10CUEE CONFERENCE PROCEEDINGS
10th International Conference on Urban Earthquake Engineering
March 1-2, 2013, Tokyo Institute of Technology, Tokyo, Japan

Damage of Six-Story RC Building with Coupling Beams Having Aspect Ratios Larger than 2.0

Yuko KATO[1], Atsushi NAGAYA[2], Kazuyoshi HOTTA[3], Ying WANG[4], Susumu TAKAHASHI[5], Masaki MAEDA[6], Santiago PUJOL[7], Toshikatsu ICHINOSE[8]

1) Graduate Student, Department of Architecture Engineering, Nagoya Institute of Technology, Nagoya, Japan
2) Graduate Student, Department of Architecture Engineering, Nagoya Institute of Technology, Nagoya, Japan
3) Undergraduate Student, Department of Architecture Engineering, Nagoya Institute of Technology, Nagoya, Japan
4) Graduate Student, Department of Civil Engineering, Purdue University, West Lafayette, Indiana, USA
5) Assistant Professor, Department of Architecture Engineering, Nagoya Institute of Technology, Nagoya, Japan
6) Professor, Department of Architecture and Building Science, Tohoku University, Sendai, Japan
7) Associate Professor, Department of Civil Engineering, West Lafayette, Indiana, USA
8) Professor, Department of Architecture Engineering, Nagoya Institute of Technology, Nagoya, Japan

y-20116021@goo.jp, nagoyan_nitech@yahoo.co.jp, kazuyoshihotta@gmail.com, wang763@purdue.edu
takahashi.susumu@nitech.ac.jp, maeda@archi.tohoku.ac.jp, spujol@purdue.edu, ich@nitech.ac.jp

Abstract: Several buildings in Tohoku University were severely damaged by the 2011 Great East Japan Earthquake. In this paper, the damages in one of these buildings are reported and the reason of this damage is discussed. The target of this investigation was a six-story reinforced concrete building. The building was constructed in 1968 and retrofitted in 1997. This building is assumed to have enough strength to resist earthquakes in Japanese seismic evaluation. The base shear coefficient of this building is estimated to be 0.6. Severe damage was observed in short beams with web openings in upper stories. On the other hand, the mainly observed damages in columns and shear walls are thin cracks less than 1mm wide. The observed failure is different from what was considered in the seismic evaluation. Therefore, the actual strength of the frame is estimated by using the limit analysis in which the observed failure mode is assumed. In the limit analysis model, the rotation angles of short beams are much higher than those of shear walls. Shear walls might be elastic when the beam failures occurred. To prevent this failure, the coupling beam should be reinforced.

1. INTRODUCTION

A six-story Reinforced Concrete (RC) building located in the Engineering Campus of Tohoku University was severely damaged by the 2011 Great East Japan Earthquake. Remarkable damages were observed in short beams with web openings in upper stories. The building experienced several severe earthquakes since it was built in 1968, and it was retrofitted in 1997. This building was assumed to have enough strength to resist earthquakes in Japanese seismic evaluation.

The damaged short beams with web openings in transverse direction are coupling beams with stirrups (p_w= 0.18 (%)) as shear reinforcement. Shear failures of coupling beams were reported by researchers (Mitchell *et al.*, 1995 and National Research Council 1973), and it is well known that efficient shear reinforcement, such as tight stirrups or diagonal bars, is necessary. Most of the reported coupling beams have aspect ratios (clear span / depth) no more than 1.0.

In addition, various experimental programs studying the effect of shear span ratio and reinforcement of coupling beams are reported. Paulay, T. (1969) proposed a reinforcement pattern with diagonal bars to increase ductility of coupling beams. In this study, the shear span ratios ($M/Vh = l/2h$) of coupling beams were lower than 1.0. Theodosios P. Tassios *et al.* (1996) recommended that coupling beams with shear span ratios lower than 0.75 shall be reinforced with diagonal bars, and shear span ratios larger than 1.33 may use normal detailing. They also proposed alternative reinforcement patterns, with dowel bars and rhombic reinforcements.

According to the American Concrete Institute (ACI) 318-11 code (2011), diagonally oriented reinforcements are effective when coupling beams have aspect ratio less than 4.0, and coupling beams with aspect ratio less than 2.0 shall be reinforced with two intersecting groups of diagonally placed bars symmetrically.

Photo 1 Exterior view of the building (transverse side)

Photo 2 Exterior view of the building (longitudinal side)

In this building, aspect ratios of the short beams are 2.4~2.7 (except a part of rooftop beams), and these beams with stirrups as shear reinforcement experienced large deformations and severely damaged in shear. In addition, web openings through the short beams reduced their strengths.

Accordingly, the short beams of the building is a case of shear failed coupling beams with relatively higher aspect ratios compared with well-known observed damages, experimental objects and the existing code.

This paper reports observed damages based on field investigation and discusses a possible failure mechanism of the building.

2. CONFIGURATION OF THE BUILDING

The building was constructed in 1968 in Tohoku University, Sendai, located approximately 130 km from the epicenter. It is a reinforced concrete building with six stories above ground and one below and a one-story penthouse. It has seven spans in longitudinal direction and three spans in transverse direction (Fig. 1). The upper stories have the same distribution as first story plan. Storage, bathrooms and vertical communication nucleus are separated by walls in the middle longitudinal axis. The rooms are around this nucleus, giving a configuration symmetrical to the building. The penthouse has three spans between axis 4 and 7 in longitudinal direction and one span between axis B and C in transverse direction (Fig. 2).

The building was retrofitted in 1997 according to the 1990 Japanese Standard (Building Research Institute 2001). The main changes during the retrofit were addition of new walls and replacement of reinforced concrete wall panels with thicker ones for the first three stories. According to the field investigation, one of actual retrofit was different from the plan. In particular, a shear wall of longitudinal axis B, between axis 1 and 2, first story (a broken line in Fig. 1), was 180mm thick in existence and retrofitted with 100mm thicker one although it designed 120mm thick in existence and retrofitted with 130mm thicker one in the plan. But it is assumed that this difference does not affect the damages of the building. The other actual retrofits are same to the plan.

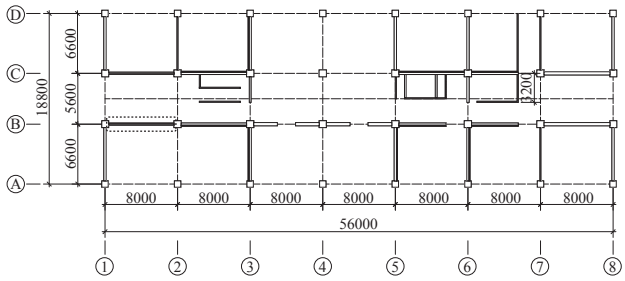


Figure 1 First story plan

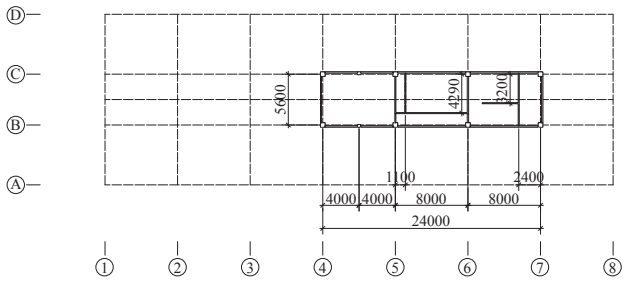


Figure 2 Story of the penthouse plan

3. OBSERVED DAMAGES

3.1. Beams

Figures 3, 4, 6 and 7 show observed cracks in selected frames. Bold lines in the figures represent cracks of more than 1mm wide. Hatched areas represent the blocked areas. In longitudinal direction, severe damages concentrated on beams above openings (Fig. 3 and Photo 3). In transverse direction, severe damages concentrated on coupling beams with web openings (Figs. 4 and 7 and Photos 4 and 5). These observed damages indicate that the short beams failed in shear. Figure 5 shows cracks observed in short beams of transverse direction (axis 3) on each story (broken lines in Fig. 4 show these

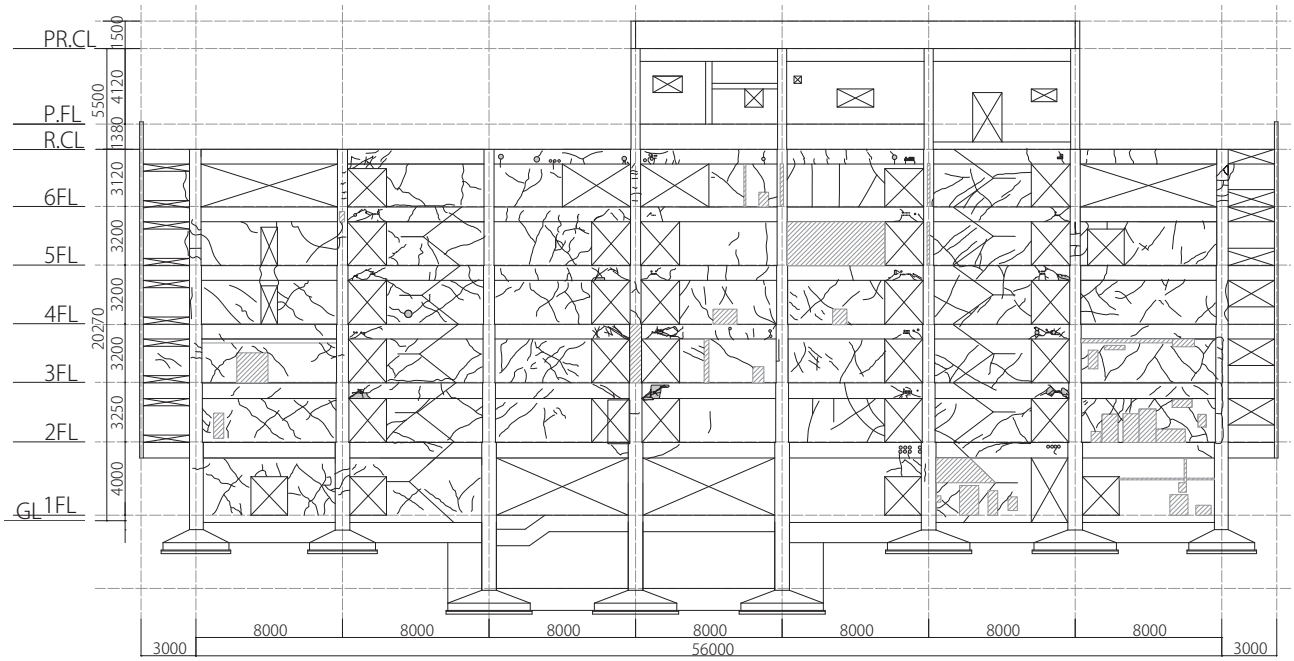


Figure 3 Observed cracks of longitudinal direction (axis C)

(a) Full view (b) Detail

Photo 3 Damaged beam above an opening (3F - axis C)

locations). Figure 8 (a) shows observed cracks of the short beam of transverse direction (axis 5) in fifth story (a broken line in Fig. 7 shows this location). Shaded areas represent web openings and spalled concrete. Numbers represent crack width measured by crack scales in the field investigation. Numbers in parenthesis are crack widths estimated from photos because it was impossible to measure them in the field investigation. Figure 8 (b) shows a section of beam in Fig. 8 (a). This shows a section of the beam end.

In both longitudinal and transverse directions, most of the short beams have web openings (diameter of 10~200mm) as shown in Figs. 5 and 8, and cracks connected with these openings were remarkable. Some cracks and detached concretes were observed in areas of cold joint especially in upper stories.

In transverse direction, axis 3, 5 and 6 are frames that have coupled shear walls as shown in Figs. 4 and 7. On the other hand, the other frames have shear walls, which are not continuous walls as shown in Fig. 6. Therefore, there are no short beams in these frames where damages were minor, and minor flexural cracks were observed.

3.2. Structural walls

In both longitudinal and transverse directions, cracks in structural walls were distributed throughout the frames. Most cracks were less than 1mm wide. The damage of structural walls was minor in comparison with that of beams, because crack widths of structural walls were less than those of beams (Figs. 3, 4, 6 and 7).

3.3. Penthouse

Maximum crack widths of 1.5mm were observed in structural walls of the penthouse, and spalled concrete was not observed. The damage of the penthouse was minor in comparison with that of the main building. In the field investigation, cold joints were observed in upper structural walls.

4. ANALYSIS

4.1. Seismic evaluation results

Prior to the seismic retrofit in 1997, the seismic capacity of this building was evaluated according to the Japanese Standard (1990). In this evaluation, beams are assumed to be strong enough. Therefore, seismic capacity is computed from contributions of columns and walls.

Figure 9 shows story shear coefficients of each story. In the penthouse story, the story shear coefficients are reduced to 76% in longitudinal direction and 86% in transverse direction. These reduction factors are computed based on the Japanese Standard (1990) and used for considering the torsional response caused by the eccentricity of stiffness in both horizontal and vertical directions. Variations in strength through stories shown in Fig. 9 were relatively small in both directions. In transverse direction, the open frames without coupled shear walls were retrofitted with additional shear walls from first to third stories. Therefore, the story shear coefficients in fourth or upper stories are relatively lower (Fig. 9). Also in the field investigation, increase

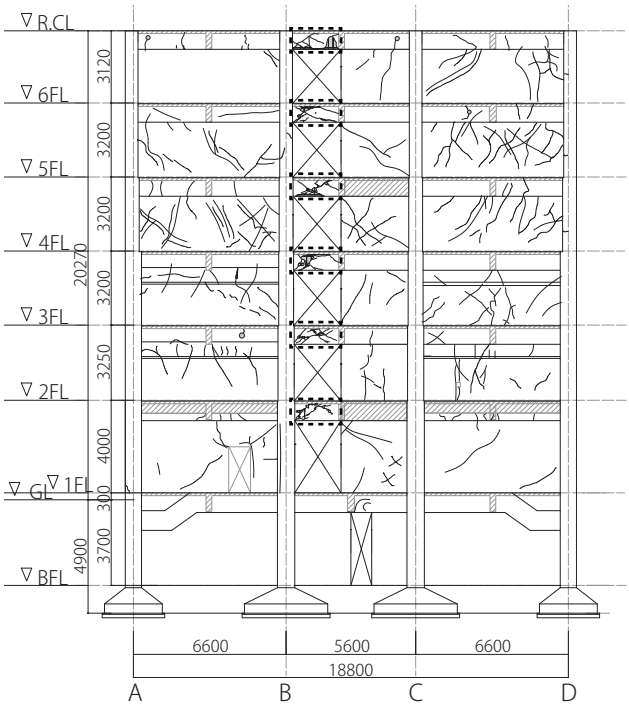


Figure 4 Observed cracks of transverse direction (axis 3)

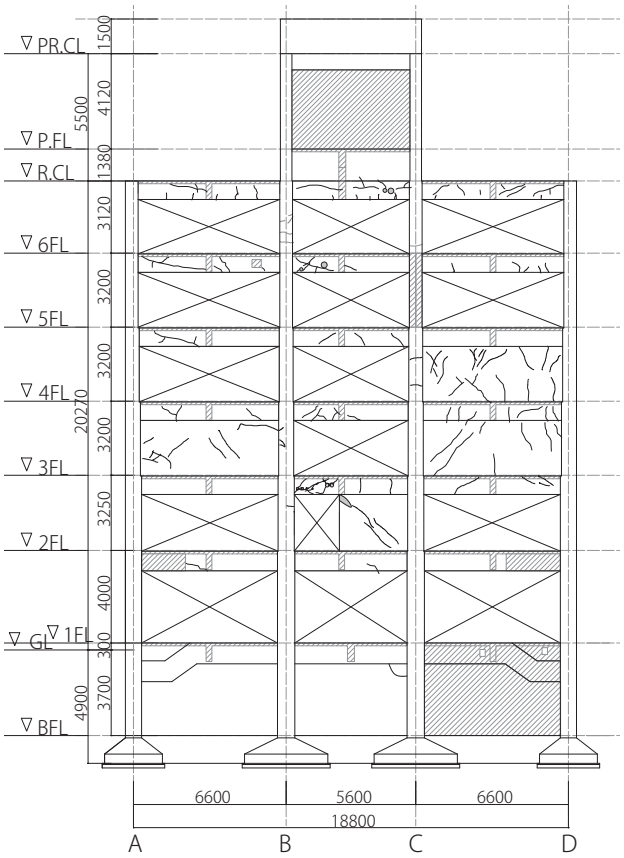


Figure 6 Observed cracks of transverse direction (axis 4)

of crack widths was observed on fourth to fifth stories. The story shear coefficients are more than 0.57 in both directions. Therefore, the building was assumed having enough strength based on the evaluation. Nevertheless

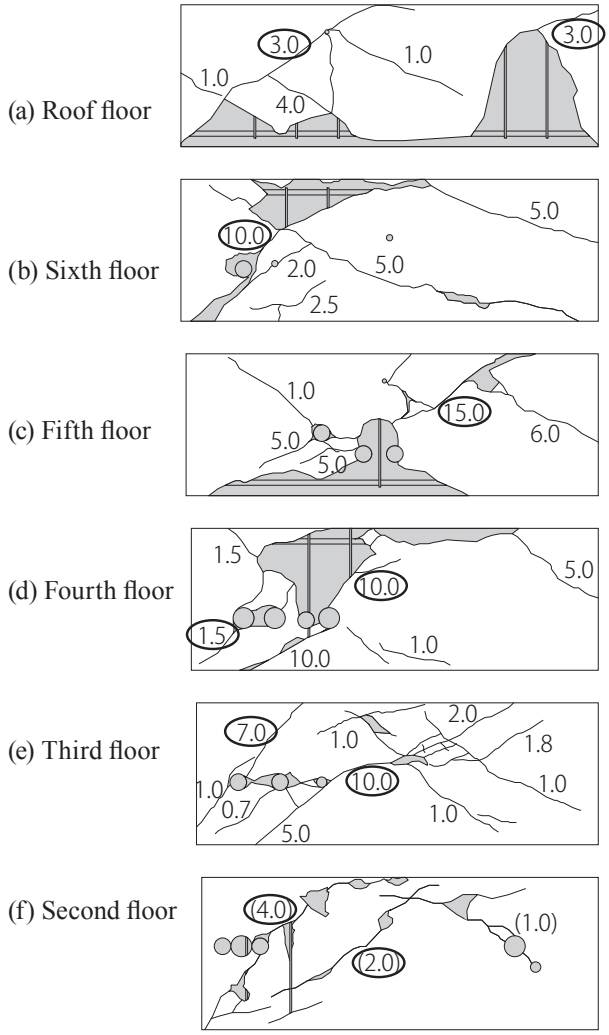


Figure 5 Observed cracks of the short beams (axis 3)

the building was severely damaged. The differences between the actual damage and the calculated values are assumed attributing to the difference of failure modes. In particular, beams were assumed to be rigid during the evaluation, but failure of the short beams take a lead over that of columns and walls actually. Therefore, a failure model considered the field investigation data is set in following section.

4.2. Calculation of the ultimate lateral strength

In this section, a failure model based on the field investigation data is assumed, and the ultimate lateral strength in the transverse direction is calculated by virtual work method.

4.2.1. Procedures

The characteristics of damage in transverse direction can be described as following: first, damage of coupled shear walls and columns were relatively small. Second, severe damage concentrated on the short beams. Axis 3 has both coupled shear walls and the short beams (Fig. 4), and it was damaged characteristically and retrofitted in first to third stories. Therefore, a failure model shown in Fig. 10 is assumed as this frame, and the calculation is conducted based on the following assumptions.

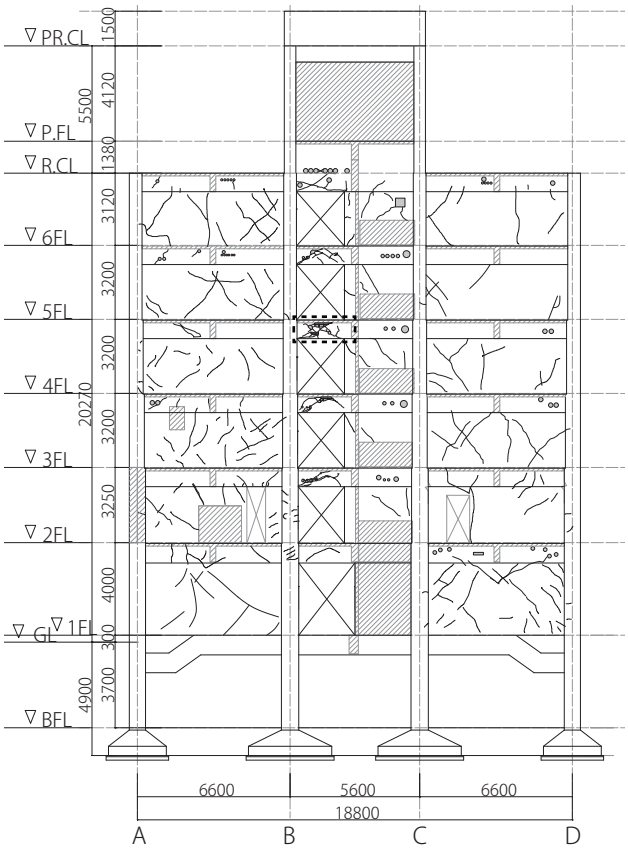


Figure 7 Observed cracks of transverse direction (axis 5)

Photo 4 Damaged short beam (4F - axis 5)

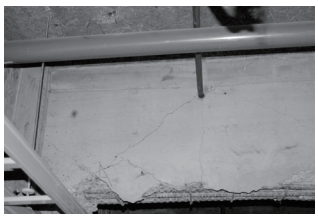


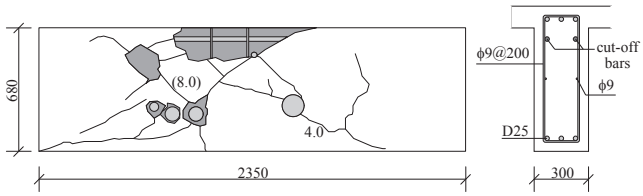
Photo 5 Damaged short beam (6F - axis 3)

The coupled shear walls rotate as shown in Fig. 10 and vertical reinforcement bars are assumed to be yielding at the bottom of the first story, where a_t represents cross-section area of longitudinal reinforcement in boundary column and σ_y represents yield strength of the steel. N_1 and N_2 are the permanent loading axial forces on the boundary columns of the walls. The circles at beam-ends in this figure represent the plastic hinge.

In the analysis, two models have been proposed. In the first one, the short beams of the span L_B fail in shear: the beams are completely destroyed and capacity in hinges is negligible. In the second one, the beams yield in flexure at both ends.

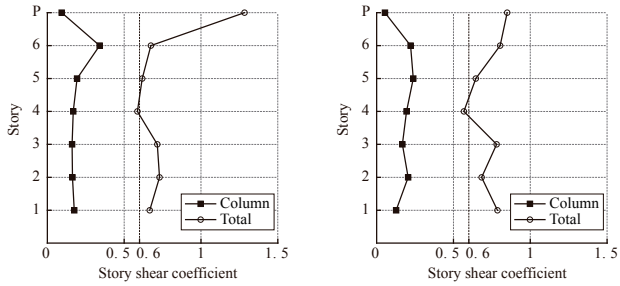
4.2.2. Results and discussions

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



(a) Observed cracks (b) Section

Figure 8 Observed cracks of the short beam of transverse direction in fifth story (axis 5)



(a) Longitudinal direction (b) Transverse direction

Figure 9 Story shear coefficients of each story

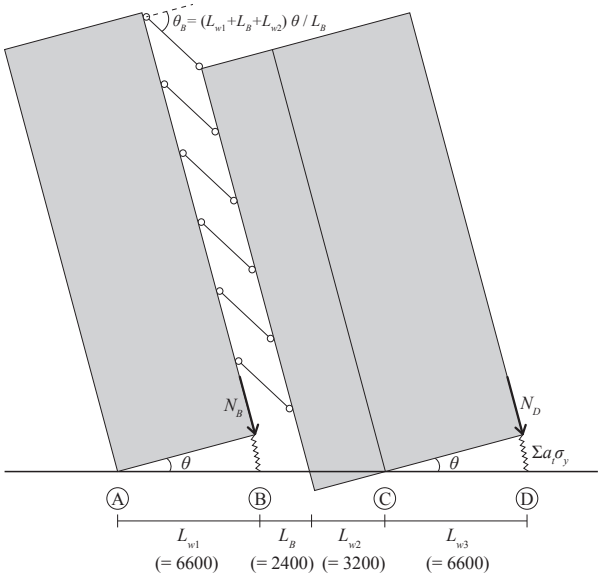


Figure 10 The assumed failure model

of the cracks found in the walls of the building. As to the results of the analysis, the lateral capacity of the frame axis 3 is 10MN, in the case of considering the contribution of the short beams. In the case of neglecting those beams, the lateral capacity is 8MN. Thus, the actual value would exist between these two results and the beams make an important contribution to the capacity of the frame actually. According to the documents of the seismic evaluation, the lateral capacity of the frame axis 3 is 10MN. It is a coincidence that this value is almost equal to the analytical result with considering beam contributions, however, this indicates that the building is assumed to have enough strength.

The rotation angles of short beams, θ_B in Fig. 10, is

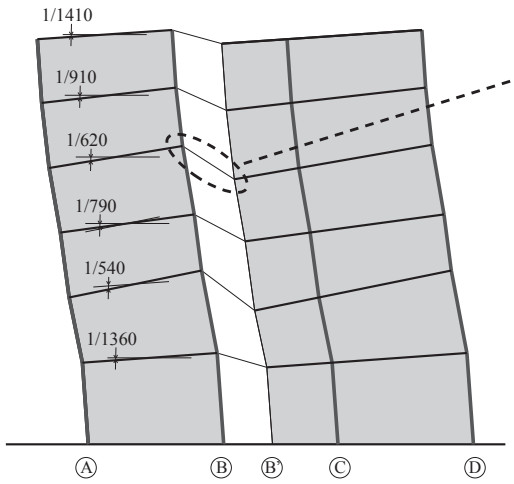


Figure 11 The assumed deformation model

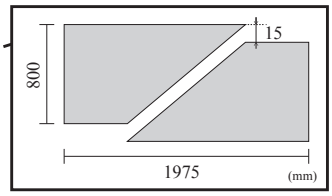


Figure 12 A pattern diagram of shear deformation

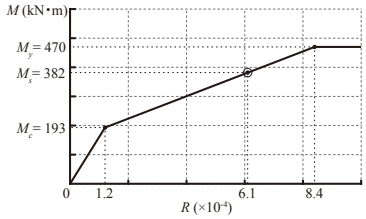


Figure 13 Moment - rotation relation

5.1 times larger than those of shear walls, θ in Fig. 10. It suggest that extremely large deformation will be induced to the short beams if the deformation of the building is relatively small.

4.3. Estimation of the actual deformation

Figure 11 shows assumed deformation model based on following assumptions.

The circled numbers in Fig. 5 are residual shear crack widths assumed to be induced by the deformation shown in Fig. 11. The rotation angle of the beam attributed to shear deformation can be estimated roughly from these crack widths and clear spans of the beams on each story (Fig. 12). In Fig. 11, a shear wall between axis A and B is rotate around axis A due to a column on axis A. This is based on the assumption that the location of the neutral axis is near the centroid of the column. A shear wall between axis B' and D is rotate around axis C due to a column on axis C.

For example, the rotation angle of the beam attributed to shear deformation in fifth story is 15/1975 ≈ 1/132 (Fig. 12). The rotation angle of the beam attributed to flexural deformation can be estimated from a moment - rotation relation based on the AIJ Standard (Architectural Institute of Japan 2010)(Fig. 13): in reaching the ultimate shear strength, the rotation angle is 6.1×10^{-4} (≈ 1/1640), and it is sufficiently small in comparison with that of attributed to shear deformation. Accordingly, the rotation angle of the shear walls in fifth story based on the model in Fig. 10 is θ = 1/620 (noted in Fig.11). In the 1990 Japanese Standard, a standard rotational deformation in ultimate is 1/250. In addition, the short beam has lower shear strength than flexural strength (Fig. 13 M_y: flexural moment, M_s: shear moment). Consequently, the coupled shear walls deformed in the range of elastic deformation, but the short beams extremely deformed, and the short beams was assumed to fail before the shear walls. It correspond to the observed crack patterns.

According to the ACI 318-11, the coupling beams with aspect ratio less than 2 shall be reinforced with diagonal bars. The coupling beams of this building failed in shear in spite of having aspect ratios larger than 2 because the shear strength of these coupling beams were not so large. The coupling beams should be reinforced to have larger shear strength than flexural strength and should not have openings.

5. LESSONS LEARNED AND CONCLUSIONS

(1) Both longitudinal and transverse directions, the damage of columns, structural walls and the penthouse was relatively minor.
(2) In longitudinal and transverse directions, severe damages concentrated on short beams. Most of these beams had web openings, and cracks connected with these openings were highly visible.
(3) From the seismic evaluation results, the story shear coefficients were more than 0.57 in both directions, and no story had an obvious weakness.
(4) The coupled shear walls deformed in the range of elastic deformation, but the short beams deformed extremely.
(5) The conventional reinforced coupling beams with aspect ratios larger than 2.0 suffered severe damage. In the previous researches and ACI 318-11 recommendation, coupling beams with aspect ratios lower than 2.0 were mainly target of special shear reinforcement.
(6) The beams shall be reinforced appropriately not only in terms of strength but also that of deformation in order to use a building continuously after earthquake damage with minor repairing.
(7) Web openings through the short beams are recommended avoiding.

Acknowledgments:

This work was financially supported by the Japan Science and Technology Agency and the U.S National Science Foundation. They are gratefully appreciated.

References:

D. Mitchell, R. H. DeVall, M. Saatcioglu, R. Simpson, R. Tinawi, and R. Tremblay (1995), "Damage to concrete structure due to the 1994 Northridge earthquake," Canadian Journal of Civil Engineering, Vol. 22, pp.361-377.

National Research Council (U.S.). Committee on the Alaska Earthquake (1973), "The Great Alaska Earthquake of 1964," National Academy of Sciences, Washington, D.C.

Paulay, T. (1969), “Coupling of Reinforced Concrete Shear Walls,” Proceedings, 4WCEE, Santiago, Chile, B-2, pp. 75-90.

Theodosios P. Tassios, Marina Moretti, andAntonios Bezas (1996), “On the Behavior and Ductility of Reinforced Concrete Coupling Beams of Shear Walls,” ACI Structural Journal, V. 93, No. 6.

American Concrete Institute (2011), “ Building Code Requirements for Structural Concrete,” ACI Committee 318, Farmington Hills, Michigan.

Building Research Institute (2001), “Seismic Evaluation and Retrofit,” The Japan Building Disaster Prevention Association.

Architectural Institute of Japan (2010), “AIJ Standard for Structural Calculation of Reinforced Concrete Structures,” Gihodo, Tokyo.