17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

ANALYTICAL STUDY OF RC BUILDING WITH SOFT FIRST STORY DESIGNED AFTER 1981 AND DAMAGED IN THE 2016 KUMAMOTO EARTHQUAKE

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Abstract

The current Building Standards Act includes provisions to prevent the collapse of buildings in the event of a major earthquake, to ensure people's safety. However, some buildings with soft first story which were designed in accordance with the new seismic standard were heavily damaged in the 2016 Kumamoto earthquake and could no longer be used. Because of this background, the post-earthquake functional use of buildings should be secured in addition to seismic safety performance which the current design code is required. To develop this design method, it is necessary to propose appropriate analytical model which can express post-earthquake damage state of the building, so as to properly understand the damage of members.

Accordingly, this study examines analytical models which can express damage state of members for the RC building with soft first story. Specifically, we examine the effect of different analytical model for members which influence dynamic response of the building, and compare it with actual damage to confirm the validity of the modeling. In addition to that, we examine the design method to secure the post-earthquake functional use of the building.

A target structure is a RC building with soft first story which was constructed based on the domestic seismic standard after 1981 and was damaged in the 2016 Kumamoto earthquake. The structural calculation document for the building shows that many structural slits were installed in the wall of the bay in the long direction, but we could not clearly confirm this in the field survey. Therefore, to examine the effect of different model of walls, we prepare open frame model and other models assuming some structural slits with different specifications, and analyze these models. By comparing the damage of the member predicted from the analysis result with the actual damage, installation method of the structural slits is expected and validity of the investigated models are demonstrated. Furthermore, a design method for reducing this kind of damage will be discussed.

Keywords: Soft First Story, Post-Earthquake Functional Use, Analytical Models, Structural Slits, Kumamoto Earthquake

1. Introduction

1.1 Study objectives

The current *Building Standards Act* includes provisions to prevent the collapse of buildings and to protect lives in the event of a major earthquake. However, after the 2016 Kumamoto earthquake, many people encountered hardships in their daily lives because buildings lost some of their functionalities. Therefore, guaranteeing the functional use of buildings in addition to seismic safety is important after earthquakes. Design principles to secure this functional use of buildings are necessary to achieve this objective. An initial step is to build a model that can reproduce the damage of actual buildings after an earthquake, and thus determine the extent of damage to members.

The present study built a model to reproduce the damage to members of a four-story RC (reinforced concrete) building with a soft first story that was damaged in the 2016 Kumamoto earthquake. The validity of the model was verified by looking at how the differences in the modeling of members affected the response and comparing the responses of the model and the actual damage. In addition, a fundamental investigation of issues important for proposing design methods of buildings that allow post-earthquake functional use was conducted.

1.2 Study methods

A four-story reinforced concrete collective housing building with a soft first story was modeled in the present study. This building was built in 1998 and was subject to the domestic seismic standard introduced after 1981; however, the first story collapsed in the 2016 Kumamoto earthquake. This building contained some structural slits; however, the information for these could not be obtained from the structural calculation sheet of the initial construction. Therefore, to investigate the effects of different wall models on the building response, an open frame model, a model with slits on two sides, and a model with slits on three sides were prepared. The building consisted of five flames connected only by slabs except for the first story that was interconnected by foundation beams. Multi flame models with five flames connected by pinned beams or slabs were built to investigate the effect of differences in modeling slabs without beams on the building response.

Static increment and dynamic response analyses were performed on the derived models. A seismic wave observed in a nearby area was used in the dynamic response analysis. The design condition for the structural slits was estimated and the validity of the built models demonstrated by investigating the effects of modeling the differences of the walls and slabs on the building response and then comparing the damage to members based on analysis of the derived models to the actual damage.

The cross-section of pillars in the soft first story was also calculated based on the currently recommended soft first-story design methods and its effects on the building response was determined.

2. Overview of the Studied Building

2.1 Location and structure

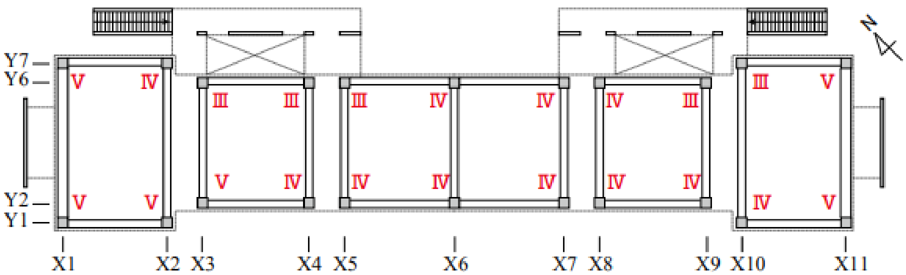
The present study undertook an analysis of a four-story reinforced concrete collective housing building in Uto City, Kumamoto Prefecture. This building was built in 1998 and was subject to the domestic seismic standard introduced after 1981. The plane shape was rectangular, and the long direction was along the northwest-southeast axis. There were 10 spans in the long direction and one span in the short direction. It was symmetric in the long direction. Four buildings had one span each in the short and long directions, and one building had two spans and one span in the short and long directions, respectively. The five buildings were connected by foundation beams on the first story and slabs on the second and higher stories. The structure was a pure open frame in the long direction and an open frame with multi-story shear walls in the short direction. The foundation was a pile foundation. There was a 220 mm-thick multi-story shear wall along both edges in the long direction of the first story and multiple 220 mm-thick wall pillars were located immediately below the stair room (northeast side of the Y4 frame). The first story was a pure soft first-story structure.

2.2 Structure and materials

Figs. 1 and 2 show the first story plane shape and framing drawings, respectively. The concrete strength was 210 kgf/cm^2 and the steel type was SD295 for D16 and below and SD345 for D19 and above (see Table 1). Table 2 shows the information of the major pillars, beams, and walls.

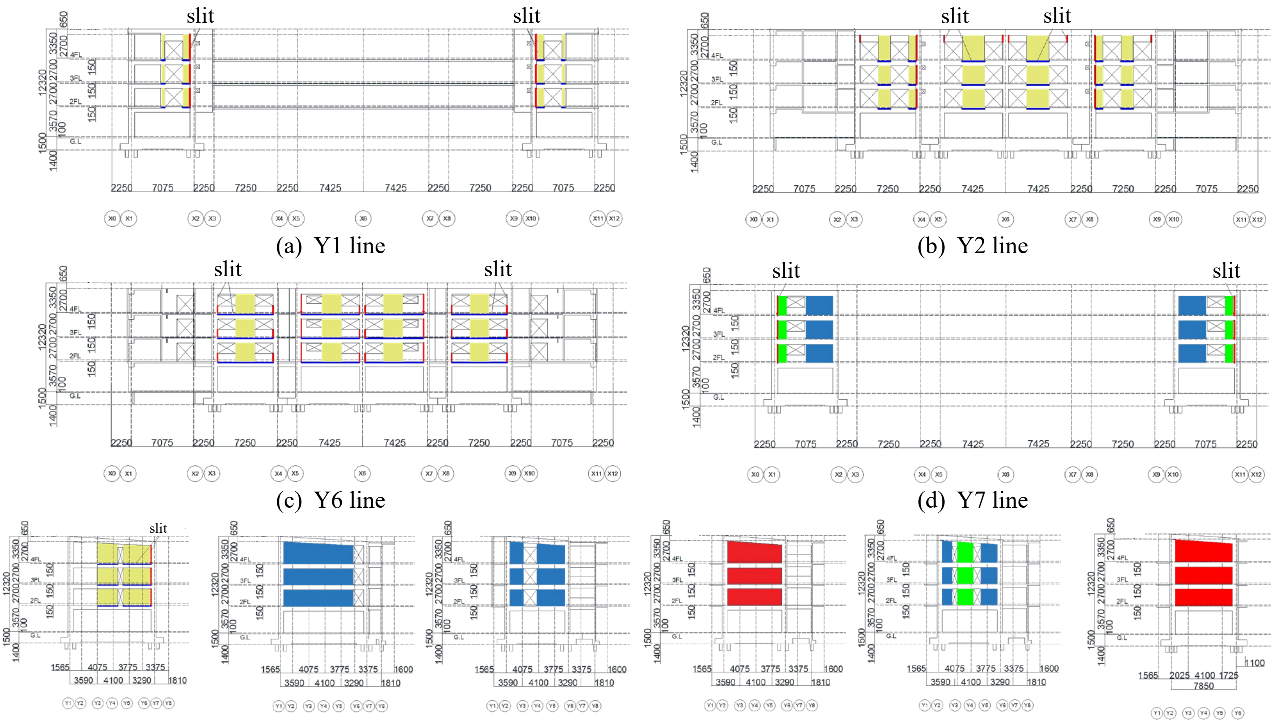
Table 1 – Materials

Concrete	Type	Strength (Kg/cm^2)	Member
	Ordinary concrete	210	All
Reinforcement	Type		
	Deformed bar	SD295	D16 or less
		SD345	D19 or more
	Compression fitting (The Diameter : D19 or more)		



Red character : the member damage evaluation results

Fig. 1 – The first story plane shape including damage status

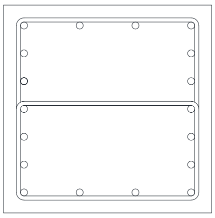
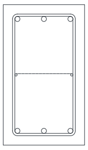


(a) Y1 line (b) Y2 line (c) Y6 line (d) Y7 line (e) X1, X11 lines (f) X2, X10 lines (g) X3, X9 lines (h) X4, X8 lines (i) X5, X7 lines (j) X6 line

Fig. 2 – Framing drawings including categorization of walls (unit: mm)

Table 2 – The details of the cross-section (pillars, beams, walls)

(a) pillars, beams (unit: mm)

Location	First story pillars (X3~X9 lines)	Second story beams
Cross-section		
Dimension	650×650	350×600
Main reinforcement	18-D22	3-D22+3-D22
Hoops shear reinforcement	⊟-D13-@100	□-D10-@200

(b) walls (unit: mm)

Thickness	150	180
Reinforcement	D10-@200S	D10-@200D
Width fixing reinforcement		D10-@1000

2.3 Overview of damage

The survey results of the damaged subject building, according to the Quick Report of the Field Survey on the Building Damage by the 2016 Kumamoto Earthquake [1] by the Building Research Institute, are given below. The damage situation of the first story is shown using the definitions in Fig. 1.

Many main rebars buckled on the first story pillars and some fractured, with most damage at the pillar tops (Photos 1 and 2); however, there was similar damage in several of the pillar bottoms. There was cracking in the walls along the periphery; however, the damage was less severe than in the pillars. Thus, the walls along the periphery were not considered in the present study because these would not have acted as earthquake resisting elements in conjunction with the pillars.

(a) Pillar viewed from the east

(b) Enlarged view of capital

Picture 1 – X11-Y1 pillar (Damage degree V)

(a) Pillar viewed from the east

(b) Enlarged view of capital

Picture 2 – X5-Y2 pillar (Damage degree IV)

3. Analysis Overview and Results

3.1 Damage analysis using a frame analysis model

The SNAPver.7 was used as analytical software. The present study employed static increment and dynamic response analyses. The analysis conditions are outlined below.

3.1.1 Overview of the modeling of the building

Structural slits were used to improve the seismic resistance in the building; however, we could not determine whether the slits were along two or three sides because the design documents from the initial construction were not clear. Therefore, an open frame model and models assuming slits along two or three sides were included in the present study. The building consisted of five flames connected by slabs only except for the first story that was connected by foundation beams. The effect of the slab on the building response was not clear, thus multi flame models were employed where the five flames were connected by pinned beams or slabs. The present study did not model slabs, minor beams, or stairs. Beams and slabs were considered as rigid floors to prevent deformation in the axial direction.

3.1.2 Modeling of the walls

1) The long direction was modeled as a pure open frame and the short direction was an open frame with partially sheared walls (X2, X4, X6, X8, and X10 frames were modeled as shear walls and the rest were modeled as open frames).

2) Frames in the long direction were modeled as walls with slits on two sides (structural slits along pillars).

3) Frames in the long direction were modeled as walls with slits on three sides (structural slits along pillars and the beam).

3.1.3 Modeling of the entire building

4) The five flames were connected with pinned beams.

5) The five flames were connected with flat slabs.

The four models were derived by combining the modeling approaches given above (Table 3).

Table 3 – Model list

Model name	Combination of modeling approaches	Modeling of slabs without beams
Model 1	1) × 4)	Pinned beams
Model 2	2) × 4)	Pinned beams
Model 3	3) × 4)	Pinned beams
Model 4	2) × 5)	Flat slabs

The following investigations were considered using these models:

(1) Models 1, 2, and 3: Effect of the wall model on the building response.

(2) Models 2 and 4: Effect of the modeling of slabs without beams on the building response.

■ Definition of nodes

Pillars, beams, and mullion walls were modeled as wires located at the centerline (members with walls were also modeled at the pillar and beam center position). Nodes were set at junctions between the members and between a member and the ground. There were no degrees of freedom regarding the displacement and rotation of nodes at the pillar and wall bottoms for the first story and there were no other constraints on the degrees of freedom.

■ Designation of rigid zones and dangerous cross-section positions

Wall face position D/4 (does not enter from the pillar, beam, or wall face toward the junction side). The rigid zone edge was located at D/4 from the face position toward the junction side (D was the member height including the wall).

The categorization of walls is shown in Fig. 2.

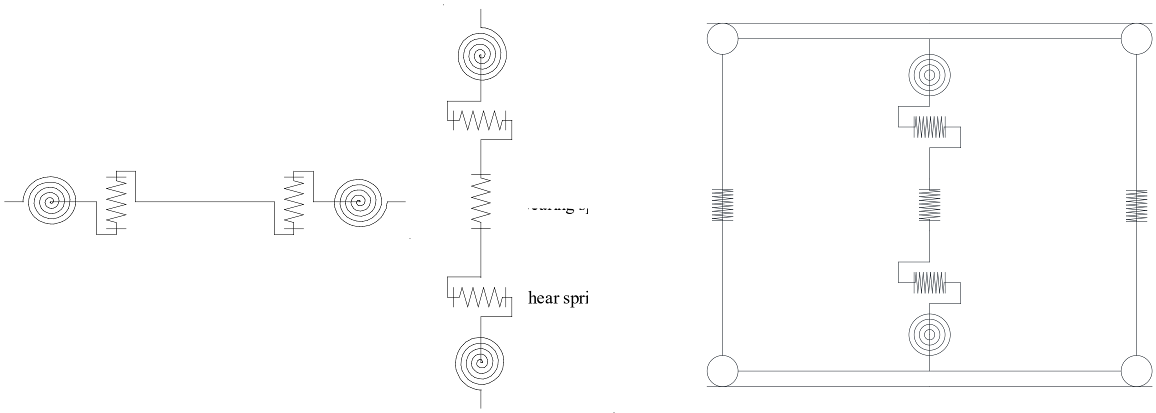
The yellow wall was modeled as a mullion and hanging wall when the model had slits in two and three sides, respectively. The green, blue, and red walls indicate mullion, wing, and shear walls, respectively.

3.1.4 Modeling of members

1) Spring model settings

Fig. 3 shows the uniaxial spring model of various members.

Pillars, beams, mullion walls, shear walls, and flat slabs were modeled as elastoplastic models that considered cracking and yield. Pillars and mullion walls had torsion, shear, and bearing springs at the dangerous cross-section position (the bearing spring was at the center of the flexible length of the member), whereas beams and flat slabs had torsion and shear springs at the dangerous cross-section position. When modeling a shear wall with three pillars, it consisted of a beam where the length of the shear wall was a rigid zone, two vertical members at both sides of the shear wall that consisted of a bearing spring that was connected by pins at both ends, and a vertical member at the center of the shear wall that had torsion springs on the top and bottom ends, shear springs, and a bearing spring. Torsion and shear springs were treated using a trilinear model that considered the cracking and yield (Fig. 4) and bearing springs were handled by a bilinear model that was elastic on the compression side and yielded on the tensile side (Fig. 5).



(a) Beams, Flat slabs (b) Pillars, Mullion walls (c) Shear walls

Fig. 3 – Conceptual diagram of uniaxial spring models

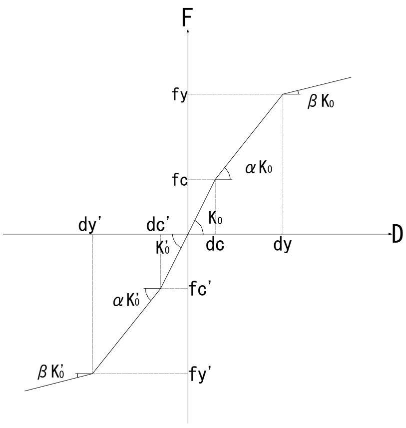


Fig. 4 – Skeleton curve of torsion and shear springs

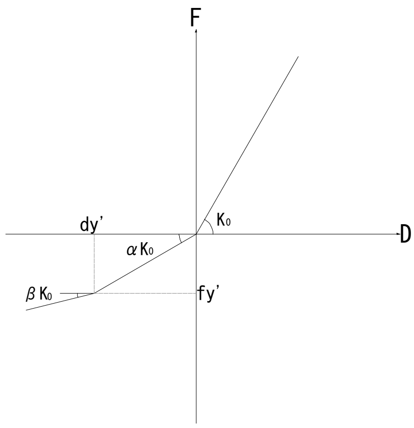


Fig. 5 – Skeleton curve of bearing springs

2) Calculation of the stiffness and proof stress

The stiffness and proof stress of the members were calculated based on the following assumptions and formulas from [2] and AIJ Standard for Structural Calculation for Reinforced Concrete Structures [3].

(a) The inflection point-height ratio assumed antisymmetric bending, except for the cantilever members.

(b) The axial force when calculating the bending proof stress of pillars was the axial force in the building weight summary and an axial force ratio of 0.2 was used when calculating the axial force of the mullion walls.

(c) The M-N interactions of the variable axis forces were not considered, and the bending proof stress was fixed to that of the initial axial force.

(d) The stiffness reduction rate after yield was 0.001 times the initial stiffness.

(e) The material strength was taken from the actual building structural design summary.

(f) The stiffness increase rate for the hanging walls with slits on three sides was 1.5.

3.1.5 Analysis conditions

(a) Static increment analysis conditions

A static analysis with increment load was conducted. The density distribution was an external force distribution based on the Ai distribution. Analysis was performed separately for the long and short directions.

(b) Dynamic analysis conditions

The seismic waves from the foreshock and main shock of the 2016 Kumamoto earthquake were used, with the data obtained from K-NET Uto [4]. The seismic waves were input from two directions, the north-south and east-west, in the present study. An instantaneous stiffness proportional damping with a damping constant of 5% was used.

3.2 Results of the analysis of each model

Figs. 5 and 6 show the story shear force deformation angle relationships in the long and short directions, respectively, which were obtained from the static increment analysis of each model. Figs. 7 and 8 provide plots of the maximum inter-story deformation angle in the long and short directions, respectively, for each model obtained from the dynamic response analysis using seismic waves from the foreshock and main shock of the 2016 Kumamoto earthquake. The present study focused mainly on the soft first story because the target building had a soft first story and damage was concentrated on this story.

3.2.1 Effect of wall modeling differences on the response

The story shear force obtained from the static increment analysis on a multi flame model is discussed. Model 1, where the long direction was a pure open frame and the short direction was an open frame with partially sheared walls, and model 3, where the frames in the long direction were walls with slits on three sides, are compared first. The first story shear force in the long direction was 10000 kN and 11000 kN in models 1 and 3, respectively (Fig. 5). The first story shear force of model 3 was 1.1 times larger than that of model 1 and no deformation concentration was found on the first story. Story shear stresses for the other stories of model 3 were approximately 1.1 times larger than those of model 1. The story shear force of model 2, where frames in the long direction were walls with slits on two sides, was 1.15 times larger than that of model 1 for the first story in the long direction (Fig. 5). Here, the deformation concentration on the first story was observed as per the actual damage. Using walls with slits on two sides increased the horizontal stiffness of the second story and higher stories that had walls, resulting in deformation concentration on the first story.

There was no great difference in the short direction first story shear force of models 1, 2, and 3 for the entire building (Fig. 6). However, the end (X1) face damage was intense in its actual damage. The shear and axial forces on the first story end (X1) face were compared. The shear force for model 1 was smaller than that for models 2 and 3 by approximately 20 kN and the values for models 2 and 3 were the same (Table. 5(a)). Therefore, there was no great difference in the three models. In contrast, the axial force was the smallest in model 1, which did not consider walls on the end frames, and was the largest in model 2 that strongly reflected the effect of walls on the end frames (Table. 5(b)). When tensile and compressive forces act alternately on a main rebar, the bond fracture from the tensile forces and the bulging from subsequent compressive forces are expected. Moreover, bulging of the main rebar can cause cracking and peeling of the concrete and ultimately buckling of the rebar. Model 2 had the highest axial force and was therefore subject to the most fracture. Buckling of the main rebars and decreased axial support performance were observed in the damage of the actual building; thus, the target building was estimated to have slits on two sides.

The dynamic response analysis results showed that between model 1, where the long direction was a pure open frame and the short direction was an open frame with partially sheared walls, and model 3, where the frames in the long direction were walls with slits on three sides, the deformation along the long direction was smaller in all stories for model 3 for the foreshock seismic waveform (Fig. 7). However, the deformation for the first story was larger in model 3 than that in model 1 for the main shock seismic waveform. Considering that the walls increased their stiffness in the second and higher stories, the deformation of the first story became larger. The effect of the walls was more profound in model 2, where frames in the long direction were walls with slits on two sides. Here, the stiffness in the second and higher stories was very large and the deformation of the first story was extremely large.

There was no great difference between models 1, 2, and 3 in the inter-story deformation angle in the short direction, even though the first story deformation angle in model 1 was slightly larger (Fig 8). The only difference between models 1, 2, and 3 in the short direction was that of the slit conditions of the end (X1) frame. Model 1 did not consider walls in the end (X1) frames; thus, the increase in deformation was attributed to the increased eccentricity in the X1-2 wing.

3.2.2 Effect of slab modeling differences on the response

Comparing models 2 and 5 where the five flames were connected by pinned beams and slabs, respectively, there was no great difference in the story shear force in either the long or short directions (Figs. 5 and 6) based on the static increment analysis. The inter-story deformation angle from the dynamic response analysis did not show great differences in either the long or short directions (Figs. 7 and 8); thus, they were almost the same. Therefore, the effect of slabs on the building response was very small and the differences in the multi flame modeling barely affected the response.

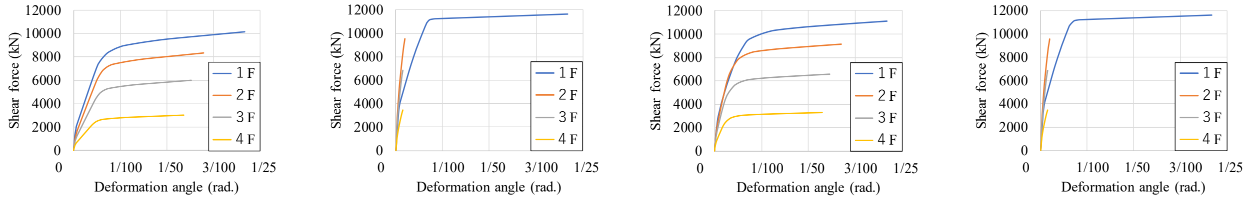
3.2.3 Comparison of actual and multi flame model damage

The actual damage and the damage from model 2 (frames in the long direction were walls with slits on two sides), which was close to the actual building, were compared to verify the validity of the model.

Comparison with the member damage evaluation results [1] from the actual damage inspection was conducted by categorizing the plasticity rate of members obtained from the dynamic response analysis using appropriate boundary values. The appropriate values were based on the damage category evaluation standards [5]. Although the damage category was not defined by the plasticity rate μ for the bending members, the damage category evaluation standard was III, IV, and V for the plasticity rates of approximately 1, 1 to approximately 80% of the maximum proof stress, and more than 80% of the maximum proof stress, respectively. The boundary plasticity rate was determined to approximately match these boundaries. Three categories were used in the present study, which are $\mu < 4.5$, $4.5 < \mu < 8.5$, and $8.5 < \mu$ for categories 1, 2, and 3, respectively.

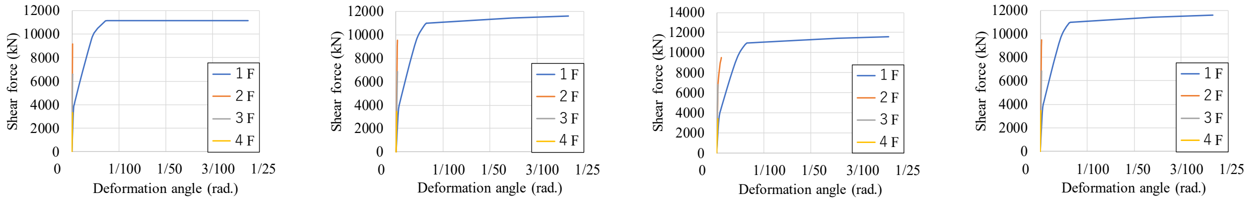
The adopted plasticity rate of each pillar was the smaller of the pillar top and bottom values for each input seismic wave. Fig. 9 shows a list of plasticity rates for the first story pillars of model 2. Here, the three categories above are denoted in yellow, orange, and red. The member damage evaluation results [1] are shown in Fig. 10, where damage categories III, IV, and V are colored in yellow, orange, and red, respectively. The fracture results from model 2 and the damage evaluation results roughly matched. Therefore, the structural slits should have been along the two sides, if there were any, and the model was verified to be reasonable.

According to Figs. 9 and 10, the damage in the long direction was relatively accurate. In contrast, in the short direction, the actual damage was more severe in the Y1-2 passage than that in the Y6-7 passage; however, the damage was basically symmetrical from the analysis. One possible reason is that the stairs located at the northeast of the Y6-7 passage, which were not included in the analysis, had the ability to bear structural proof stress. Therefore, the model was reasonable in general; however, stairs must be modeled to reproduce damage more accurately.



(a) Model 1 (b) Model 2 (c) Model 3 (d) Model 4

Fig. 5 – Comparisons of Q-R curves (The long direction)



(a) Model 1 (b) Model 2 (c) Model 3 (d) Model 4

Fig. 6 – Comparisons of Q-R curves (The short direction)

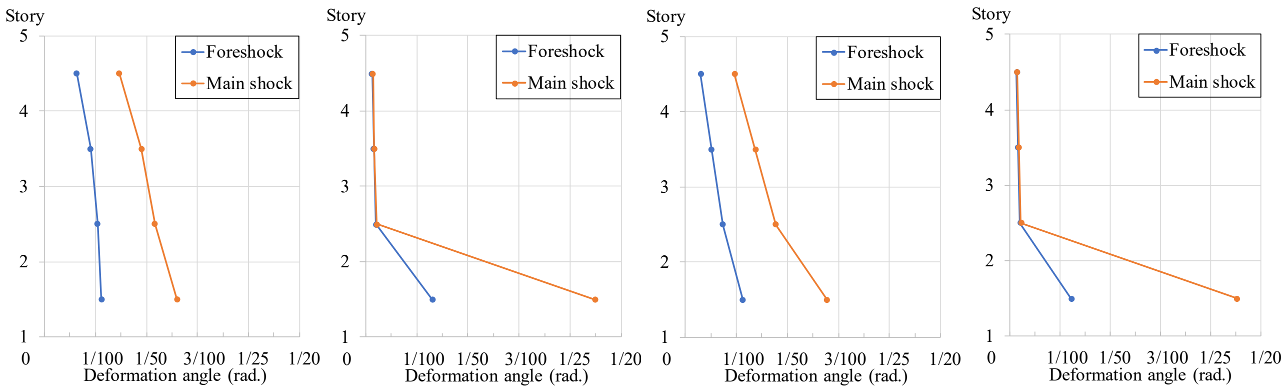
Table 5 – Results of the analysis of each model pillars in the soft first story on the end (X1) face

(a) Share force (unit: kN)

Member	Model 1	Model 2	Model 3
X1-Y1 pillar	399	418	418
X1-Y7 pillar	410	429	429

(b) Compressive / tensile axial force (unit: kN)

Member	Model 1	Model 2	Model 3
X1-Y1 pillar	50／-70	390／-294	102／-111
X1-Y7 pillar	50／-71	391／-317	122／-134



(a) Model 1 (b) Model 2 (c) Model 3 (d) Model 4

Fig. 7 – Maximum story deformation angle (The long direction)

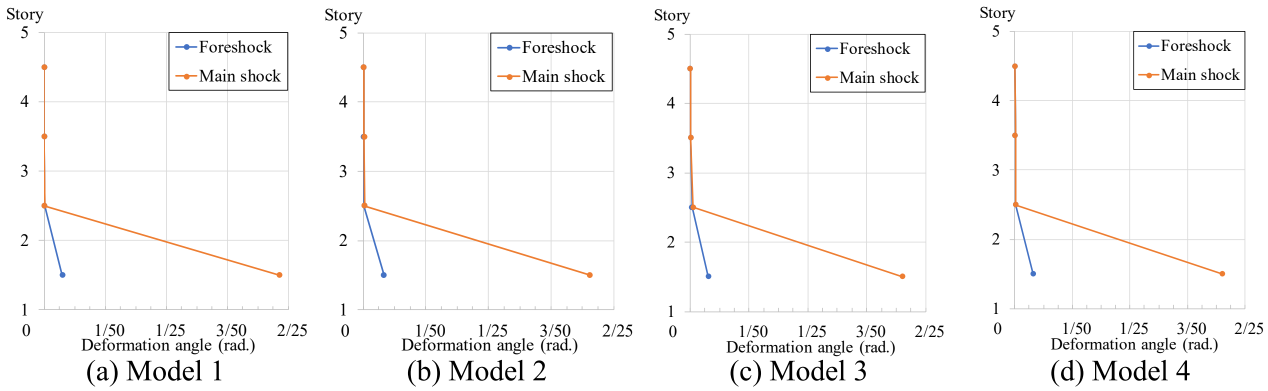
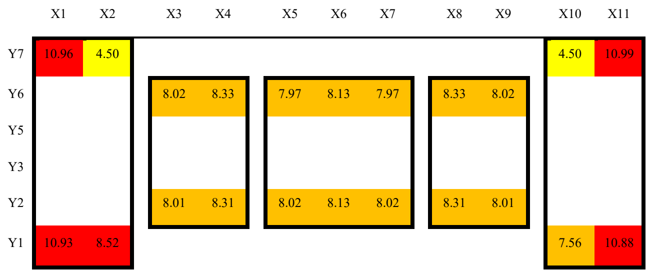


Fig. 8 – Maximum story deformation angle (The short direction)



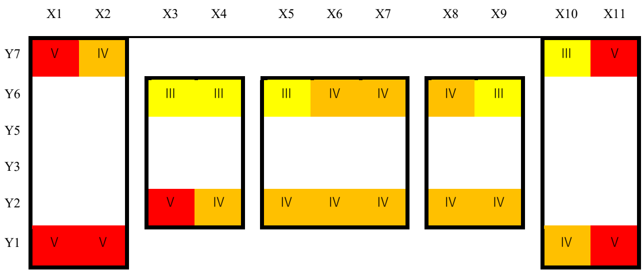


Fig. 9 – Plasticity rate of model 2

Fig. 10 – The member damage evaluation results

4. Investigation based on currently recommended soft first-story design methods

A building that was designed using the standard design methods at construction was modeled and analyzed. Deformation was found to be concentrated in the soft first story and led to a fracture in this story. The actual damage was similarly large in the soft first story. Therefore, the soft first-story pillars were redesigned based on a currently recommended soft first-story design method, which was a “design method that tolerates soft first-story fracture and total fracture” [2]. For the other parts, the long direction was considered as a pure open frame and the short direction was an open frame with partially sheared walls installed at X2, X4, X6, X8, and X10 frames.

The first story pillar material was changed to satisfy the design methods recommended in [2]. The following shows how the soft first-story pillars were redesigned.

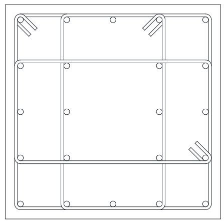
The pillar dimensions were determined and the rebars were placed to satisfy relevant specifications and rules. Therefore, a model of the building was created, and static increment and dynamic response analyses were conducted. The static increment analysis results were checked to confirm that the base shear coefficient exceeded 0.5 and the fracture mode was the bending fracture. To further examine the variable axis force in the pillars of the soft first story, the tensile maximum pillar top moment when loaded in the static increment analysis, M_l, and the final bending moment of the pillar considering the maximum tensile and compressive axial forces on the pillar bases, M_u, were compared. The latter must be larger than the former. The design and verification process were iterated to minimize the size of the members.

The results of the analysis are shown in Table 6, which shows the materials, and Table 7, which shows the information on the members of the soft first story after the analysis.

Table 6 – The improved materials

Concrete	Type	Strength (Kg/cm^2)	Member
	Ordinary concrete	210	All
Reinforcement	Type		
	High strength bar	SBPD1275	U12.6
	Deformed bar	SD490	D25

Table 7 – The information on the improved members of the soft first story (unit: mm)

Cross-section	
Dimension	925×925
Main reinforcement	22-D25
Hoops shear reinforcement	-U12.6-@50

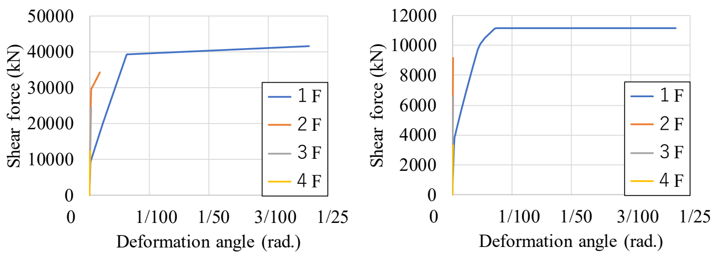
4.1 Results of soft first-story pillar investigations

The following shows the results of the static increment and dynamic response analyses in the short direction where the shear walls were installed.

The static increment analysis showed that the base shear coefficient exceeded 0.5 and the bending fracture was the preceding fracture mode. The maximum tensile and compressive axial forces with a positive load in the short direction were 1218 and 1245 kN, respectively. In contrast, with a negative load in the short direction, the maximum tensile and compressive axial forces were 1229 and 1257 kN, respectively, and the tensile maximum pillar top moment was 2365 kN·m. The final bending moment of a pillar calculated using these axial forces was 2516kN·m. The first story shear force with the redesigned pillars was 3.7 times that of model 1, where modeling conditions were the same except for pillars on the first story (Fig. 11). Therefore, the proof stress improved significantly.

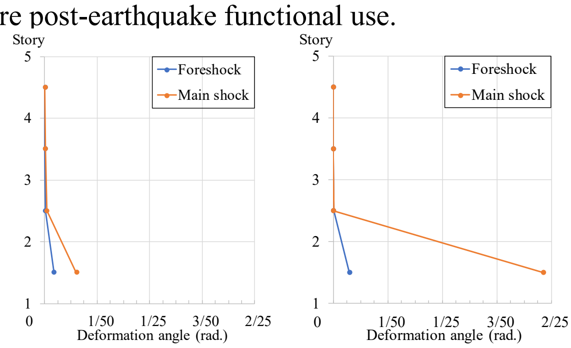
The dynamic response analysis showed that the maximum inter-story deformation angle of the first story was 0.5 and 0.15 times for the foreshock and main shock, respectively, in the redesigned soft first-story pillar mode compared to that of model 1, where modeling conditions were the same except for pillars on the first story. Thus, the deformation in the redesigned pillar model was much smaller than that in model 1.

Appropriate materials for soft first-story members based on currently recommended soft first-story design methods were identified as SD345-D22 to SD490-D25 for the main rebar and SD295-D13to SBPD1275-U12.6 for the shear reinforcement. The dimensions were 650 mm × 650 mm to 925 mm × 925 mm; thus the members were excessively large. Although the currently recommended soft first-story design methods increase the proof stress and significantly reduce the deformation, the members are too large and unrealistic for actual construction. However, there is a large demand for soft first-story buildings in Japan and many more of these will be built. A future study should involve investigating the design methods that reduce the size of members as much as possible but still ensure post-earthquake functional use.



(a) Redesigned model (b) Model 1

Fig. 11 – Comparisons of Q-R curves



(a) Redesigned model (b) Model 1

Fig. 12 – Maximum story deformation angle

5. Summary and Future Tasks

The present study modeled a soft first-story RC building that was damaged in the 2016 Kumamoto earthquake. The effects of member modeling differences on the response were investigated. The actual damage was compared with that of modeling from the viewpoint of responses and the following insights were obtained:

(1) Effect on the response of difference in modeling

The shear force of the first story was 11000 kN in model 3, where frames in the long direction were walls with slits on three sides. The value was 11500 kN in model 2, where frames in the long direction were walls with slits on two sides. A higher horizontal stiffness in the second story and higher stories, which had walls, resulted in a deformation concentration on the first story.

(2) Effect on the response of difference in modeling of slabs without beams

The story shear force and deformation angle in each story were almost the same; thus, there was barely any effect on the response.

(3) Estimation of structural slit design conditions

Comparison of actual damage and dynamic response analysis results showed that the walls of the building had slits on two sides, confirming that model 2 in the present study was the most reasonable.

(4) Seismic performance required in soft first-story structures in current standards

Current soft first-story building design to prevent severe damage requires excessively large members, thus design methods that reduce member size as much as possible while ensuring post-earthquake functional use must be investigated.

6. Acknowledgements

We acknowledge the use of the temporal acceleration data from seismic motion observation systems K-NET and KiK-NET operated by the National Research Institute for Earth Science and Disaster Resilience as seismic motion observation data.

7. References

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