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AN UNEXPECTED BEAM SIDE SWAY MECHANISM OF A REINFORCED CONCRETE BUILDING IN 2016 KUMAMOTO EARTHQUAKE

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Abstract

A number of reinforced concrete buildings suffered damage in 2016 Kumamoto Earthquake, and the Ministry of Education, Culture, Sports, Science and Technology conducted the post-earthquake damage evaluation of the school buildings in Kumamoto prefecture after the earthquake. One of the junior high school buildings showed an unexpected beam side sway mechanism after the earthquake, although it had a typical plan of Japanese school buildings such that hanging walls and spandrel walls were attached to the girders. The building was a three-story reinforced concrete school constructed in 1973, which was designed without the design concept of forming a weak beam mechanism. It showed large residual cracks in floor slabs and girders with hanging walls after the earthquake. The maximum residual crack width was 2.0 mm on the floor slab and 1.0 mm on the girders with hanging walls mainly in the north frame of the 3rd story. The residual seismic capacity was calculated from the residual crack width of all its members in the post-earthquake damage evaluation, and the damage degree was “minor”.

Dynamic response analysis of a three-dimensional frame was carried out for evaluating the damage of the school building. The columns, girders and walls were idealized by the rigid spring model based on its structural drawing. The input ground motion was a strong earthquake motion record at K-NET Kumamoto station, which was 4.5 km far from the building. In the conventional analytical model, spandrel walls and hanging walls were regarded as the rigid zone for columns. The sections of these walls were a part of the girders, and it gave the large stiffness and strength of the girders compared to those values of columns. It caused the brittle shear failure of short columns during the response of the earthquake. In the modified analytical model, it assumed that the anchored length of the wall reinforcement was not enough, and the contribution of those reinforcements on the beam moment strength was ignored. The beam side sway mechanism was obtained in this analytical model, but the plastic hinges of the girders distributed mainly in the south frame. Finally, the full section of the floor slab was taken into account to evaluate the ultimate moment strength of the girders in the analysis. The plastic hinges of the girders were observed in the north frame, and a fair correlation was observed between the analytical result and the post-earthquake damage observation result.

Keywords: 2016 Kumamoto earthquake, post-earthquake damage observation, frame analysis, beam side sway mechanism, effective slab width

1. Introduction

In Japan, a number of reinforced concrete buildings have been designed to evaluate the lateral load carrying capacity with the beam side sway mechanism since the revised building code was established in 1981, but there have been few cases in which the beam side sway mechanism has clearly occurred in the buildings designed according to the old building code due to the earthquake damage so far. This is because that the beams in those old buildings has high ultimate moment strength due to the attachment of the standing wall or the hanging wall, and it causes the story collapse mechanism in the building. The seismic performance of the existing reinforced concrete buildings is evaluated based on the story shear capacity of the building assuming rigid beam in the conventional method. Kumamoto earthquake occurred on April 16 2016, and it caused significant damage on the three-story reinforced concrete school building in Kumamoto city, which was designed according to the old building code. Large residual cracks on floor slabs and beams and the beam side sway mechanism of the frame was observed [1]. In this study, based on the structural drawing, the static loading analysis and the earthquake response analysis of the building with three-dimensional nonlinear frame model. It simulates the earthquake damage of the building and the mechanism of the damage is investigated by those analysis.

2. School building

Figure.1 shows the south elevation of the school building. The school building was a three-story reinforced concrete school building constructed in 1973. It has twelve spans in the longitudinal direction, and there are multi-story structural walls at the end of the two spans. It has typical floor plan of the Japanese school building with two spans in the transverse direction, classrooms on south and a narrow corridor on north. The beams in the north frame have the attached standing walls and hanging walls, and beams in the south frame have the attached standing walls. It has the pile foundation. Figure.2 shows the cross section of the representative member of the building.

The beam side sway mechanism was observed in the school building, and the 2nd and 3rd story of the building are heavily damaged. The residual seismic capacity of the building was investigated based on the residual cracks on the beams and columns in the post-earthquake damage observation. The seismic index R of the capacity was 89% in the longitudinal direction of the 2nd story, and it indicates the degree of the building damage is minor. In determining the degree of damage, almost all of the beams are covered by the ceilings except for the beams in the north frame, so that the damage on the beams could not be identified. The cracking damage of the beams was regarded as the damage of the attached columns base on 2001 version of "Post Earthquake Damage Evaluation and Restoration Technology Guidance for Seismic Damaged Buildings "[2].

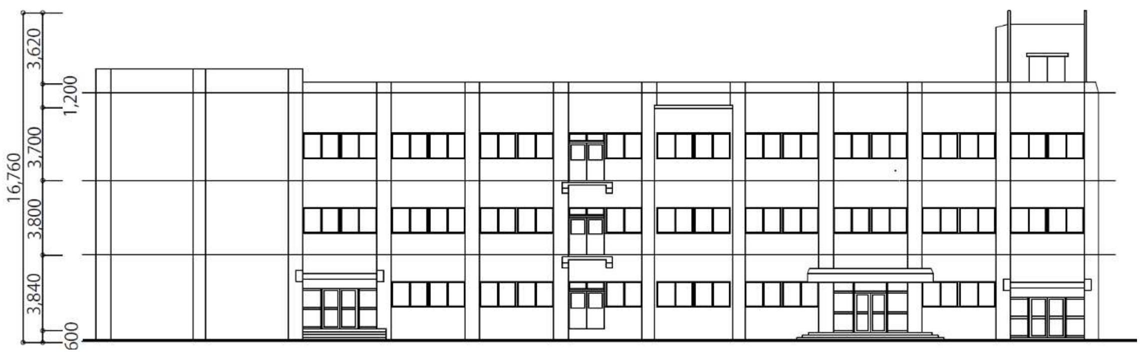

Figure.1 South elevation of the school building

column	beam
740×450 Main bar 12-D22 Hoop ϕ13@150	300×600 Main bar 5-D25 Stirup ϕ9@150

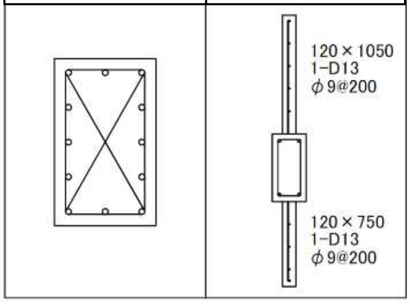

Figure.2 Cross section of the representative beams and columns

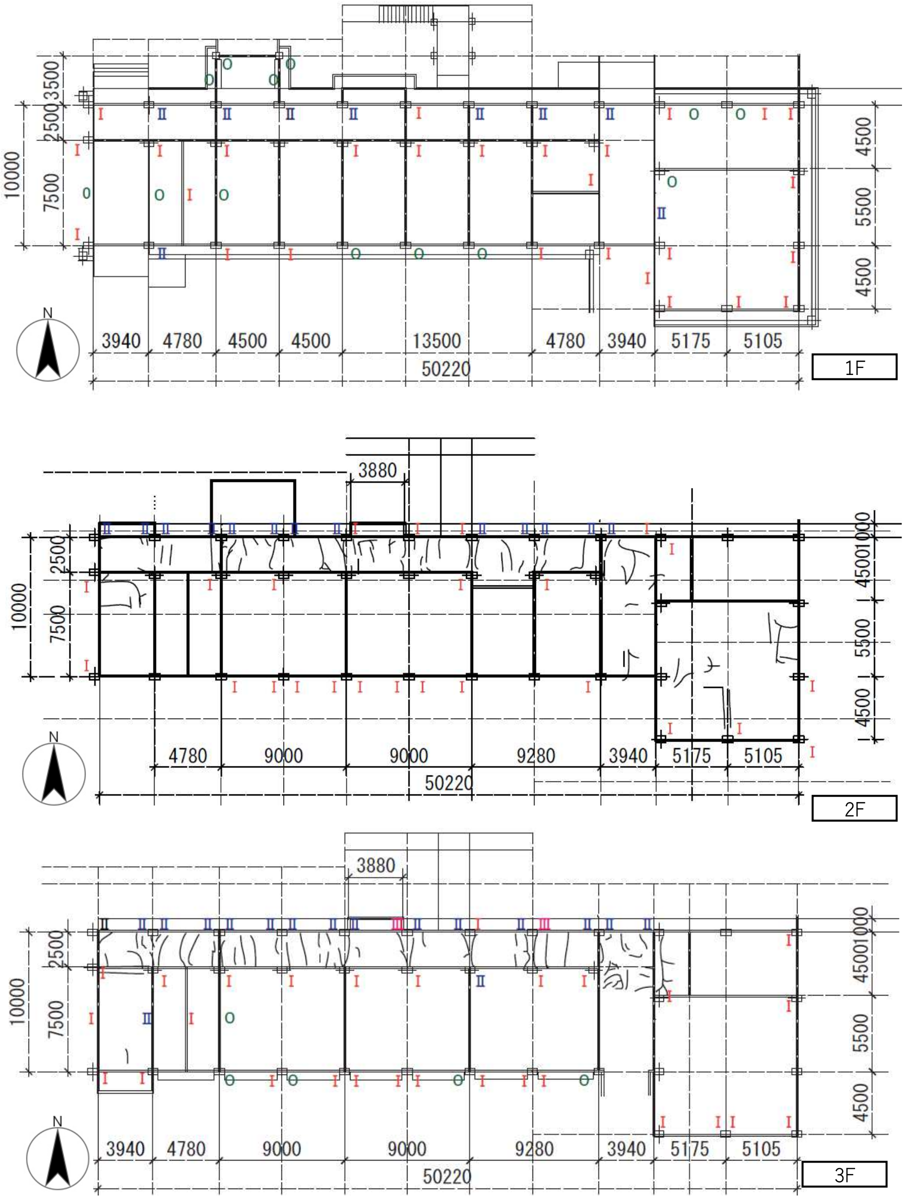

Figure.3 Post-earthquake damage evaluation for the beams and floor slabs

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(a) Cracks on floor slabs


(b) Cracks on beams with hanging walls

Photo.1 Residual cracks on the longitudinal beams and floor slabs

Figure 3 shows the result of the post-earthquake damage evaluation for the beams and floor slabs on the 1st floor to 3rd floors. Photo 1 shows residual cracks on the longitudinal beams and floor slabs in the north frame. The beams on the north side has 1050 mm height standing walls and 750 mm hanging walls in the cross section and large residual cracks was observed at the end of the standing walls and hanging walls. The maximum crack width is 1mm on those beams. A number of large residual cracks are observed in the 2nd and 3rd floor. The maximum crack width is 2 mm on those beams. The ceiling materials dropped off in the classroom and music preparation room at the end of the 3rd floor. There is one short column on the 1st story due to the existence of the huge height of the standing walls, and the shear cracks occurred in this short column locally.

2. Analysis model

This building shows obvious the beam side sway mechanism although it has high standing walls and hanging walls with beams constructed according to the old building code. This beam side sway mechanism is simulated by the analysis with the three-dimensional frame model. The static loading analysis and the earthquake response analysis in the longitudinal direction are carried out by Program CANNY [3]. The columns and beams are idealized by the rigid-spring models and the structural walls are idealized by Three-Vertical-Line Element model. The flexural cracking strength, yielding strength, and the stiffness degrading ratio of those members are calculated based on the AIJ Standard for Structural Calculation of Reinforced Concrete Structures [4]. The hysteresis of the flexural deformation is idealized by conventional Takeda model. The hysteretic parameter of the unloading stiffness in Takeda model is 0.4. The shear springs of members are elastic in the model, because it does not occur the shear failure in this building. The axial deformation is idealized by the axial-stiffness model considering the concrete section and the reinforcement in the cross section.

In this building, rippled reinforcing rebars are used for the beam-column main bars, but round bar is used for the transverse reinforcement of the beam and columns and the wall reinforcement, so that it can be assumed that the bond strength is not sufficient in those round rebars. Three analysis models have been proposed. In one model, the transverse reinforcement in the standing and hanging walls are anchored to the columns and another model ignores those reinforcements due to the shortage of its anchorage length. The other model only idealizes the attached concrete wall section to bear the compressive force and ignore the tensile yielding of the wall reinforcements. The plastic hinges of columns locate at the end of the standing and hanging walls because the standing wall and hanging walls are attached to the column surface.

The cooperated slab width affects the evaluation of the ultimate moment strength of T-shape beam. In recent study, it is proposed and verified that the full slab section contributes on the ultimate moment strength of T-shape beam by the static loading tests on the three-dimensional reinforced concrete assembled frame [5]. In conventional design model, 1m slab width is regarded as a part of the T-shape beam section, and the slab concrete section and reinforcement are included. The lower slab reinforcements are usually ignored because the bond strength is not sufficient for the anchorage of the tensile reinforcement. In this study, two analysis models are proposed. The conventional cooperated slab width contributes on the stiffness and strength of T-shape beam in one model, while the full slab section contributes on the stiffness and strength of T-shape beam in the other model. The assumption of the four analysis models are listed in Table.1. The moment strength ratio of the columns to the beams in the north frame and south frame is also shown in Table.1. The ratio is lower than 1 in the conventional model A, and it indicates the story collapse mechanism occurs in the frame due to the increment of the ultimate moment strength by the attachment of the hanging walls and standing walls. The ratio is also lower than 1 in the south frame of model D. This model ignores the contribution of the wall transverse reinforcement, but the neutral axis position shifted to the compressive concrete wall section, and the ultimate moment strength of beams exceeded that of columns. The ratio in model B and C is higher than 1 and it indicates those models can simulate the beam side sway mechanism.

Table.1 Parameter for each analytical model

	Anchorage of wall reinforcement	Attached wall section	Slab corporative width	North frame (X5Y6)	South frame (X5Y2)
Model A	Sufficient	Included	1m	0.55	0.88
Model B	Not enough	Ignored	Full span	2.18	2.42
Model C	Not enough	Ignored	Full span	2.18	2.10
Model D	Not enough	Included	Full span	0.67	0.98

The load distribution pattern for the static loading analysis is the inverted triangular shape. Input acceleration in the earthquake response analysis is the seismic ground motion of the main shock in 2016 Kumamoto Earthquake at the observation site of K-NET Kumamoto, which is 4.6 km far from the building [6]. The viscous damping factor is proportional to the tangent stiffness and 5%.

3. Analysis results

Figure.4 shows the load-displacement relationship in the static loading analysis of each model. The base shear coefficient at 0.67% 1st story drift is 0.85 for model A and D, and it is 0.74 for model B and C. The assumed base shear coefficient CT is 0.80 according to the guideline for the seismic performance evaluation of the existing reinforced concrete buildings [7], and the base shear coefficient in model A and D exceeds this value. The drift ratio of yielding point in model B and C is larger than that in model A and D. The maximum response of each story is also plotted in Figure.4. The maximum story drifts exceeded 0.8% in model B and C. It is consistent with the large residual cracks on beams and floor slabs after the earthquake. The maximum story drifts are smaller than 0.3% in model A and D. The large shear force is acted on the short columns and it indicates that the shear failure of the columns precedes in 1st story before the yielding of the columns as represented in the analytical model. The plastic hinges location in model C and D is shown in the Figure.5. The beams on 2nd floor and 3rd floor yielded as well as a large residual crack on the floor slabs in the post-earthquake damage observation. The beams on the top floor do not yield in the analysis, while a large residual crack was observed at the end of the hanging walls on the top beams in the post-earthquake damage observation.

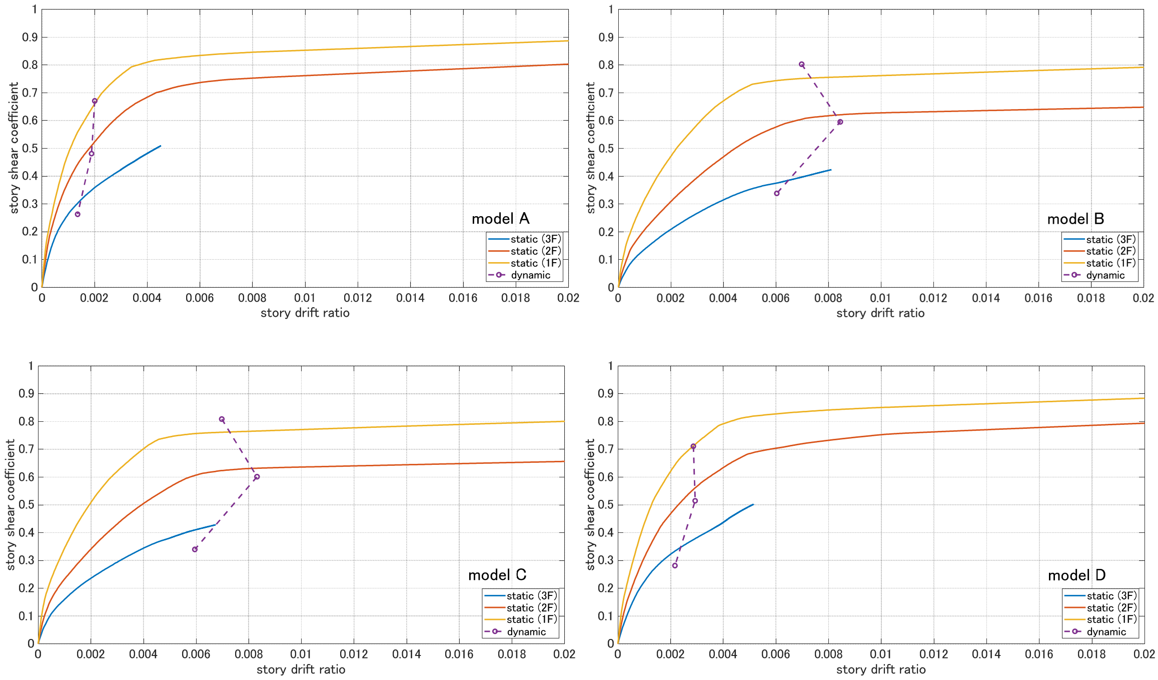
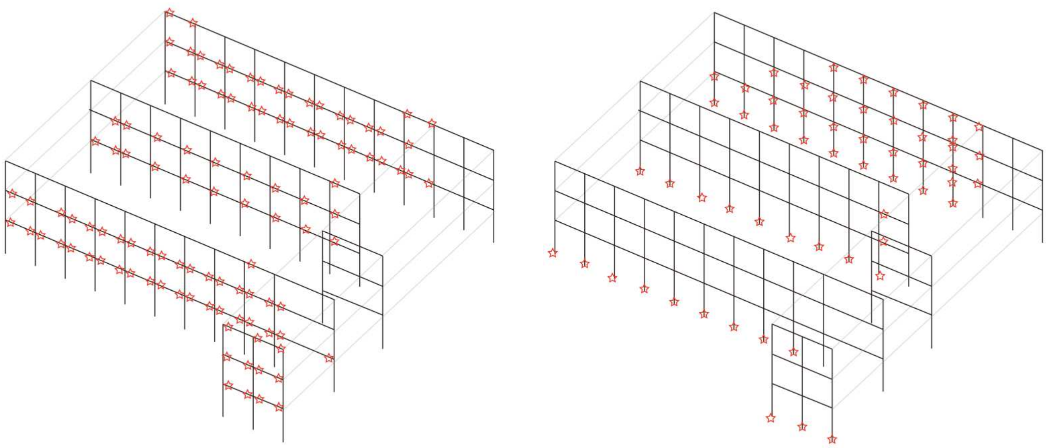


Figure.4 Load-displacement relationship in the static loading analysis of each model



(a) Model C (b) Model D

Figure.5 Plastic hinges location in the analytical models

Table.2 ductility factor of beams in model B and D (3rd floor beam)

Model	Frame	X1-X2	X2-X3	X3-X4	X4-X5	X5-X6	X6-X7	X7-X8	X8-X9	Average
B	Y2 South	1.75	1.66	1.86	1.89	1.89	1.89	1.90	1.85	1.83
	Y5 North	1.64	1.40	1.63	1.66	1.67	1.68	1.70	1.70	1.63
C	Y2 South	1.06	1.38	1.53	1.57	1.56	1.56	1.57	1.55	1.47
	Y5 North	1.12	1.38	1.61	1.64	1.65	1.66	1.67	1.64	1.55

The ductility factor of beams is compared in model B and C in Table.2. The maximum story drift is similar in those two models but the higher ductility factor of beams is obtained in the north frame of model C, because the transverse span length is wider in the classroom rather than in the narrow corridor and the contribution of the full slab section on the ultimate moment strength of the beams is quite large in the south frame. The actual damage on beams in 2nd and 3rd floor concentrated on the south frame, and it indicates the wider effective slab width affected the location of the plastic hinges and mechanism of the buildings.

4. Conclusion

In this study, the static loading analysis and earthquake response analysis are carried out to simulate the beam side sway mechanism in the damaged school building designed by old Japanese building code. It proposes the several assumptions for the analytical models which is different from the conventional design model. Following conclusion are obtained from this study.

・Static and dynamic analysis was carried out using four models. Three models ignore the transverse reinforcement of attached standing and hanging walls anchored to the beams because bond strength is low for a round rebar. Two models ignore the attached concrete wall section as well as the wall reinforcement. The cooperative slab width of T-shape beam is assumed to be either conventional 1 m or full span length.

・In the model considering the wall cross section or the wall reinforcement of the beam with the standing or hanging walls, it occurs the story collapse mechanism which is different from the observed damage. On the other hand, in the model ignoring the wall cross section and reinforcement, the beam side sway mechanism occurs, which is consistent with the observed damage. The maximum story drift ratio is about 0.8% in the earthquake response analysis, which is also consistent with the large residual cracks on the 2nd and 3rd floor slab.

・In the model in which the floor slab cooperation width is 1 m, the flexural ductility of the beam on the south side is relatively large, and this result is different from the observed damage. On the other hand, in the model in which the floor slab was full width effective, only the beam strength on the south side increases, and the damage of the beam on the north side is relatively prominent. The analytical result tended to be approximated to the observed damage.

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