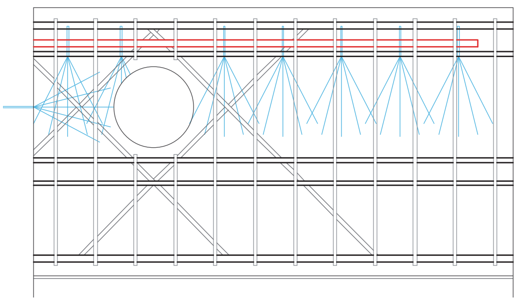

Fig. 9 Bars and anchors

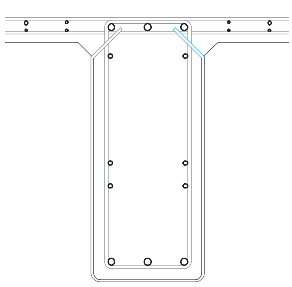

Fig. 10 Section at midspan

In addition, stirrups were visible in 26% of the beams that were not covered with carbon fiber. Therefore, the damage was so great that a high number of members had exposed bars. Also, in some cases has been detected bad concrete casting problems, especially in the beams of the fourth story.

Severe damages were also detected in the coupling beams of the frame number 14. They have two 300 Ø openings at their midspans and were highly damaged around this area (Photo 12). These openings have been detected also in all the coupling beams of the frame 11 and in some of the frame 9.

Photo 12 Coupling beams damaged around openings

Structural walls

Figures 3-6 indicate that the cracks were prominent in the walls in the 2nd and upper stories. Furthermore, most of them were narrower than 1 mm. Because flexural cracks tend to close after earthquake while the shear cracks usually remain, we conclude that flexural deformations occurred in the 2nd story and were more dominant than shear deformations.

On the other hand, the seismic retrofit documents indicate that 0% and 64% of the walls in the 2nd story were predicted to yield in flexure in X and Y directions, respectively. We conclude that the prediction was not correct both in X and Y directions.

From fifth to sixth stories the number of walls decreases in the X axis, specifically in the frame I. Their thickness has also decreased in the fifth floor. This may have caused a weakening in the sixth story and the reason of the failure of walls in the upper stories. It should be noted that in the retrofit documents, 0% of the walls in the 6^{th} story were predicted to yield in flexure in X direction, but no shear failure was observed in the story.

Compressive failure associated with flexural failure was observed in walls without boundary column as shown in Photo 13. The detail of the wall is provided in Fig. 11. The wall has double 9 Ø 150@ reinforcement. Some bars were not found in the plans but were visible at the place (Photo 14), the one on the left side may be a reinforcement at the end of the wall. According to the recent study (Takahashi et al, in press), the flexural drift capacity of RC wall can be computed as sum of elastic and plastic drifts and plastic, where the plastic drift is estimated by using the neutral axis depth, plastic hinge length and ultimate compressive strain of concrete. The calculated neutral axis depth is 1250 mm from the border of the wall and is shown in Fig. 11. The height of the observed compressive failure zone was almost 2.5 times of the wall thickness (Photo 13) as predicted by Takahashi et al (in press). From Photo 13, it is supposed that the compressive strain of concrete was very large. If we assume that the compressive strain was from 1% to 2%, the flexural drift is estimated from 0.6% to 1% by the methodology proposed by Takahashi et al. Therefore, actual lateral drift of this building is assumed to be comparable to these values.

Photo 13 Compressive failure
Floor 6 Frame I between frames 11-12

Photo 14 Detail of bars from the other side of photo 13

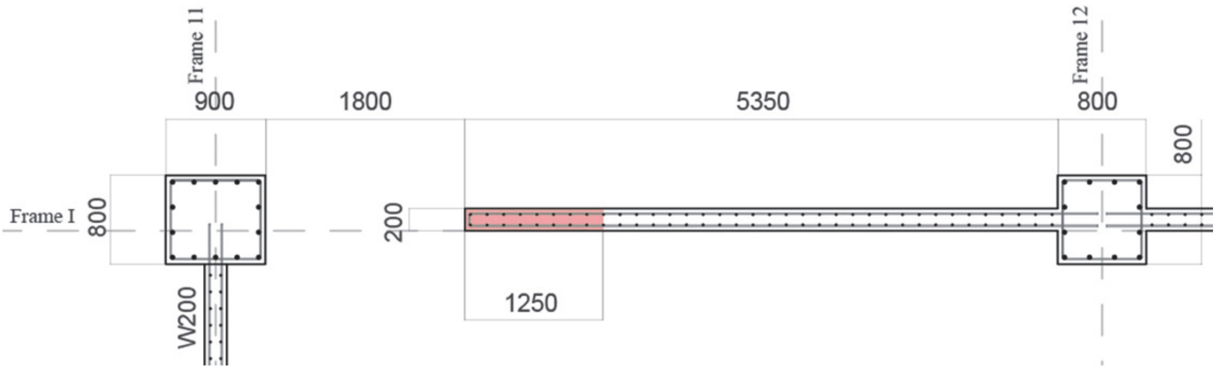

Fig. 11 Detail of the wall at 6th story, between frames 11-12 of the frame I

However, some mistakes in the retrofit documents have been found. During the retrofit the walls in the 2nd story in frame 8 between frames J-K and in frame 10 between frames H-I, and also in 3rd story in frame 10 between frames H-I were not taken into account. This negligence caused underestimation of the seismic capacity in Y direction in the 2nd and 3rd stories. In addition, the following walls that actually are not structural were taken into account during the calculations of the retrofit: 6th story in frame I between frames 7-8, 9-10 and 10-11, and 7th story in frame I between frames 7-8. These wrong suppositions caused overestimation of the seismic capacity in X direction in the 6th story.

Penthouse

The penthouse of the building was so severely damaged that required reinforcement just after the earthquake in order to prevent collapse (Photos 15, 16 and 17). Both columns and walls were completely destroyed at the first story of the penthouse, while the second story was in a similar condition to the rest of the building.

Photo 15 Column of the penthouse after the earthquake

Photo 16 Column of the penthouse after the earthquake

Photo 17 Later reinforced column of the penthouse

In the penthouse, the reinforcement bars used are 9 mm @200 double in 200 mm thickness walls and single in 120 mm thickness walls (Fig. 2). According to the pictures the used bars are all plain in the penthouse walls. In the columns the reinforcement bars are 14-D25 for the first story and 10-D25 for the second story and their stirrups are 9 Ø @150. It is also noteworthy that the penthouse of this building is quite slender. Its geometric slenderness is 1.40 and it may have affected the damaged inflicted to the walls and columns of the penthouse.

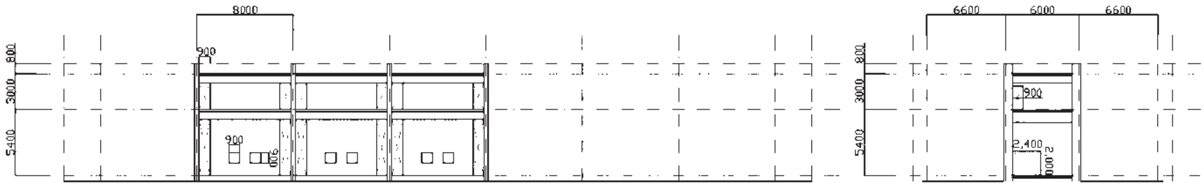

Fig. 12 Penthouse elevations

ANALYSIS

For the analysis of each story of the building the C/A_i coefficient has been calculated. In the current Japanese seismic design code (Building Research Institute 2001), Ai factor represents vertical distribution of a seismic story coefficient relative to that at the first story. Figure 13 shows the distribution of Ai factor along the height of the building; note that Ai is 1 at the first story and very large at the penthouse. Figure 14 shows the weight above each story, W. Multiplying Ai and W, we get the story shear force that corresponds to base shear coefficient of 1.

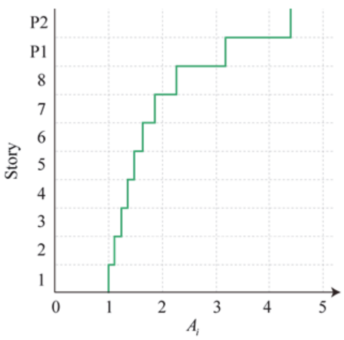

Fig. 13 Distribution of A_i

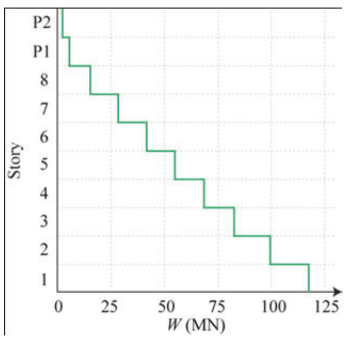

Fig. 14 Weight above each story

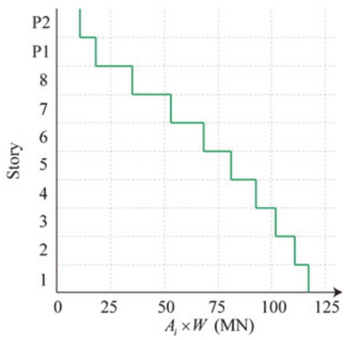

Fig. 15 Story shear force corresponding to base shear coefficient of 1

In the seismic retrofit document, strength of each column and wall is described. Adding the strengths in each story, we get the strength of each story, Q_i. Dividing the strength by A_iW in Fig. 15, we get equivalent base-shear-coefficient as shown in Figs. 16 and 17, where the contributions of the columns and walls are also indicated. In the seismic retrofit document, it is also reported that the drift capacity of each story is larger than 0.4 %. Based on the Japanese standard (Building Research Institute 2001), this building is expected to withstand strong earthquake.

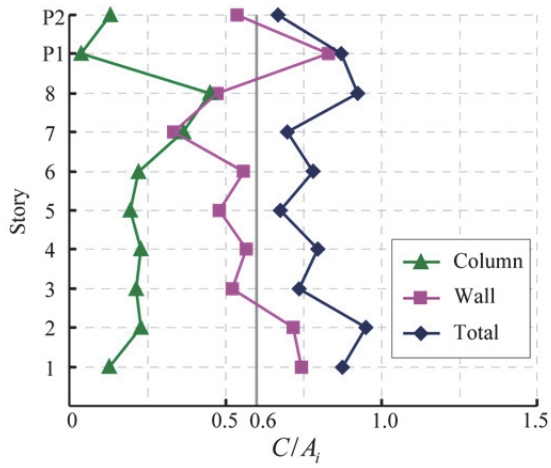

Fig. 16 X direction

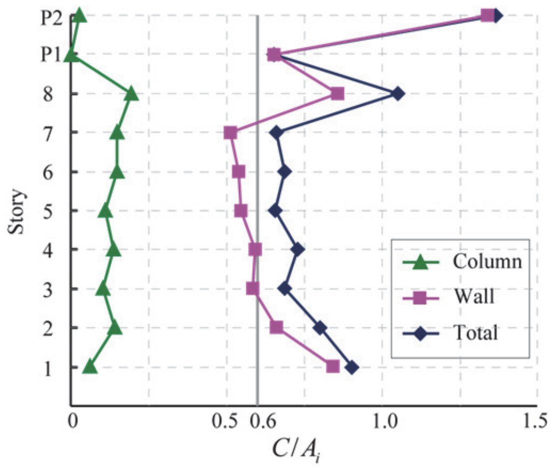

Fig. 17 Y direction

According to this graphs in X direction the second and eighth stories are the strongest ones and the second story of the penthouse is the weakest one. Referring to the Y direction the second story of the penthouse is by far the strongest, and the fifth and first story of the penthouse are the weakest.

Equivalent base-shear-coefficients are not small. On the other hand, the damage of this building was serious especially in beams and penthouse. To investigate this reason, another analysis is conducted. For the analysis of the beams and walls the model of calculation of the frame number 11 has been chosen as representative. The reasons are that it has walls in all the stories and is around the average contribution of the rest of the walls at the 2^{nd} story level (Table 1), which is the story from which the walls begin having significant damages. This table has been calculated according to results of the seismic evaluation documents, adding the contribution of each element per frame in the 2^{nd} story.

Table 1 Contribution of each frame of the second story

Frame	7	8	9	10	11	12	13	14	Average
MN	18.7	8.3	12.2	4.8	10.7	4.9	11.0	18.7	11.2

Due to the damages seen in frame 11 (Fig. 5) the short beams between frames I-J failed in shear. Therefore, the failure mode as shown in Fig. 18 is assumed to estimate the lateral capacity of this frame. In this model the walls of frames H-I and J-K are strong enough while the coupling beams fail. For the analysis the first story is considered as rigid, because it does not have significant damages. These walls rotate as shown in Fig. 18 and vertical reinforcement bars are assumed to be yielded at the bottom of the second story, where a_t is the cross section area of the longitudinal reinforcement on boundary column and σ_y is the yield strength of the steel. The values N_1 and N_2 are the long term loading axial forces through the boundary columns of the walls. The double circle in this figure represents the plastic hinge of the joint and its plastic moment is computed as sum of the plastic moment of the column and that of the beam.

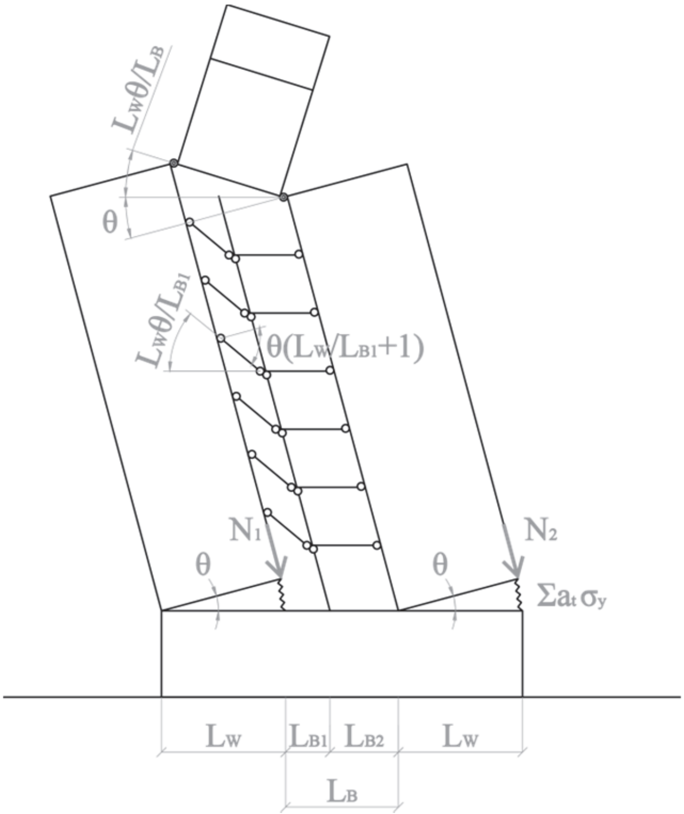

Fig. 18 Model of calculation for the frame number 11

During the analysis, two models have been proposed. In the first one the short beams of the span L_{B1} of the model are completely destroyed, and in the second one the beams yield in flexure at both ends. According to the documents of the seismic evaluation, the lateral capacity of the frame number 11 is 10.7 MN; considering the walls from the second, third and fourth story as flexural walls and the rest as shear walls. According to the model of the analysis, all the walls have been considered as flexural walls because of the hinge in the base of the walls of the model. This assumption is admissible since the small width of the cracks found in the walls of the building indicates that almost every wall yield in flexure rather than fail in shear. As to the results of the analysis, the flexural capacity of the frame number 11 is 8.7 MN, in the case of considering the contribution of the coupling beams. In the case of neglecting those beams the flexural capacity is 6.5 MN. Thus, the real value is between these two results and is a fact that the beams make an important contribution to the capacity of the frame, being around the 25% of the total capacity.

The results of the analysis are around 81% for the first case and 61% for the second one of the total capacity specified in the documents of the seismic evaluation. This suggests that the base shear coefficient of the building may be smaller than the one provided in these documents. In addition, some mistakes have been found in the calculations of the documents of the seismic evaluation. They considered some walls that actually do not exist in both directions of the building.

Regarding to the analysis of the penthouse, the results of the analysis in Figures 16 and 17 show how the damages at the first level of the penthouse could be predicted when comparing the coefficients with the ones of the other stories. However, the C/A_i coefficient of the penthouse is still 0.653, which should be enough.

The huge damage in the penthouse happened also despite the fact that the areas of the walls in X and Y directions were 2.5% and 1.3%, respectively, of the total floor area. These values are high if the penthouse is considered as a building itself. But in this case, it cannot be said these values are high because it is located on the top of the building and the amplification must be considered.

To study the penthouse the contribution of walls in each frame has been analyzed. The walls are very similar and all have columns in both sides. But due to the openings that have the frames 9, 10 and 11 reduction factors of their capacity must be taken into account (Building Research Institute 2001). Thus, they are actually around 44% and 67% of their capacity (Table 2). These differences of capacity produce an eccentricity in the transverse direction of the penthouse.

Table 2 Capacity of walls in the first story of the penthouse

Frame	PH1		
	Q_{su} (MN)	γ reduction factor	$\gamma\ Q_{su}$ (MN)
8	4.3	1.00	4.3
9	4.3	0.44	1.9
10	4.3	0.67	2.9
11	4.3	0.67	2.9

CONCLUSIONS

(1) In the transverse direction, one half of the frames had coupling beams. More than 50% of these beams had 300 Ø openings and failed in shear, including those retrofitted using carbon fiber. One of those frames was analyzed assuming rigid plasticity of each element and neglecting the contribution of the coupling beams. The computed strength was approximately 61% of that obtained in the document of the seismic retrofit.
(2) In the longitudinal direction, a wall in the sixth floor yielded in flexure with compressive failure of wall panel. The wall panel was not provided with a boundary column. In the document of the seismic retrofit, it was predicted to fail in shear.
(3) The penthouse had some eccentricity in the transverse direction. This may have caused a torsional response of the penthouse which damaged more in the transverse direction than in the longitudinal one.

More research needs to be done to understand the behavior of this structure at the Tohoku earthquake.

ACKNOWLEDGEMENTS

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