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PROCESS OF COLLAPSE FOR RC FRAME INCLUDING SHEAR COLUMN

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Abstract: In this paper, experimental study for axial collapse of reinforced concrete structure composed of shear and flexural member was conducted. From the experiments and equilibrium of force when column failed by axial load, axial capacity analysis model of failed shear column was proposed. Idealized backbone curve of failed shear column was also constructed from the model. This model showed good agreement with shear column of specimens. After that, pushover collapse analyses of frame model of specimens were conducted. The model mentioned above was installed to the frame model. From the results, collapse analysis method and model are almost valid in comparison with especially more brittle specimens.

1. INTRODUCTION

Some of old reinforced concrete structures might have shear critical members. Failure of shear critical columns may cause a total collapse of building because of rapid degradation of horizontal and axial capacity. In Japanese seismic code and design standards (AIJ.2004), deterioration in shear resistance in column has not been considered because of complexity and unclearness of such behavior. Therefore, safety limit state of buildings (maximum deformation point of buildings to prevent collapse) is generally taken at the first occurrence of shear failure of a structural member.

However, this safety limit state is conservative because buildings might not collapse directly after shear failure if the horizontal and vertical forces can be redistributed from failed members to surrounding members. In addition, previous research (Mukai et.al 2010) showed that redistribution behavior was observed in experiment of RC frame including shear and flexure columns.

Main goal of this paper is to estimate that kind of behavior of RC frame. In this paper, an analytical model of residual axial capacity of column was constructed. After that, static cyclic loading experiments of reinforced concrete frames composed of brittle shear and ductile flexural columns were conducted to confirm suitability of the model.

2. MODEL OF RESIDUAL CAPACITY OF BRITTLE SHEAR COLUMN

2.1 Review of Previous Model

For reasonable collapse assessment, analytical model of deterioration of shear and axial capacity is needed. In past, models to assess the backbone characteristics of such brittle members considering shear-axial interaction were proposed by Elwood and Moehle (2004) and Yoshimura (2008), and the authors (2012).

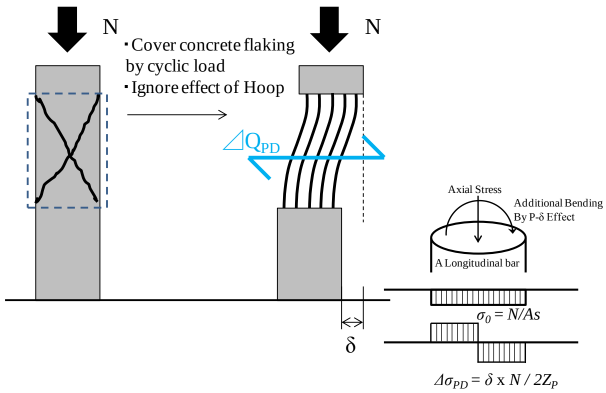


Figure1 Residual axial capacity model

Residual axial capacity model in authors' previous paper (2012) is shown in Figure1. And EQ.(1) is constructed from equilibrium of stress when longitudinal bars yield and column reaches axial collapse.

$$N_{R\_L} = \frac{As \cdot \sigma_y}{\left(1 + \dfrac{As \cdot \delta}{2Z_p}\right)} \tag{1}$$

Where; N_{R_L}: residual axial capacity by longitudinal bars, Z_p: plastic section modulus of longitudinal bars, As: total area of longitudinal bars, σ_y: yield stress of longitudinal bars, δ: lateral drift of column.

As shown in Figure1, P-δ effect to longitudinal bars was considered in Eq.(1). Basic assumption of this model is as follows;

1) Cover concrete was totally removed due to shear failure and cyclic loading
2) The influence of transverse reinforcements is small and therefore ignorable
3) Horizontal force of column is assumed to reach zero by capacity deterioration when vertical load N exceeds vertical capacity of column

In this paper, the effect of transverse reinforcement (mentioned assumption 2)) will be taken into consideration and is thoroughly studied.

2.2 Effects of transverse reinforcement

Transverse reinforcement contributes to residual capacity of column by confining crushed concrete. This effect is illustrated in Figure 2. After shear failure, wedge shaped area (see figure 2) is formed top and bottom of the damaged area. Transverse reinforcement resists the axial load *N* by confining the crushed concrete area. When vertical collapse occur, transverse reinforcements yield and equilibrium of force is constructed as shown in Eq.(2). Eq.(2) and Eq.(1) are added to form Eq.(3).

$$N_{R\_T}\cdot\tan\gamma = \frac{2D_e\cdot A_{sw}\cdot\sigma_{wy}}{s\tan\phi} \quad (2)$$

$$\begin{aligned} N_R &= N_{R\_L}+N_{R\_T} \\ &= \frac{A_s\cdot\sigma_y}{\left(1+\dfrac{A_s\cdot\delta}{2Z_p}\right)} + \frac{2D_e\cdot A_{sw}\cdot\sigma_{wy}}{s\tan\phi\tan\gamma} \end{aligned} \quad (3)$$

Where; N_R: total residual axial capacity, N_{R_T}: residual axial capacity by transverse reinforcement, D_e: effective width of the column, A_{sw}: total area of transverse bars, *s*: space of transverse bars, φ:angle of shear crack, γ:angle of wedge shaped area.

In previous research by Yoshimura (2008), backbone characteristics of residual shear and axial capacity were proposed (as shown in Figure3). In the backbone curve, collapse displacement is defined as R_u which is the point where shear capacity reaches zero and axial load N exceeds axial capacity N_R. Based on Eq.(3), the collapse displacement R_{u0} could be calculated by Eq.(4). Referring to Yoshimura model and Eq.(4), backbone curve of residual capacity could be calculated.

$$R_{U0}=\delta/h_0 = \frac{2Z_p}{A_s\cdot h_0}\left(\frac{A_s\cdot\sigma_y}{N-\dfrac{2D_e\cdot A_{sw}\cdot\sigma_{wy}}{s\tan\phi\tan\gamma}}-1\right) \quad (4)$$

Where; h_0: internal height of column.

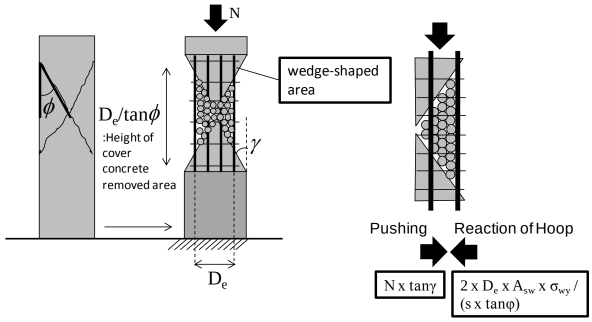


Figure2 Confinement effect of transverse bars

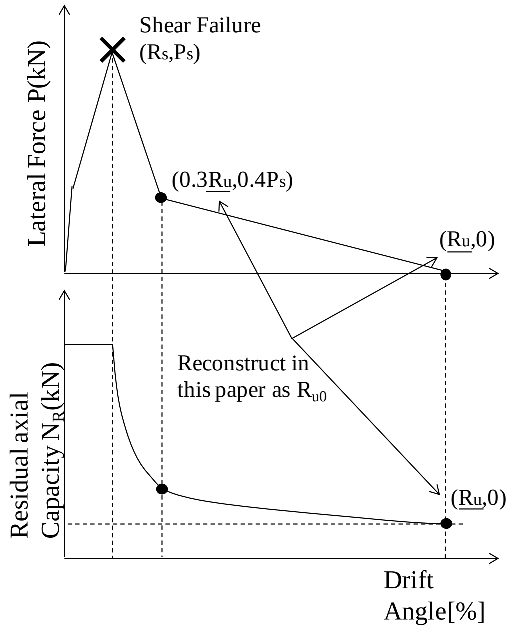


Figure3 Backbone curve of shear and axial capacity

3. EXPERIMENTAL STUDY OF RC FRAME INCLUDING SHEAR COLUMN

3.1 Test Specimen

Tests were conducted for five specimens; the example drawings are shown in Figure 4 and 5. These specimens are single story and two bays frame structures. Specimen F0101 (conducted with Fukuyama et al (2011)) was designed as a one-half scale and the others were designed as a 3/8 scale of typical building in Japan. Every member except for center column was designed so that flexure yielding precedes shear failure. Center columns were not provided with adequate transverse reinforcement and they were designed to demonstrate shear failure. Material properties of the specimens are shown in Table 1.

Main parameters of these specimens are p_w which is the area of transverse reinforcement normalized by the hoop spacing and column width and $N/A_s\sigma_y$ which is applied axial load divided by capacity of longitudinal reinforcement of center column. $N/A_s\sigma_y$ is index of axial capacity carried by longitudinal bars. Table2 shows the basic properties of center

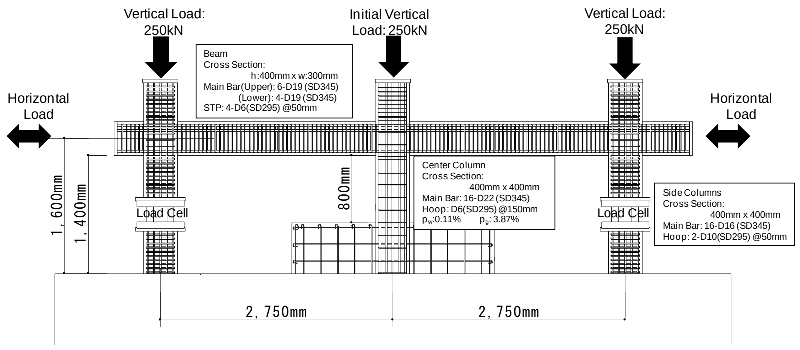


Figure 4 Drawing of specimen F0101

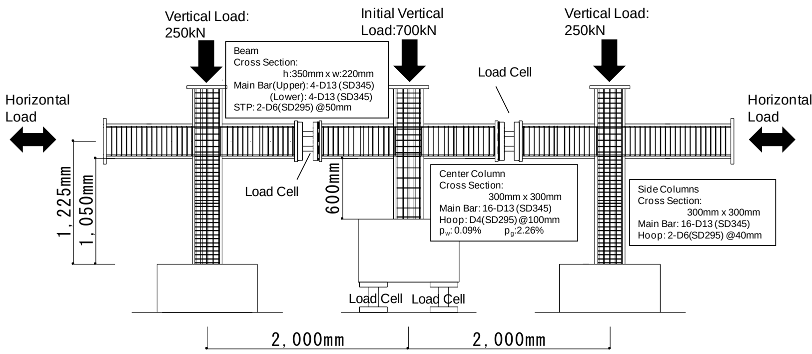


Figure 5 Drawing of specimen F0109

Table1 Material properties

	Concrete	
	Compressive Stress[N/mm^2]	Elastic modulas [N/mm^2]
F0101	34.7	2.56x10^4
F0109	26.9	2.60x10^4
F0103 F0203 F0102	29.2	2.25x10^4
		Yeld stress [N/mm2]
F0101	D6	370
	D10	318
	D16	378
	D19	379
	D22	396
F0109	D4	352
	D6	295
	D13	349
F0103 F0203 F0102	D4	365
	D13	381
	D16	366
	D19	382

Table2 Basic properties of center columns

Name	F0101	F0109	F0203	F0103	F0102
Cross Section[mm]	400x400	300x300	300x300	300x300	300x300
Longitudinal Reinforcement	16-D22(SD345)	16-D13(SD345)	12-D16(SD345)	12-D16(SD345)	12-D19(SD345)
p_g[%]	3.87	2.26	2.65	2.65	3.83
Transeverse Reinforcement	D6@150(SD295)	D4@100(SD295)	D4@40(SD295)	D4@100(SD295)	D4@100(SD295)
p_w[%]	0.11	0.08	0.21	0.08	0.08
Initial Vertical Load[kN]	250	700	300	300	250
η	0.05	0.29	0.12	0.12	0.10
N/Asσ y	0.10	0.99	0.34	0.34	0.20
R_{u0}[%]	10.71	1.18	2.42	1.96	4.81

columns. Center column of F0101 specimen has higher axial capacity as shown in Table2. Specimen F0109 has the poorest lateral reinforcement, therefore, expected to have brittle axial collapse. F0203 and F0103 are almost identical specimen except for transverse reinforcement. These specimen has designed to confirm effects of N_{R_T} (residual axial capacity by transverse reinforcement). F0102 has almost same longitudinal reinforcement ratio (p_g: section area of longitudinal bars normalized by cross section area of each column) with F0101 although axial load is two times larger than F0101.

In addition, side columns and half of beams of F0109 were reused at F0103, F0203 and F0102 because these members were less damaged.

3.2 Measurement and Loading Cycles

Horizontal cyclic loads are applied by two hydraulic jacks fixed at the ends of beams. Load cells to measure axial stress and shear stress are installed at middle of side columns of F0101 (see Figure4), bottom of center column and middle of beams of the other specimens (see Figure5).

F0101 was subjected to three cycles of 1/800rad, 1/400rad, 1/200rad, 1/100rad, 1/50rad and 1/33rad of story drift angle which are calculated as average horizontal displacement of the side columns divided by story height which is the height from top of stub to center of top joint of the each column. The others were subjected to two cycles of 1/800rad, 1/400rad, 1/200rad, 1/100rad and 1/67rad of story drift angle.

In addition, although cyclic loading were conducted until specimens showed axial collapse, additional axial load was applied to center column of specimens which didn't show axial collapse.

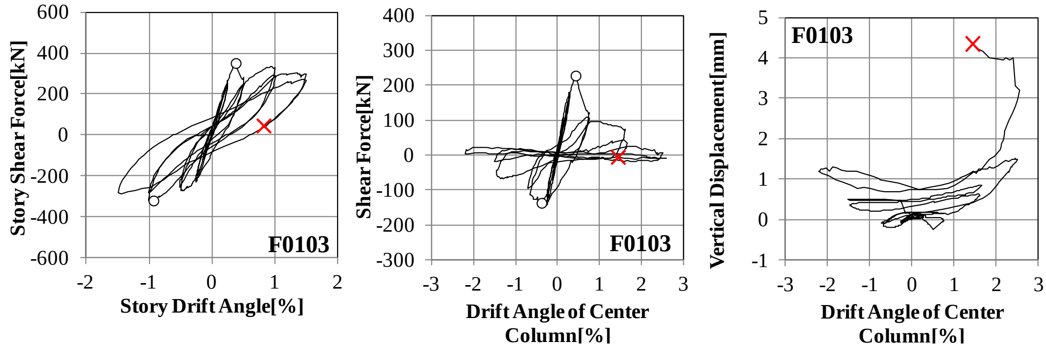


Figure6 Results of F0103

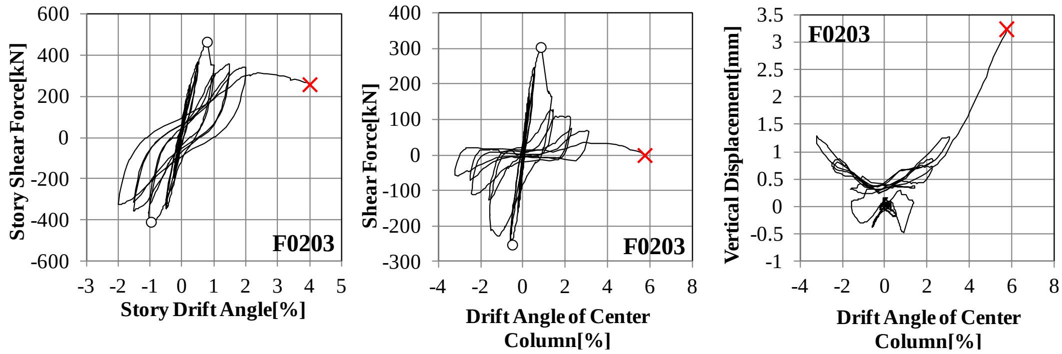


Figure7 Results of F0203

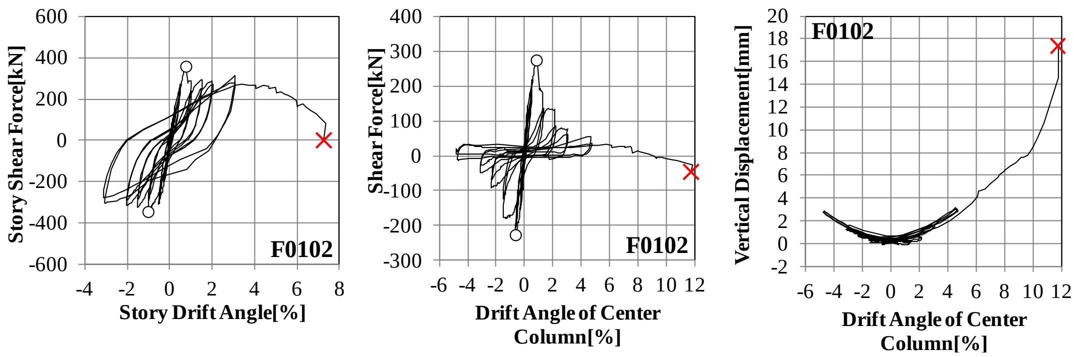


Figure8 Results of F0102

3.3 Test Results

In this paper, results of F0203, F0103 and F0102 will be mentioned in detail and refer to authors' previous paper (2011) for F0101 and F0109. Results of F0103, F0203 and F0102 are shown in Figure6, Figure7 and Figure8, respectively.

F0103 specimen showed about 350kN of maximum story shear force at 0.36% of story drift angle (see Figure6). After that, shear failure of center column occurred and shear capacity of the column and story rapidly decreased. After the cycle of 1/67rad, vertical displacement (downward direction as +) rapidly increased and vertical collapse (3hinges in the beams) was observed. In addition, at the point of vertical collapse, shear capacity of the center column reaches almost zero.

F0203 showed more ductile behavior than F0103(see Figure7). About 450kN of maximum story shear force was shown at 0.8% of story drift angle. Gradual decreasing of shear capacity was observed after shear failure of center column. In the process of decreasing, although vertical displacement was increased, vertical collapse was not observed until 4% of story drift angle. Ductility of F0203 was significantly improved by comparing F0103 which was given a half of transverse reinforcement.

In specimen F0102, at 0.75% of story drift angle, about 350kN of maximum story shear force and shear failure of center column were observed (see Figure8). Shear capacity of story decreased until 2% of story drift angle, after that, it raised until 4% of story drift angle. The reason is that shear force of side columns raised at that region regardless of decreasing of shear capacity of the center column. In region of over 4% of story drift angle, story shear capacity was decreased because side columns also showed shear failure (see Figure9). Vertical displacement of a side column significantly increased, therefore, relative vertical displacement of center column which means deformation of a beam wasn't increased. It is the reason why vertical displacement when F0102 collapsed such a large vertical

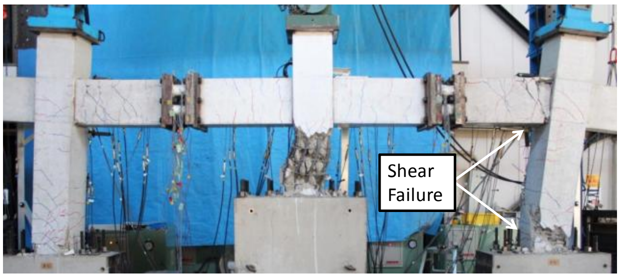


Figure9 F0102 at 6% of story drift angle

displacement (see Figure8). Then horizontal loading was stopped and additional axial load was applied to center column. Because of additional loading, vertical displacement of the column is rapidly increased and specimen showed vertical collapse.

3.4 Observation of Pictures

Figure10 shows pictures when shear failure occurs. Angles of shear crack are almost 30° on average, therefore, 30°was taken as φ.

After the loading, for center column of F0103, F0203 and F0102, broken concrete was raked from the core of columns to observe the wedge shaped area mentioned above as shown in Figure11. The angles of wedge shaped area is also shown in Figure11, they ranged from 24° to 57° and 40°on average, therefore, 40°is taken as γ.

In addition, referring Figure11, although snapped transverse reinforcements were observed, all the bars are may not be effective to resist to axial load. On average, half of transverse reinforcements were snapped and they're estimated to work as confinement, therefore, the effect of transverse reinforcements are considered as half(see Eq.(5),(6)).

$$N_R = \frac{A_s \cdot \sigma_y}{\left(1 + \dfrac{A_s \cdot \delta}{2Z_p}\right)} + \frac{2D_e \cdot A_{sw} \cdot \sigma_{wy}}{s \tan 30° \tan 40°} \frac{1}{2} \tag{5}$$

$$R_{U0} = \frac{2Z_p}{A_s \cdot h_0} \left(\frac{A_s \cdot \sigma_y}{N - \dfrac{D_e \cdot A_{sw} \cdot \sigma_{wy}}{s \tan 30° \tan 40°}} - 1 \right) \tag{6}$$

4. PUSHOVER ANALISYS

4.1 Analytical model and method

Pushover analyses are carried out to check suitability of the model mentioned above and estimate vertical collapse of structures. Structural model is shown in Figure12. First, members were distinguished as flexure members or shear members. Flexure members consist of an elastic shear spring, an elastic axial spring and inelastic flexure springs. Tri-linear model is applied to inelastic flexure springs based on AIJ standard (2010). Shear members have elastic flexure springs,

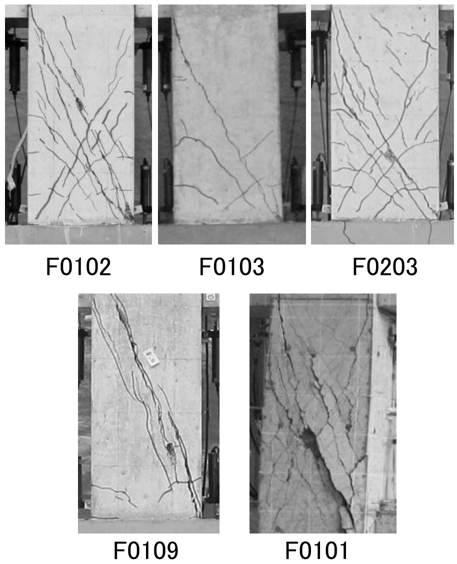


Figure10 Pictures at shear failure

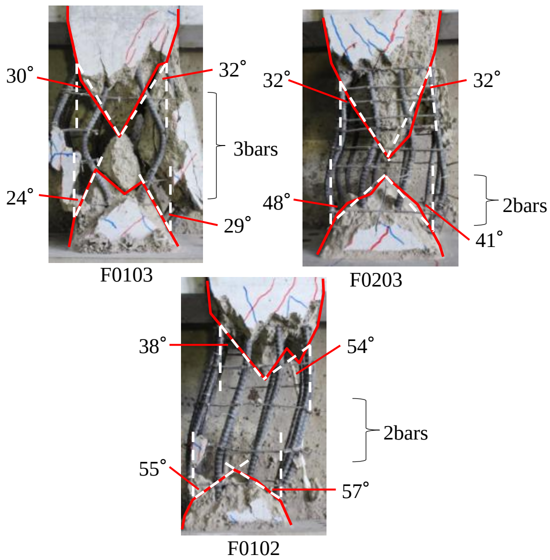


Figure11 Angles of wedge shaped area and snapped transverse reinforcement

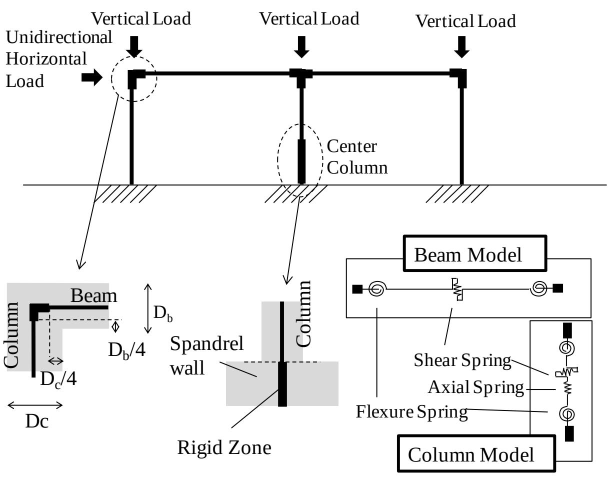


Figure12 Structural modeling

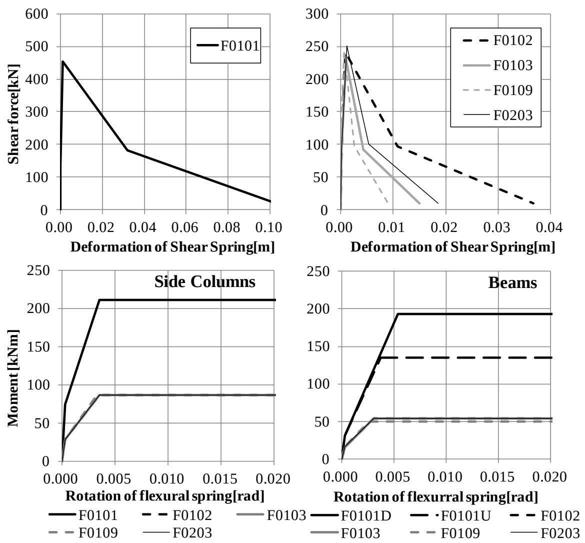
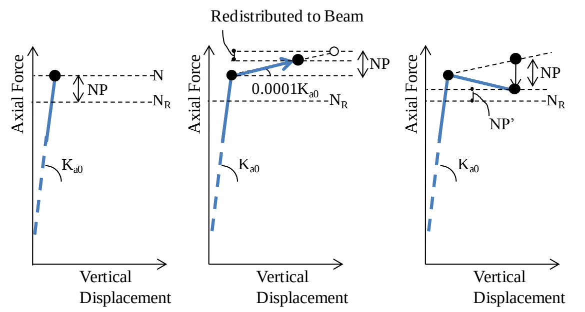


Figure 13 Backbone curve of inelastic springs



(a)Calculate NP (b) Calculate Force to Beams (c)Cancel NP

Figure14 Calculation of inelastic axial spring

an inelastic shear spring and an inelastic axial spring. quad-linear model similar to Figure 3 is applied to inelastic shear spring assuming 10kN as minimum shear capacity. Inelastic shear springs and flexure springs used in the analysis are shown in Figure13.

Calculation methodology of stress and displacement of inelastic axial spring is a bit complicated. As shown in Figure 14, residual axial capacity shown in Eqn.(5) and axial force carried by column is compared step by step. When former becomes lower than later, unbalanced load NP is calculated (Figure14(a)). To calculate vertical load carried by beams, then, stiffness of axial spring of the column was changed to 0.0001Ka (Ka: elastic axial stiffness) and NP was applied as vertical load to node at top of the column (Figure14(b)). After that, as shown in Figure14(c), NP was canceled from the column and NP' was calculate as residual unbalanced load (if NP' is not negligible, back to Figure14(a)).

Moreover, for F0101 and F0102 which were applied additional axial load in the middle of experiment, additional cases of pushover analysis are carried out because increasing vertical load in the middle of pushover is difficult. In these

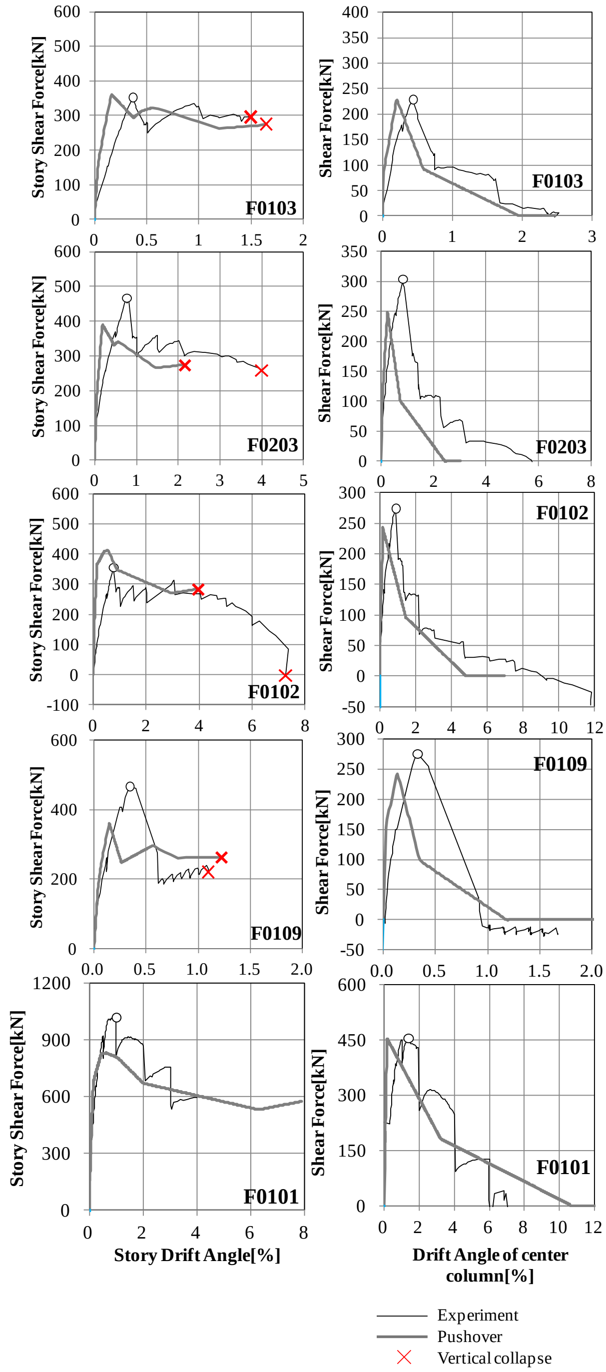


Figure 15 Comparison of experimental with analytical results

cases, maximum vertical load observed in the experiment was applied in the analysis (initial vertical load was applied in basic cases). If the model and analytical method are valid, experimental results expect to range between basic case and additional case.

4.2 Analytical Results

Figure15 shows analytical results and backbone curve of experimental results. As for Figure15, analytical results show good agreement with experiment especially their capacity degradation slope. Magnitude of correlation of

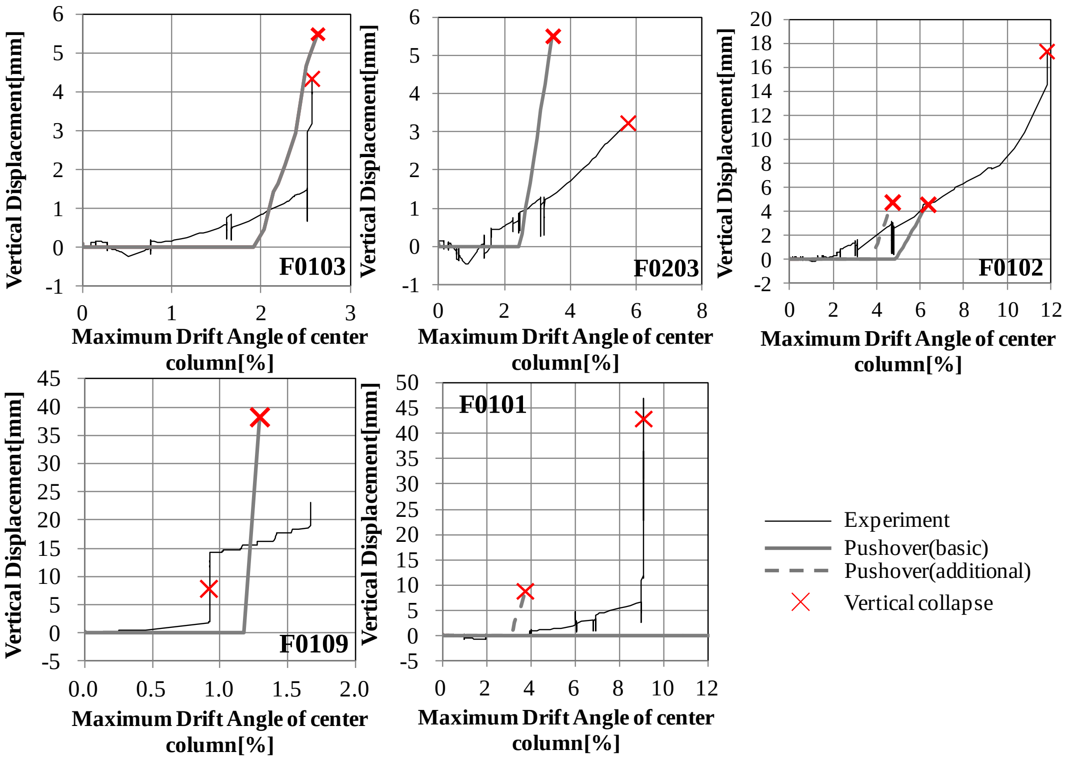


Figure16 Comparison of vertical displacement

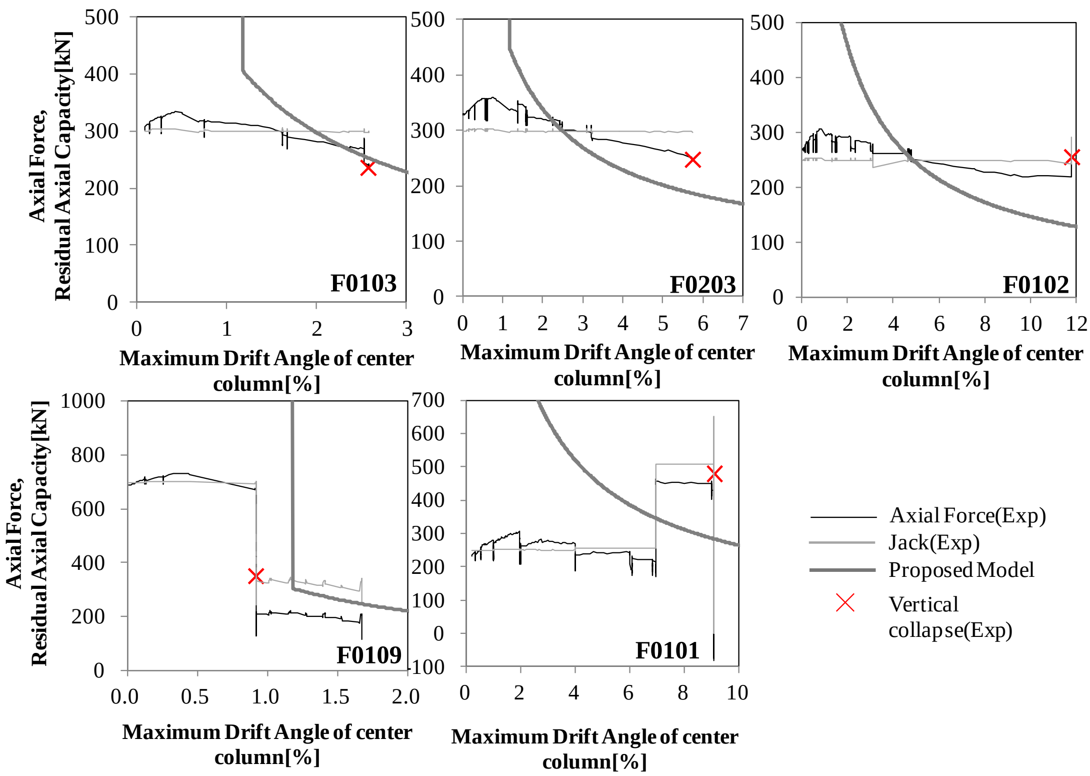


Figure17 Comparison of axial load (experimental with analytical model)

ductility of center column calculated by the model is also shows higher suitability. However, their maximum capacity were almost underestimated and elastic stiffness of analysis was more or less higher (but these are not main point of these studies).

Vertical displacement and maximum lateral drift of center column was shown in Figure16. In addition, for F0102 and F0101, the results of additional cases are also shown in Figure16. Analysis of more brittle specimens such as F0103 and F0109 shows higher suitability with the experiment especially point of vertical collapse and tendency of increment of vertical displacement. Although the other specimens are not good to estimate point of vertical collapse, all these analyses could estimate as safe side of experiment. In addition, collapse displacement of F0102 was significantly larger than both basic case and additional case of pushover.

Figure17 shows that residual axial capacity of the model and vertical loading process of the experiment. Vertical load to center column applied by jack in the experiment was also shown in Figure17, difference between axial load and vertical load by jack is distributed to beams as shear force. Axial capacity degradation and vertical load redistribution behavior was observed.

As shown in Figure17, All the specimens except F0103 and F0109 collapsed at higher axial force than residual axial capacity by the model. On the contrary, for more brittle specimen such as F0103 and F0109, residual axial capacity of specimens at collapse shows good agreement with the model. These results suggest that residual axial capacity by the model is almost valid for brittle specimens although tend to underestimate ductility for other specimens.

5. CONCLUSIONS

Contributions of this paper are shown in as follows;

1) From the Equilibrium of force, model to estimate the residual capacity of the column was constructed
2) Experimental study of RC frame including shear and flexure column was carried out. Vertical collapse was observed in the experiment.
3) Capacity degradation slope estimated by proposed model showed good agreement with experiment.
4) Displacement that vertical collapse occurred could be estimated by pushover analysis especially for more brittle specimens.

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