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DAMAGE OF RC BUILDING WITH COUPLED SHEAR WALLS CAUSED BY THE 2011 GREAT EAST JAPAN EARTHQUAKE

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Abstract: An eight-story reinforced concrete building retrofitted according to 1990 Japanese Standard was severely damaged during the 2011 Great East Japan Earthquake. The building was constructed in 1966 at Tohoku University. In 1996 the building was retrofitted using structural walls and carbon fiber. Dramatic damages were observed in coupling beams, a penthouse and a structural wall. Coupling beams failed in shear and bond. Columns and walls in the penthouse collapsed by torsion. The structural wall failed in compression at its end. Some of structural members were different from original drawings. Therefore, their strengths are re-calculated considering these differences by using 2010 Japanese Standard. Based on these calculated strengths, a pushover analysis is conducted. The failure mode in the analysis agrees with observed failure. Furthermore, it is confirmed that perpendicular walls cause large deformation of coupling beams.

1. INTRODUCTION

This paper deals with a reinforced concrete (RC) building in Tohoku University built in 1966, retrofitted in 1996 and demolished in 2012. The building had eight stories and a two story penthouse, and suffered the 2011 Great East Japan Earthquake. While it was expected to get enough strength by retrofit, the damage of this building was serious in penthouse and beams with openings. And it had been demolished

There are many reports for the damage on coupling beams due to previous earthquakes (Mitchell *at el.*, 1995). The coupling beams of this building had some difference from these reported beams: the aspect ratio of the beam was large, coupling beams had openings, and the damage concentrated on the limited area of the beam.

In addition, there was non-structural perpendicular wall between this damaged area and the other area in the coupling beam. There might be some effects from the perpendicular wall. The penthouse was also greatly damaged, though the strength of penthouse was estimated to be enough (Villanova *et al.*, 2011).

This paper reports observed damages based on a field investigation and discusses the cause of these damages by analyzing the frame in transverse direction.

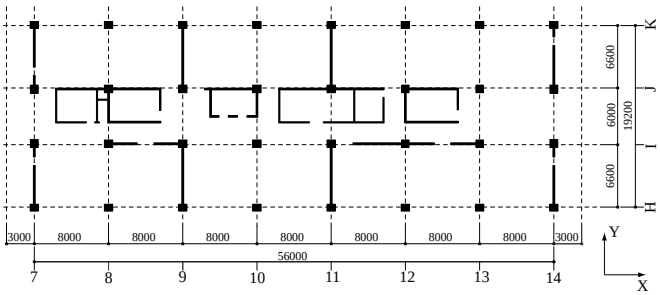


Figure 1 Typical floor plan in upper stories

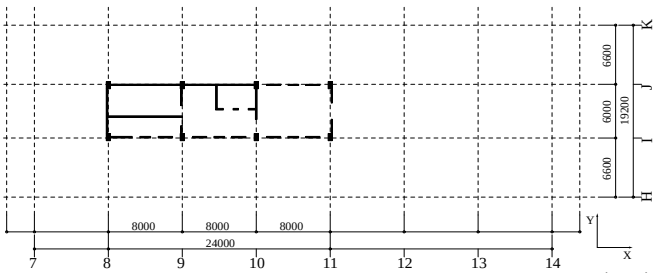


Figure 2 Floor plan of 1st story of the penthouse

2. CONFIGURATION OF THE BUILDING

The building was built in 1966 in Tohoku University, Sendai. It had eight stories and a two-story penthouse. It had seven spans (Frame 7~14) in X direction and three spans (Frame H~K) in Y direction. Figures 1 and 2 show the plan view of the typical upper floors and floor plan of the 1st story of the penthouse. In transverse direction, some frames had coupled shear walls with coupling beams. Beams had some openings for ducts.

The building was retrofitted in 1996 referred to the 1990 Japanese standard (building Research Institute 2001). The main changes during the retrofit were addition of new walls, replace RC walls for new and thicker ones and reinforcing some of the beams with carbon fiber. Addition and replacement of RC walls were described in the seismic retrofit document, but reinforcement using carbon fiber was not. The wall changes were concentrated in lower stories, from first to fifth stories, the carbon fiber reinforcement had been located mostly in the upper ones. About the location of the carbon fiber reinforcement, since there was no documentation, a deep field investigation was needed. The carbon fiber reinforcements were applied around openings in the beams. There were two kinds of fiber reinforcement: the first one is tape-shaped, the second one was a continuous fiber wrapping all beams with anchorages inside the concrete. (Photos 1 and 2) Regarding to the penthouse, no retrofit was done.

Photo 1 Tape-shaped fiber

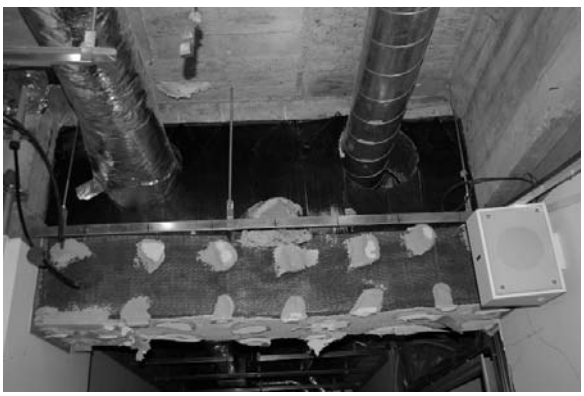


Photo 2 Continuous fiber

Figures 3 and 4 illustrate representative sections of columns and beams. The used longitudinal reinforcements were D25(#8) or D22(#7). The used stirrups were ϕ13 or ϕ9. The used materials were concrete of 18 MPa, longitudinal bars of 345 MPa for girders and columns, and 235 MPa for other elements.

In the seismic retrofit document, strength of each column and wall was described. In brief, the seismic retofit document assumes that this building collapses by columns and walls failure. The strength of each story is obtained as sum of strengths of these members. Dividing the strength by *AiWi*, we get equivalent story-shear-coefficient *C*/*Ai* as shown in Figure 5, where the contributions of the columns and walls are also indicated. 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; note that *Ai* is 1.0 at the first story and very large, approximately 4.0, at the penthouse. *Wi* factor represents the weight above each story.

Based on the Japanese Standard, this building was expected to withstand strong earthquake. According to this graphs in X direction the second and eighth stories were the strongest ones and the second story of the penthouse was the weakest one. Referring to the Y direction, the second story of the penthouse was by far the strongest, and the fifth and first story of the penthouse were the weakest. However, equivalent story-shear-coefficients in the weakest storys were more than 0.6. According to Japanese Standard, these values are estimated enough strength. On the other hand, the damage of this building was serious especially in beams and penthouse.

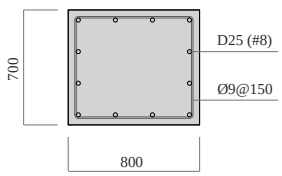


Figure 3 column section

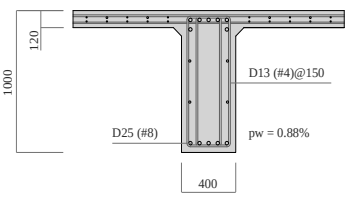
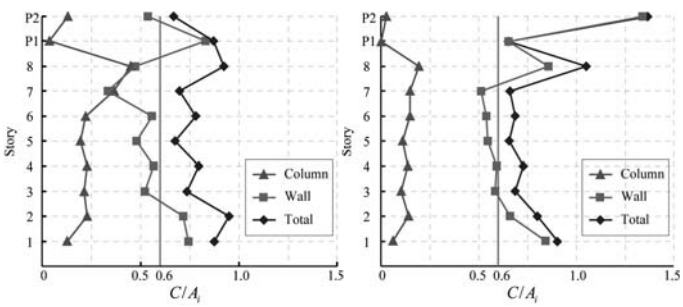


Figure 4 beam section



(a) X direction (b) Y direction

Figure 5 Equivalent story-shear-coefficient

3. OBSERVED DAMAGES

3.1 Coupling Beams

3.1.1 Frame on Axis 14

Figure 6 illustrates that severe damages were detected in these coupling beams of the frame number 14. They have two openings near their midspans. They failed in shear at the center of beam at 3rd, 4th, 6th and 8th stories, and bond failure at the 5th, 6th, 7th stories. About bond failure they have two patterns. 5th and 6th stories failed from end to end of the beams and, 7th stories failed not center but ends.

Bar arrangements of the beams were visible because concrete had spalled. They were different from the drawing. In the drawing, the longitudinal reinforcements were D25(#8) or D22(#7) bars, diagonal reinforcements were D16(#5) bars and stirrups were ϕ13 or ϕ9 bars(Figures 7 and 8). Lack of cover concrete for window frame was expressed in the figures. Figure 9 illustrates the drawing of reinforcements around the openings of 300mm in diameter. The distance between two openings was 800mm.

Photo 3 and Figure 10 show the beam of the third floor. From these investigations, the section of this beam with openings can be estimated as Figure 11. The diameter was not constant along the beam depth and it was larger in outside part of the beam. In addition, the distance between of these openings was 600mm which was estimated from photo. There were no stirrups under

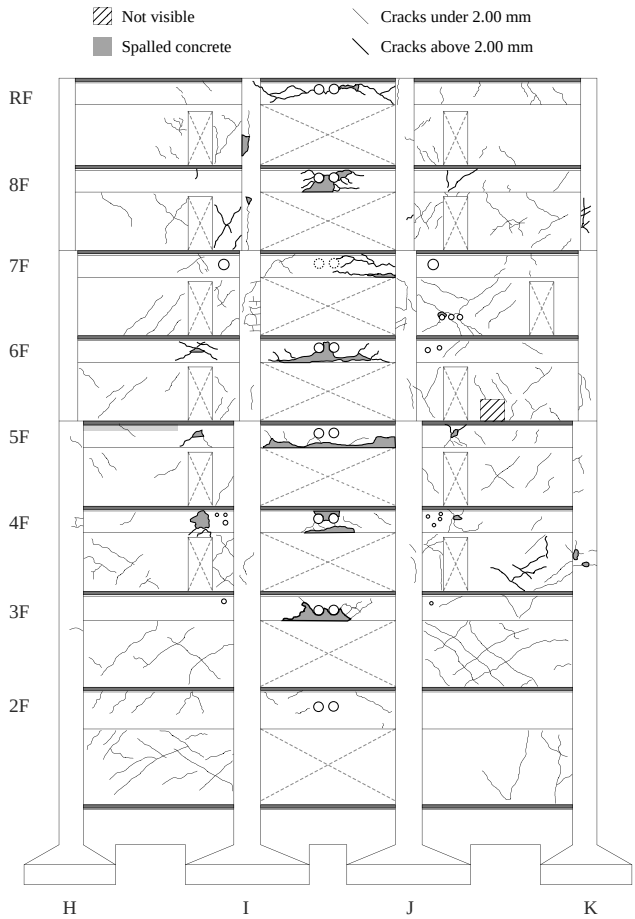


Figure 6 Frame 14 cracks schedule

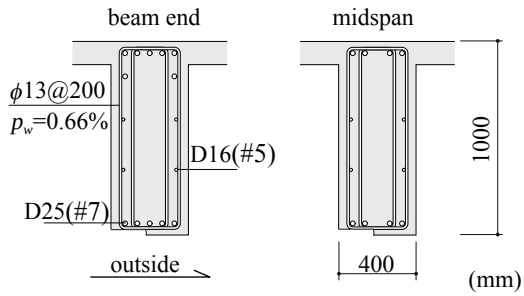


Figure 7 Section of the beam of the floor 4,5

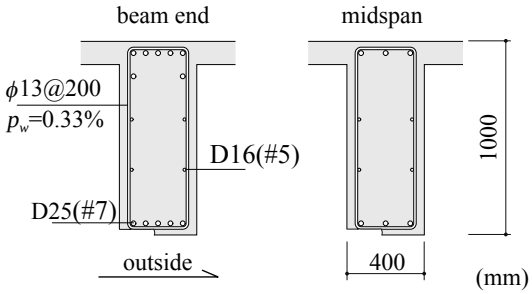


Figure 8 Section of thebeam of the floor 6,7

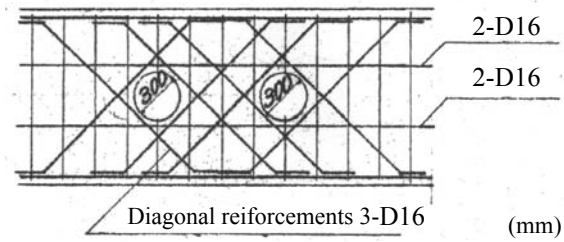


Figure 9 Drawing of reinforcement for opening

Photo 3 Beam of the 3rd Floor

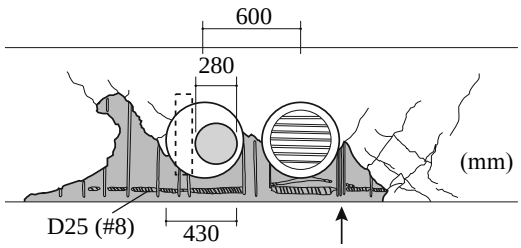


Figure 10 Detail of beam of the 3rd Floor

openings. The arrow in Figure 10 illustrates three stirrups were gathered beside the opening. In the broken area in this figure, there were no stirrups because they were cut for the opening.

Photo 4 and Figure 12 describe the beam in the fourth floor. It was different from third floor where some stirrups were gathered beside the opening. Stirrups were cut above the opening as shown in the broken area. In addition, straight anchorage of diagonal reinforcement was different from drawing which adopted bent bars. Diameter of this opening was constant along the beam width. (Figure 11)

The beam of fifth floor is provided in Photo 5. This failed in bond from end to end. Figure 13 shows the beam bottom surface viewed from below. The upper part of the figure is outside and the lower part is a window frame. Deformed bars are the longitudinal reinforcements in the beam. It shows that window frame was attached to the longitudinal reinforcement, and stirrups were cut as shown in the circle. These stirrups were cut to get the window frame into position. It was estimated that the beam got little covering depth of concrete for window frame.

The beam of 7th floor failed bond at both sides, and not shear around openings. Photo 6 was taken from inside of the building. It was detected that the beam had no opening at the 7th floor where as the ducts in the openings were found in 6th floor as shown in Photo 7.

These differences between the drawings and the practice might have caused such failure.

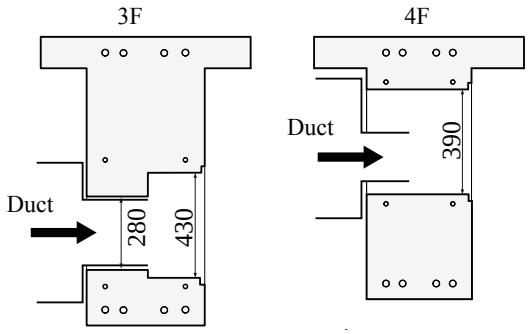


Figure 11 Image of the beam section across the opening

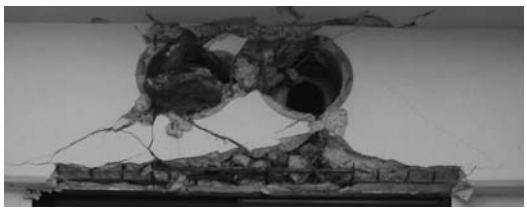


Photo 4 Beam of the 4th Floor

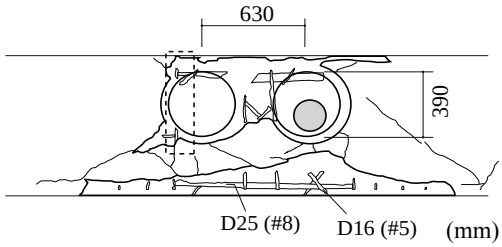


Figure 12 Detail of beam of the 4th Floor

Photo 5 Beam of the 5th Floor

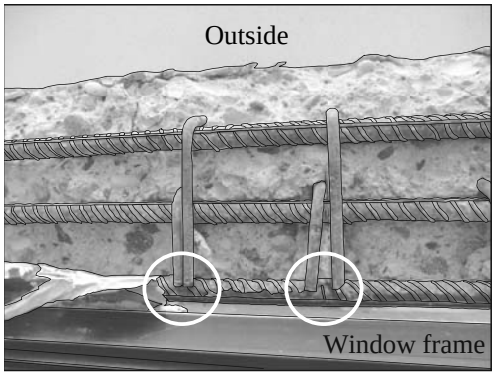


Figure 13 Look up to the bottom of the beam

Photo 6 Beam of the 7th Floor from inside

Photo 7 Beam of the 6th Floor from inside

3.1.2 Frame on Axis 11

The frames 11 were constituted of shear walls and coupling beams between these walls. Figure 14 illustrates that coupling beams of second floor and roof floor had less damage. But those of other floors suffered heavy damage as shown Fig. 15. They had perpendicular walls under coupling beams. On the South side of this perpendicular wall, the coupling beam failed in shear whereas the other side had less damage. It indicates that the coupling beam was restrained by the perpendicular wall.

Some beams were reinforced by carbon fiber around openings. The carbon fiber used to prevent shear failure. But it did not work appropriately because the anchorage length of continuous carbon reinforcements was not enough. (Photo 8) They usually need 100mm anchorage length, but actual anchorage length was 40mm. The tape-shaped fiber of the short beams were unstuck and peeled off.

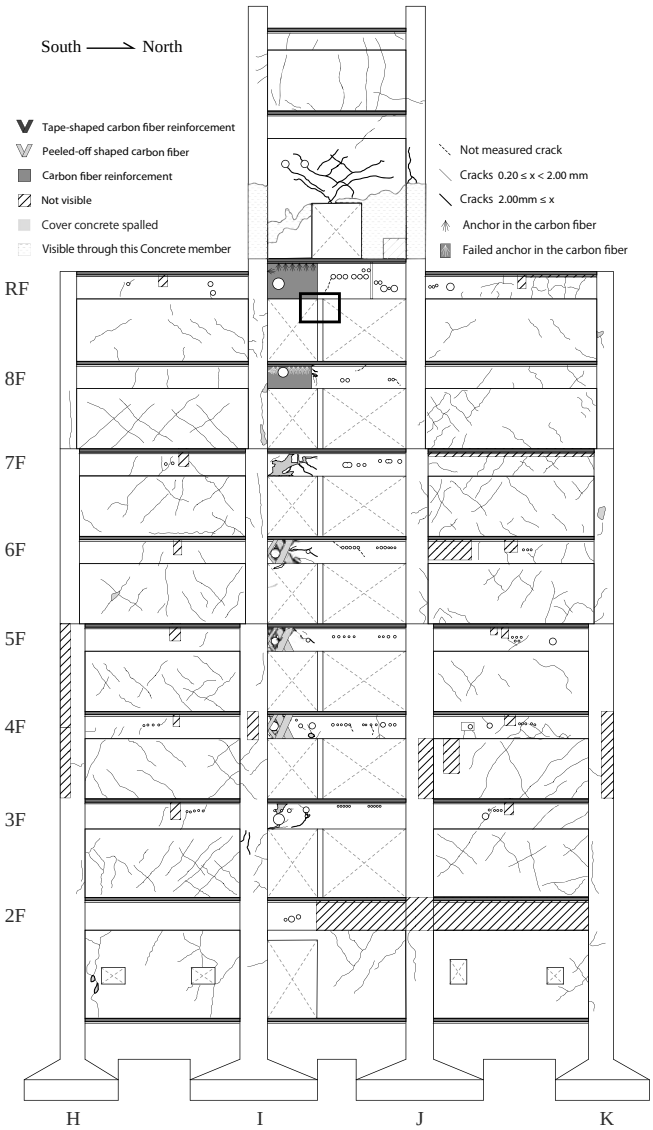


Figure 14 Frame 11 cracks schedule

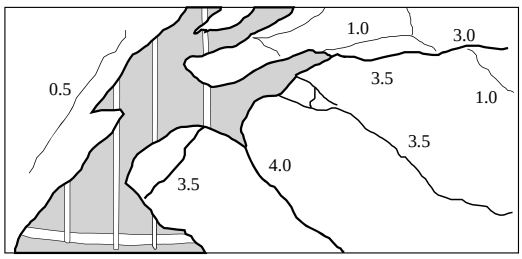


Figure15 damage of short beam of 7th floor

3.2 Penthouse

The penthouse of the building was severely damaged that required emergency strengthening just after the earthquake in order to prevent collapse. 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. In investigation, thickness of walls was 120mm, but drawing illustrates that was 180mm. (Figure 16) Damage in the first story of the penthouse was more remarkable on transverse directions than on longitudinal one. Figure 17 shows penthouse cracks schedule of each frame. Frame 8 was less damaged. Frame 11 damaged heavily.(Photo 9) Damage in east area became increasingly hard. Capacity of walls in the first story of the penthouse was calculated in each frame as shown in the work by Villanova et al. (2012). The frame 8 had approximately twice the capacity of 9, 10 and 11 frames. Therefore, Penthouse might have torsional response. This might have caused such severe damage.

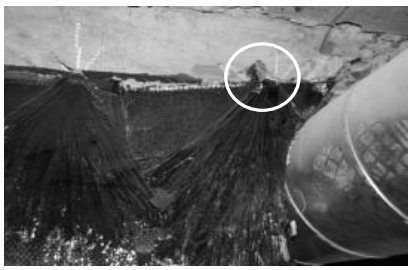


Photo 8 Failed anchorage at coupling beam floor 8

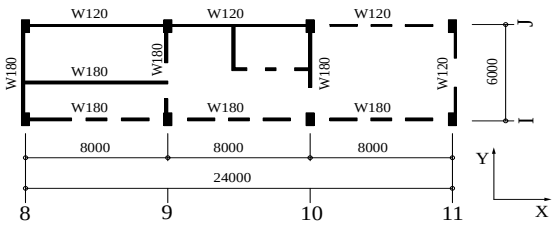
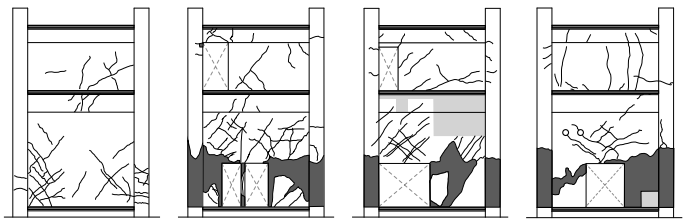


Figure 16 Penthouse 1st story (mm)

Photo 9 Wall at the first story of the penthouse



(a) Frame 8 (b) Frame 9 (c) Frame 10 (d) Frame 11

Figure 17 Penthouse cracks schedule

3.3 Structural Walls

Figure 18 illustrates frame I cracks schedule in longitudinal direction. There were few cracks in the 1st story on both X and Y directions. In the 2nd and upper stories, cracks were prominent. However, they were smaller than those of beams. Most of them were narrower than 1mm.

The pullout of bars were observed at the top of the perpendicular wall in 8th floor which is shown in Figure 19 and the square in the Figure 14. Perpendicular wall might constrain vettical displacement of the coupling beam at the mid span, and, as a result, clear span of the beam was shortened and failed in shear. Pullout of bars is a evidence of the behavior

Figure 19 Bar pullout at top of the perpendicular wall in 8th floor

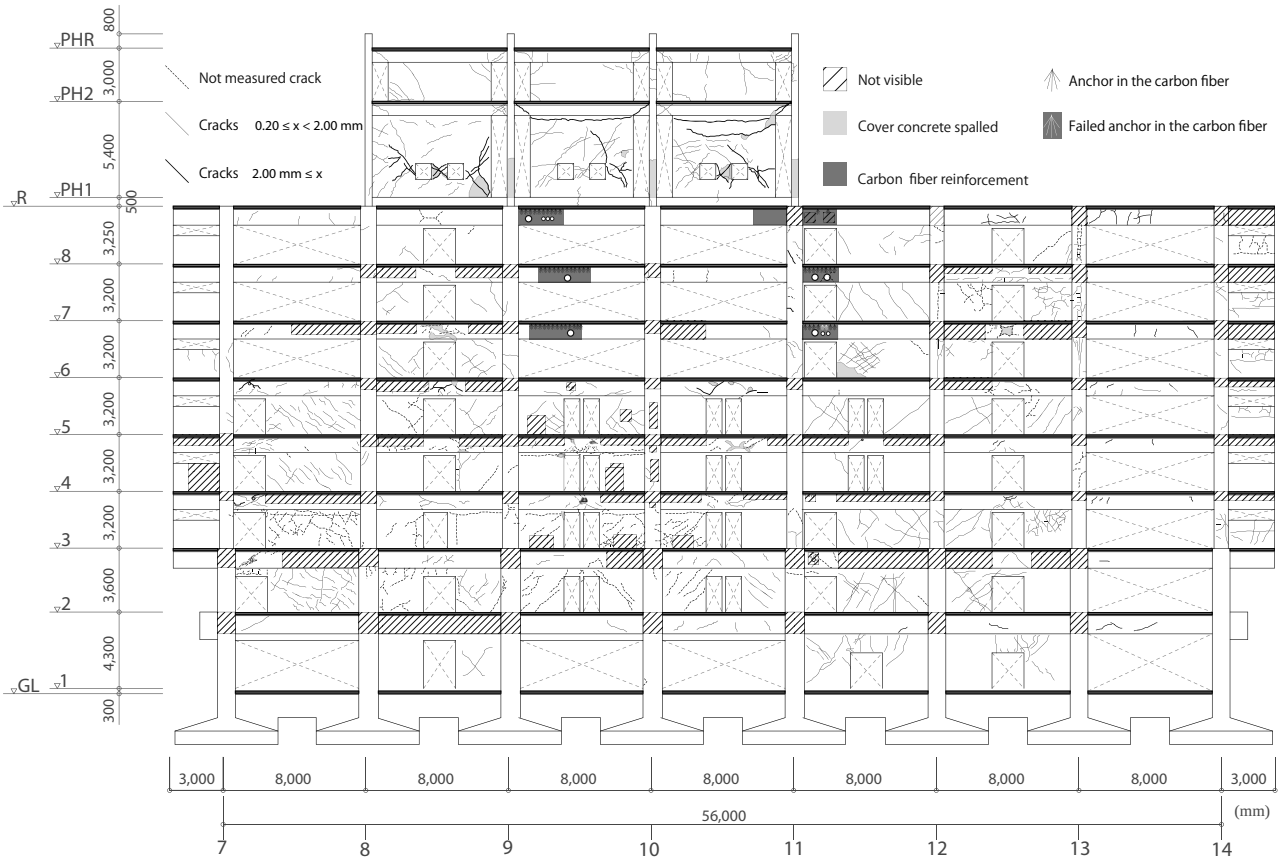


Figure 18 Frame I cracks schedule

4. PUSHOVER ANALYSIS

In this section, pushover analysis is carried out to know failure mode of this building in transverse direction, behavior of coupling beams, penthouse and perpendicular walls, and to estimate the seismic capacity of this building.

4.1 Modeling of Structural Members

4.1.1 Coupling Beams

The analytical model of the frame on 11 axis is shown in Figure 20. Coupling beams are idealized by two nonlinear rotational spring at their end and nonlinear shear spring. Coupling beams of 3rd to 8th floors are idealized with two separated beams to consider the effect of the perpendicular walls. The backbone curves of rotational springs and shear springs are represented by a tri-linear relation. The first break point in the relation represents cracked state, the second represents ultimate limit state.

There are some openings whose diameters are from 50 to 400 mm in short-span coupling beams. The shear strength of these beams are calculated considering only openings larger than 100mm in diameter. Some short-span coupling beams are retrofitted by carbon fiber. However, because the anchorage length of fiber reinforcement is shorter than demanded length, the strength contribution of this fiber reinforcement is ignored.

On the other hand, the coupling beam in roof floor is idealized a beam which is not separated and by rigid beam element with two rotational springs at its ends. These rotational springs represent the joint between columns in 8th floor and coupling beams in roof floor. Because these columns and beams are boundary columns and boundary beams, the joint fails faster than these members. However, the coupling beam depth in roof floor is twice as big as the column width in 8th floor. In brief, the column do not contributes very much to the strength and stiffness of the joint. Therefore, the backbone curves of these rotational springs use the strength and stiffness of coupling beam. The beam with wing wall in the 1st floor is also idealized by rigid beam element with two rotational springs at its ends.

4.1.2 Structure Walls

The structure wall is substituted a brace model equal to the shear stiffness of the wall. Shear behavior of the wall is idealized by two diagonal truss elements. Flexural behavior is idealized by two vertical truss elements on each side. All beams with the structure wall are assumed to be rigid.

The backbone curves of the brace model are represented by a tri-linear relation and, boundary columns are represented by a bi-linear relation. However, in the case of a wall with openings, the strength is reduced by the reduction factor based on Japanese Standard (2010).

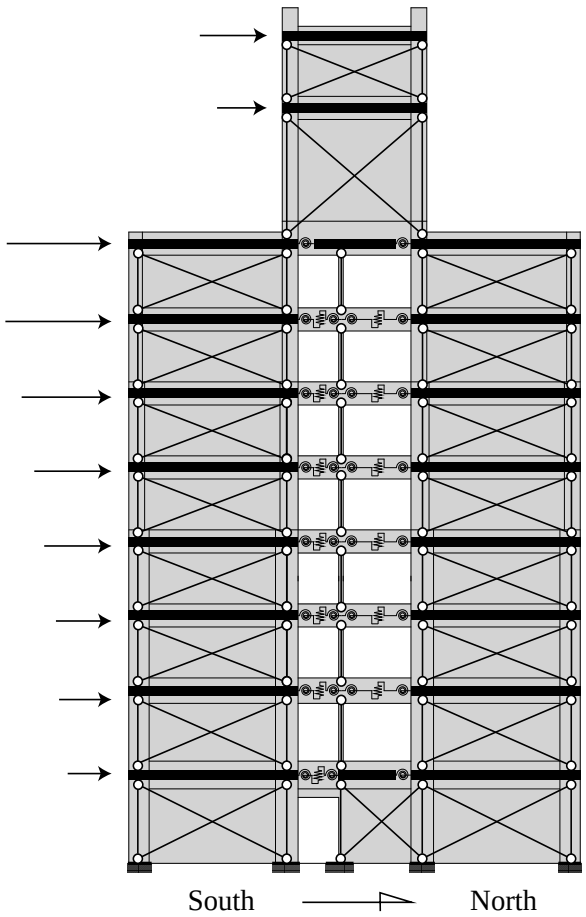


Figure 20 Analysis model

4.1.3 Perpendicular Walls

Perpendicular walls are idealized as truss elements because they do not resist to bending moment in transverse direction. The backbone curves of perpendicular walls are represented by a bi-linear relation. The compressive strength of this wall is calculated considering wall reinforcements and concrete. The tensile strength is calculated considering only wall reinforcement. However, pullout of bars is observed at the top of the perpendicular wall in 8th floor. Therefore, the tensile strength of this wall is assumed to be zero. Furthermore, the effective width of these walls in perpendicular direction is defined as shown in Figure 21. In Japanese RC Standard, a slab contributes the strength and stiffness of the beam within the width of 0.1 multiply the span length of the beam. Therefore, It assumes the perpendicular wall within the width of 0.1 multiply the clear height to resist effectively the compressive and tensile force.

4.2 Results of Analysis

Pushover analysis of the model was carried out using computer program Opensees. In this analysis, gravity loads and lateral forces are applied to each nodes in each floor. The analysis is continued to be incremented by a step to reach 1000 steps. Lateral Forces are incremented 0.001 multiplied by maximum lateral forces by every step. Therefore, maximum lateral forces are applied if the analysis reaches until final step. Maximum lateral forces are calculated by maximum story shear forces Q_{max}. Q_{max} factor represents *Wi* multiplied by *Ai*. The analysis is carried out for two cases. These parameters are the direction of lateral forces (direction from south to north, and its opposite direction).

These analyse are stopped around 500 step, because convergence test is failed. In each analysis, structure walls in 1st floor is failed in flexure. When walls are failed, story shear forces reach 0.48 Q_{max}. Therefore, the

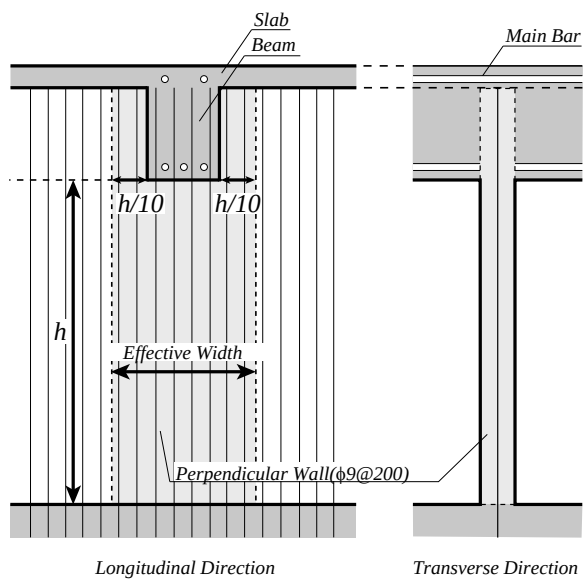
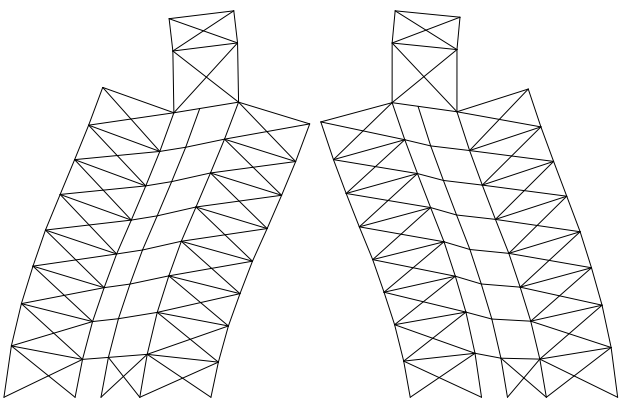


Figure 21 Effective width of perpendicular wall

base shear coefficient of this building is estimated as 0.48. Figures 22 shows the deformation of the building in the case that story shear forces are applied 0.48 Q_{max}. These displacements are illustrated by 50 magnifications. Drifts of short beams seems almost the same as long beam in Fig. 22(a). On the other hand, drifts of short beams seems larger than long beams in Fig. 22(b), Therefore, it is considered that short beam is damaged by the loading from North to South.

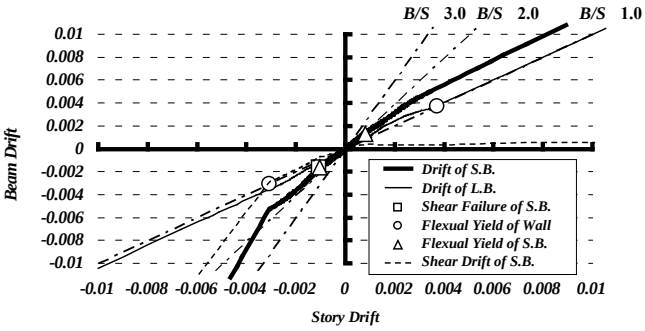
Figure 23 shows the relation between beam drift and story drift on the lower floor (1F), middle floor (4F) and upper floor (7F). Beam drift means the summation of shear drift and flexural drift. These figures show drifts in the case of both analyses. The drift of rotational spring nearby structure walls is considered larger than that nearby perpendicular walls. Therefore, flexural drift is estimated by using the rotational spring nearby structure walls. According to these relation in the area of loading direction from north to south, when walls in 1st floor failed, the drift of short beams is 1.5 to 2.0 times as large as story drift. The higher floor up, the lower the drift of short beams. On the other hand, the drift of long beam is almost the same as story drift. Figures also show the shear drift of short and long beams. It represents that the shear drift of short beams is larger than long beams. In the loading direction from south to north, the drift of short beams is almost the same as the drift of long beam. Therefore, the larger shear force acts to short beams compare to long beams in the case of loading direction south to north.

Figure 24 shows the relation between Q/Q_{max} and drifts. Q factor indicates the story shear force in each steps. Bold line indicates the drift of the building, the other indicates the drift of the penthouse first floor. The drift of the building in this graph mean the horizontal displacement at roof floor divided by the height between first and roof floor. Positive area of Q/Q_{max} in this graph represents the analysis of loading direction from south to north, opposite area represents the analysis of opposite direction. When walls in 1st floor failed, the drift of the building is about 0.65%, and the drift of the penthouse 1st floor is nearly zero in both loading directions. Even

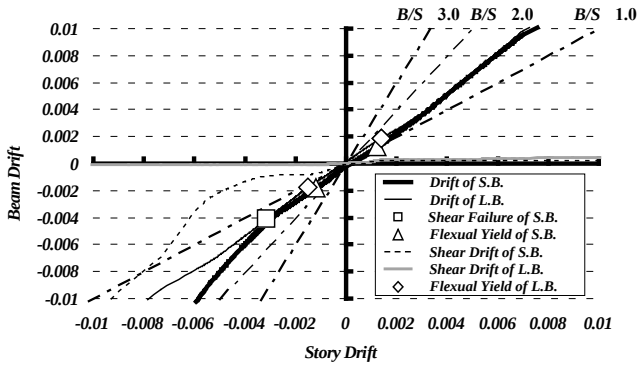


(a)South→North (b)South←North

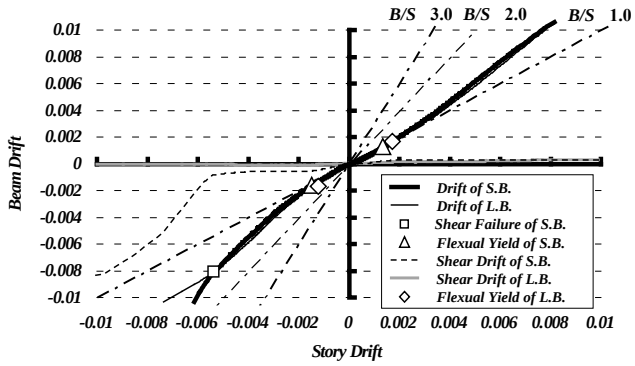
Figure 22 The deformation of the building at 0.48 Q_{max}



(a) 1st story



(b) 4th story



(c) 7th story

Figure 23 The relation between beam drift and story drift

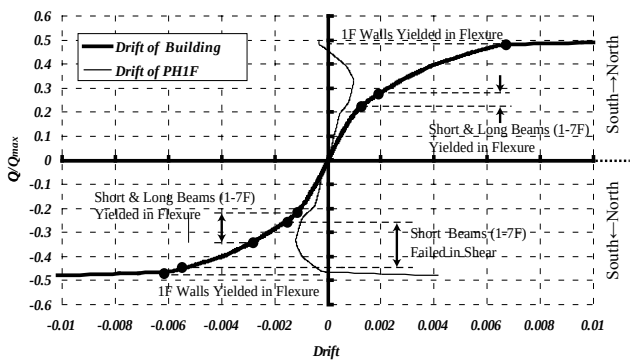


Figure 24 The relation between Q/Q_{max} and drifts

maximum drift of penthouse before walls in 1st floor failed is about 0.1%. Furthermore, in the loading direction from North to South all short beams except roof floor beam failed in flexural and shear before wall failed.

Figure 25 shows the relation between building drift and axial force of perpendicular walls. The figure represents that the perpendicular wall in 8th floor failed in bond slippage when the building drift reaches about 0.002 in south to north direction, and 0.004 in opposite direction.

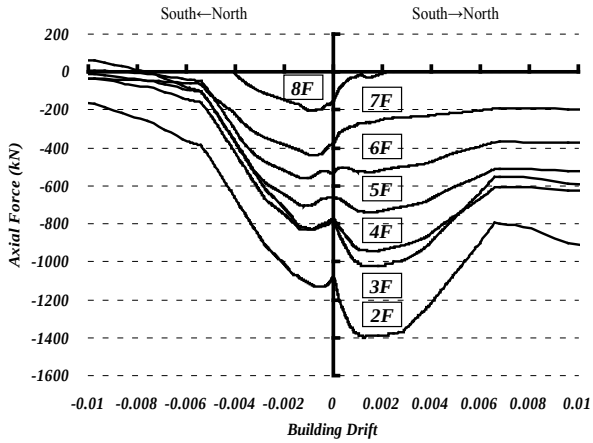


Figure 25 The relation between building drift and axial force of perpendicular walls

4.3 Discussion

Collapse model of this building in 11 axis is assumed as shown in Figure 26 and the ultimate lateral strength was calculated two patterns. The one pattern considers the work of short beam, the other one does not consider it.The strength was evaluated by using virtual work method. In this calculation, there are four assumption. First, structure walls in 2nd floor are failed in flexure. Second, the first story is considered as rigid, because it does not have significant damages. Third, top of the perpendicular wall in 8th floor is failed in bond slippage. Fourth, joints between columns in 8th floor and coupling beams in roof floor failed as shown in Fig. 27. In the result of this calculation, the ultimate lateral strength is 6.5MN without considering the effects of the beams, and 8.7MN in the other case. Furthermore, short span coupling beams are deformed 3.75 times as large as the drift of the building. According to the documents of the seismic evaluation, the ultimate lateral strength at 2nd floor is 10.7MN, and base shear coefficient is 0.8. Therefore, base shear coefficient of this calculation is estimated 0.49 and 0.65.

According to the result of pushover analysis, several behaviors of the analysis correspond with the suggested model. Regarding to ultimate lateral strength, the result of analysis is almost same as the suggested model's, but is smaller than the value of seismic evaluation. However, there are more walls in 1st than upper stories on other axes shown in Figure 28. Therefore, larger base shear coefficient than 0.48 can be expected in this building. Though drifts of short beams of the analysis are smaller than the model in Fig. 26, they are larger than story drifts as indicated in Fig. 26. Perpendicular wall in 8th floor failed in bond slippage in the pushover analysis and

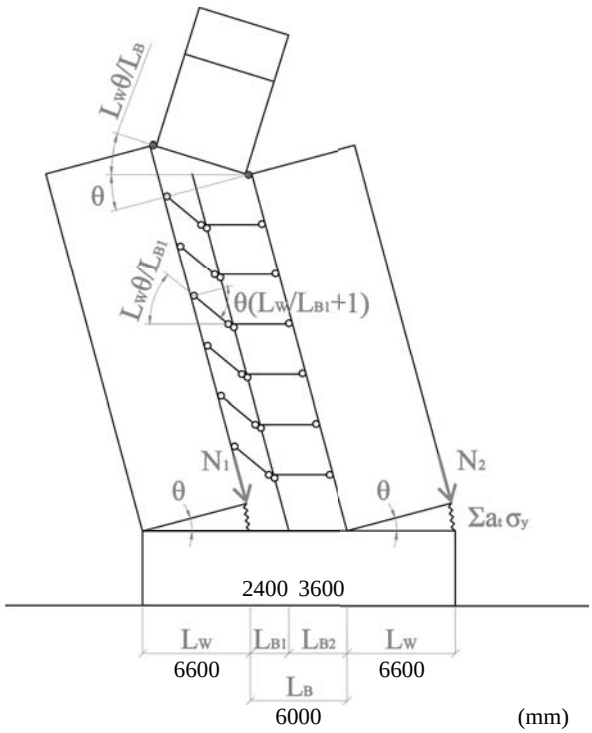


Figure 26 Model of calculation for the frame number 11

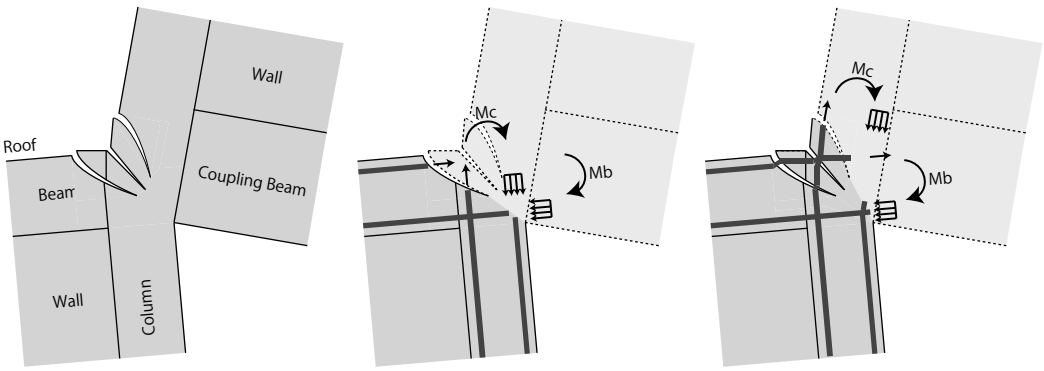


Figure 27 Assumption of the failure mode of the joint

this failure mode is also observed in field investigation. From field investigation, the failure of the penthouse is assumed to be caused by the torsional response. This failure can not be explained by the frame analysis. In fact, the result of pushover analysis shows the drift of the penthouse is much smaller than that of the main building and the penthouse does not fail.

Although the perpendicular wall is usually neglected in structural design, it has a significant role on the behavior of the coupling beam in the analysis.
It makes the short-span beam in the analysis and the failure in the analysis is similar to the observed failure.

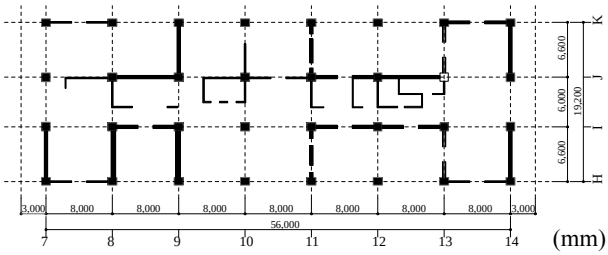


Figure 28 Floor plan of 1st story

5. LESSONS LEARNED AND CONCLUSIONS

(1)There were different points from the original drawings in coupling beams. It is one of the reason of significant damage of them.
(2)Concerning the penthouse, the capacity of wall on each frame varied widely in the transverse direction. The penthouse might have torsional response.
(3)According to the result of analysis, the building has at least 0.48 base shear coefficient. It is inferred that actual lateral forces were not over the base shear coefficient of this building because the damage of structure walls and columns were minor.
(4)Coupling beams were constrained by the perpendicular wall. It causes the large deformation of the short span beam.
(5)Short-span beam should be reinforced appropriately.

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