Simulation of Old Reinforced Concrete Column Collapse by Pseudo-dynamic Test Method

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SUMMARY:
During past severe earthquakes, many old reinforced concrete (RC) buildings suffered story collapse due to shear failure and the following axial collapse of columns. However, the process of column collapse is still unclear because it is difficult to precisely simulate this process by analytical methods. To understand column collapse, therefore, experimental approaches are necessary. In this study, pseudo-dynamic tests were conducted to examine the dynamic responses until the collapse of model specimens, which simulated old (before 1971) three-story RC buildings. The tests enabled us to detect how old buildings with shear-failing columns reach collapse. The study revealed that for ground motions at the design level intensity, old buildings in Japan that do not satisfy the present seismic regulations are in high danger of collapsing. Strengthening those buildings before future earthquakes is of the utmost urgency.

Keywords: Reinforced concrete, Pseudo-dynamic test, Old buildings, Shear failure, Collapse

1. INSTRUCTION

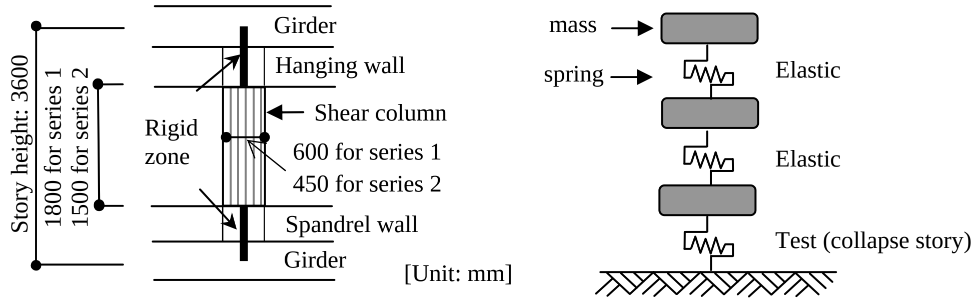
Whenever severe earthquakes occur around the world, many old reinforced concrete (RC) buildings collapse, due to shear failure and the following axial collapse of columns. However, the process of column collapse is still unclear because it is difficult to precisely simulate this process by analytical methods. To understand column collapse, therefore, experimental approaches are necessary. Shaking-table tests are the most effective for this purpose (Inoue et al. 2000, Elwood et al. 2001, Kim et al. 2002), but they are expensive and somewhat troublesome. Pseudo-dynamic tests are much easier to carry out than are shaking-table tests. While pseudo-dynamic tests of shear-failing RC columns have been conducted (Ikeda et al. 1990), tests until collapse have not been done.

In this study, the dynamic responses until the collapse of simulated old (before 1971) three-story RC buildings were studied by the pseudo-dynamic test method. The tests enabled us to detect the collapse process of old buildings with shear-failing columns.

2. OUTLINE OF TEST

2.1. Model building

Three-story RC buildings designed by Japanese codes before 1971 were considered. Figure 1 shows the buildings (series 1 and 2 in the figure are described later). The buildings were simplified to consist of a single column line and rigid beam. The first-story column, designed to fail in shear or in shear after flexural yielding, was tested by the substructure pseudo-dynamic method, whereas the second- and third-story columns were assumed to be strong enough to behave in an elastic manner during the tests.



(a) One story of a real-world building (b) Model building

Figure 1. Simulated three-story RC building

2.2. Specimen

A total of ten half-scale or full-scale model specimens representing first-story columns were fabricated. They were designed such that shear failure would certainly result. Table 1 summarizes the specimen structural properties. Reinforcement details of the entire specimen and the column sections are shown in Figure 2. The series 1 specimens are half-scale models and the series 2 specimens are full-scale models, described as follows. Series 1: height-to-depth ratio is 3.0 (height h_0=900 mm and depth D=300 mm); longitudinal reinforcement ratio (p_g) is 1.69%; hoop ratio (p_w) is 0.11%. Series 2: height-to-depth ratio is 3.3 (h_0=1500 mm and D=450 mm); p_g is 1.72%; p_w is 0.11% or 0.21%. The material properties are listed in Table 2. Each concrete strength listed in the table is the average before the first test and after the last test.

Imaginary mass was introduced so that the Seismic Capacity Index, I_S, prescribed by the Standard for Seismic Evaluation of Existing Buildings (Japan Association for Building Disaster Prevention 2001), would be 0.41, 0.49, and 0.62 for the first story by using a second-level procedure. This index is commonly used in Japan to evaluate the seismic performance of existing RC buildings. As briefly described in the Appendix, the computed I_S value is based on the strength and deformability of each column. Buildings with higher I_S values are evaluated as possessing higher seismic performance. It is widely recognized that buildings with I_S values higher than or equal to 0.6 do not suffer severe damage, including collapse, even in severe earthquakes such as the 1995 Kobe Earthquake. The first mode period of the tested buildings were computed to range from 0.12 to 0.16 seconds. Note that it was assumed that each story had the same mass and initial stiffness. During the tests, instead of the imaginary mass, two levels of axial load were considered: 0.17 and 0.20 times the concrete strength (σ_B) multiplied by the column section.

Table 1. Specimen structural properties

Series	Name[1)]	b × D	h_0	Hoop p_w (%)	Longitudinal bar p_g (%)	Axial stress ratio η[3)]	Input ground motion	I_S value	Imaginary mass (kN)	Initial period (s)
1	C13-J	300 mm × 300 mm	900 mm	0.11 (2-D6[2)]@200)	1.69 (12-D13[2)])	0.20	JMA	0.49	398	0.15
	C13-T						TOH			
	C13-H						HCO			
2	4J11	450 mm × 450 mm	1500 mm	0.11 (2-D10[2)]@300)	1.72 (12-D19[2)])	0.17	JMA	0.41	215	0.15
	4W11						WAK5-7			
	4H11						HCO			
	6J11						JMA	0.62	133	0.12
	4J21			0.21 (2-D10[2)]@150)			JMA	0.41	249	0.16
	4W21						WAK5-7			
	6J21						JMA	0.62	154	0.13

1) Series 1: In the name, the number 13 denotes the longitudinal-bar diameter in mm, and the letters J, T, and H denote the input ground motion. Series 2: In the name, the numbers 4 and 6 denote the I_S value, and the letters J, W, and H denote the input ground motion. In addition, in series 2, the numbers 11 and 21 denote the hoop ratio as a percent.
2) p_w and p_g: The numbers after D denote the bar diameter in mm.
3) Axial stress ratio η=N/(b×D×σ_B), where N: axial load, σ_B: concrete strength.

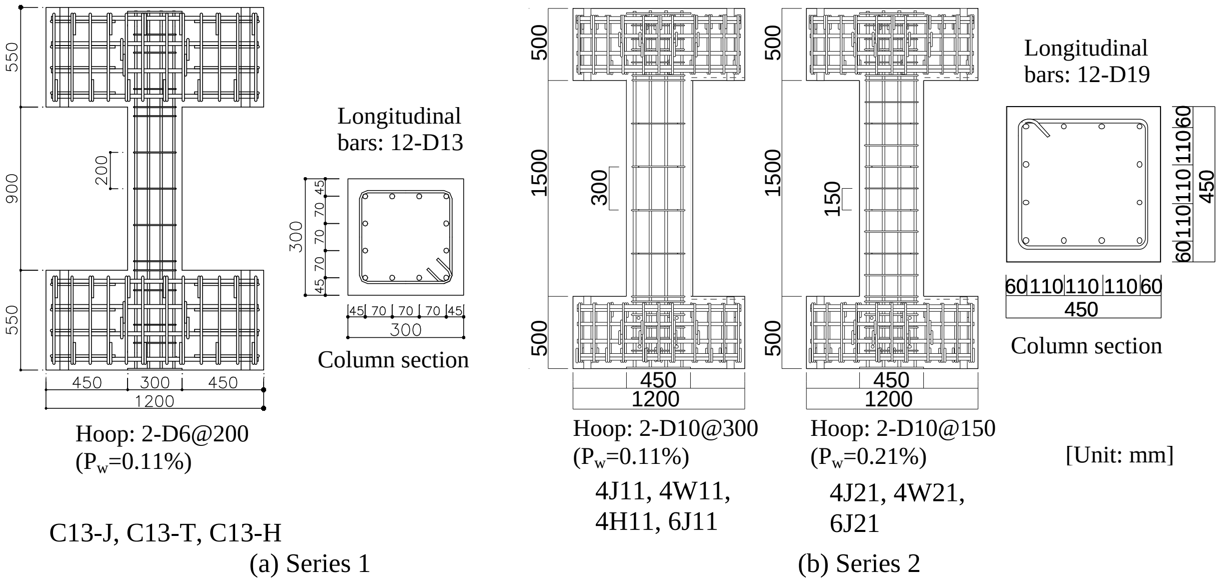


Figure 2. Reinforcement details of the specimens

Table 2. Material properties

(a) Series 1

Steel

	Yield stress (N/mm^2)	Strain at yield stress (%)
D13	371	0.201
D6	366	0.245

Concrete

Max. stress σ_B (N/mm^2)	Strain at max. stress (%)
22.0	0.186

(b) Series 2

Steel

	Yield stress (N/mm^2)	Strain at yield stress (%)
D19	403	0.206
D10	360	0.230

Concrete

Max. stress σ_B (N/mm^2)	Strain at max. stress (%)
19.5	0.185

The shear and flexural strengths of each specimen were computed by the conventional equations in Japan and are listed in Table 3 (Architectural Institute of Japan 1991).

Table 3. Computed strengths

Series	Name	Shear strength (kN)	Flexural strength (kN)	Strength ratio (Shear/Flexure)
1	C13-J	145	207	0.70
	C13-T			
	C13-H			
2	4J11	271	406	0.67
	4W11			
	4H11			
	6J11			
	4J21	304		0.75
	4W21			
	6J21			

2.3. Test apparatus and test procedures

The test apparatus is shown in Figure 3. The pantograph is placed so that the loading beam at the column top does not rotate (double curvature deformation must be realized). The numerical integration method of the pseudo-dynamic test was the central difference method, used except in the first few steps, in which the linear acceleration method was used. Damping was assumed to be of the viscous type and proportional to the initial stiffness with a damping factor of 1% with respect to the fundamental natural frequency.

The tests were performed with increasing ground motion levels until the column specimens eventually lost their axial load-carrying capacity. Four earthquake motions previously recorded in Japan were used as ground motions. Figure 4 shows the time histories of the ground accelerations of the four earthquakes: JMA (NS direction) at the 1995 Kobe earthquake, TOH (NS) at the 1978 Miyagiken-oki earthquake, HCO (N254E) at the 1994 Sanriku-harukaoki earthquake, and WAK5-7 (EW) at the May and July 2003 Miyagiken earthquake. The duration time of the ground motions of series 1 was 20 seconds and that of series 2 was 14 seconds. Considerations were the original level of the earthquake motions and the level adjusted to identify the levels necessary to induce collapse, and other levels as necessary. The input procedure of the ground motions is as follows.

Series 1: The adjusted earthquake motion was inputted only once for each specimen.

Series 2: For JMA, earthquake motions were adjusted to a maximum velocity of 37.5, 50, 75, and 100 cm/s and they were inputted successively until collapse occurred. For WAK5-7, the May earthquake (WAK5) and the July earthquake (WAK7) motions were inputted successively at the original level. For HCO, an original level (maximum velocity 29.2 cm/s) and an adjusted level (37.5 cm/s) of the earthquake motion were inputted successively.

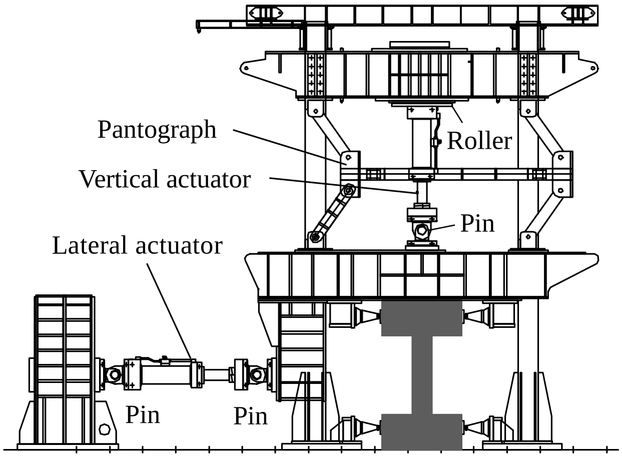
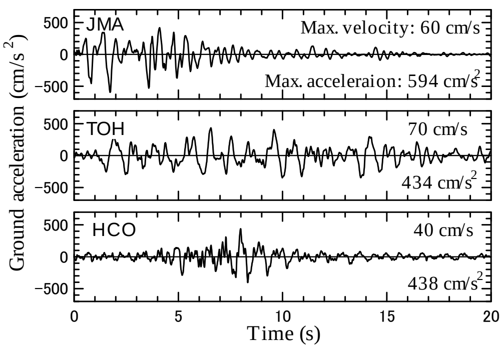
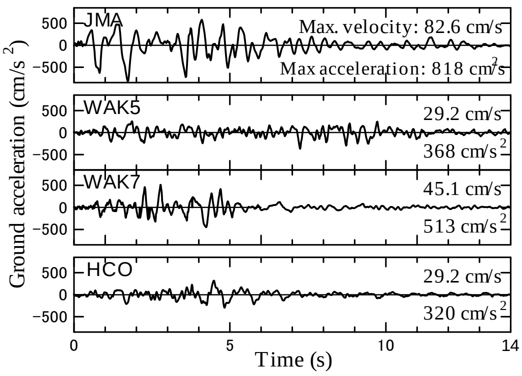


Figure 3. Test apparatus



(a) Series 1 (Adjusted level)



(b) Series 2 (Original level)

Figure 4. Time history of ground accelerations

3. TEST RESULTS

3.1. Collapse procedure

All specimens failed in shear and eventually lost their axial load-carrying capacity. The load step immediately before the sudden increase of axial shortening is defined as the ‘collapse’ step and the lateral drift at the final step is denoted as the ‘collapse drift’. The test results are outlined in Table 4. The terms ‘drift-shift ratio’ and ‘time between shear failure and collapse’ in the table are discussed later.

The time history of the lateral drift, and the lateral load vs. the lateral drift relations are shown in Figures 5 and 6 for the first story. The lateral drifts are divided by the column height. In Figures 5 and 6, each open triangle mark indicates shear failure and each solid circle mark indicates collapse. The damage states observed after collapse (C13-J, 4J11, 4J21, 6J21) and those observed after the previous input of collapse (4W11, 4W21) are shown in Figure 7. As shown in Figures 5 and 6, the lateral load suddenly decreased soon after shear failure occurred, and collapse occurred when the main shear crack widened and the lateral load decreased to approximately zero.

For specimens 4J21 and 4W21 with a hoop ratio (p_w) of 0.21%, flexural yielding occurred before shear failure. However, specimen 6J21 with the same p_w of 0.21% failed in shear without flexural yielding. The reason why the failure mode was different in spite of the same p_w is unclear. Note that the border of the shear mode and the flexural yielding mode lies between the strength ratios of 0.71 to 0.73 (Yoshimura 2008). The strength ratio of 4J21, 4W21, and 6J21 was 0.75 (see Table 3), and the difference might be affected by the strength ratio. For example, the comparison of damage states observed after collapse is shown in Figure 7(b) for 4J21 and 6J21. Specimen 4J21 eventually failed in shear after flexural yielding, so its damage state resembled that of 6J21with no flexural yielding.

The collapse procedure of 4W11 and 4W21 using original-level ground motions is as follows. As shown in Figure 7, the damage of 4W11 and 4W21, observed after the previous input of collapse (WAK5), was slight. Both specimens did not appear to collapse immediately. However, they collapsed immediately after the following ground motions were inputted. This indicates that even if column damage seems to be slight, it could lead to collapse after the earthquake.

Table 4. Outline of test results

Series	Name	Max. load (kN)	Drift at flexural yielding (%)	Drift at max. load (%)	Drift at shear failure (%)	Collapse drift (%)	Axial deformation at collapse (%)	Drift-shift ratio (%)	Time between shear failure and collapse (s)
1	C13-J	191	-	0.66	0.81	1.66	0.56	84	0.89
	C13-T	201	-	0.68	1.10	2.48	0.66	36	1.99
	C13-H	212	-	0.95	0.96	2.87	0.79	17	4.97
2	4J11	413	-	0.68	0.69	3.29	1.0	21	0.59
	4W11	432	-	0.88	0.88	2.40	0.53	35	0.56
	4H11	395	-	0.59	0.77	4.48	1.73	10	1.0
	6J11	436	-	0.72	0.72	3.24	0.55	37	1.02
	4J21	447	0.84	1.56	2.13	5.39	1.74	29	0.14
	4W21	429	0.91	1.04	1.12	2.96	0.83	53	-
	6J21	434	-	0.73	0.49	3.07	1.06	77	1.50

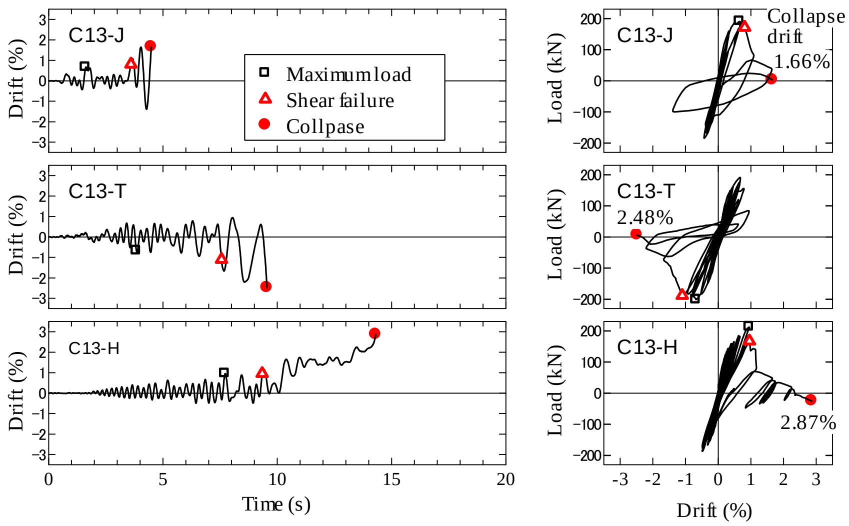


Figure 5. Time history of drift and lateral load vs. drift (series 1)

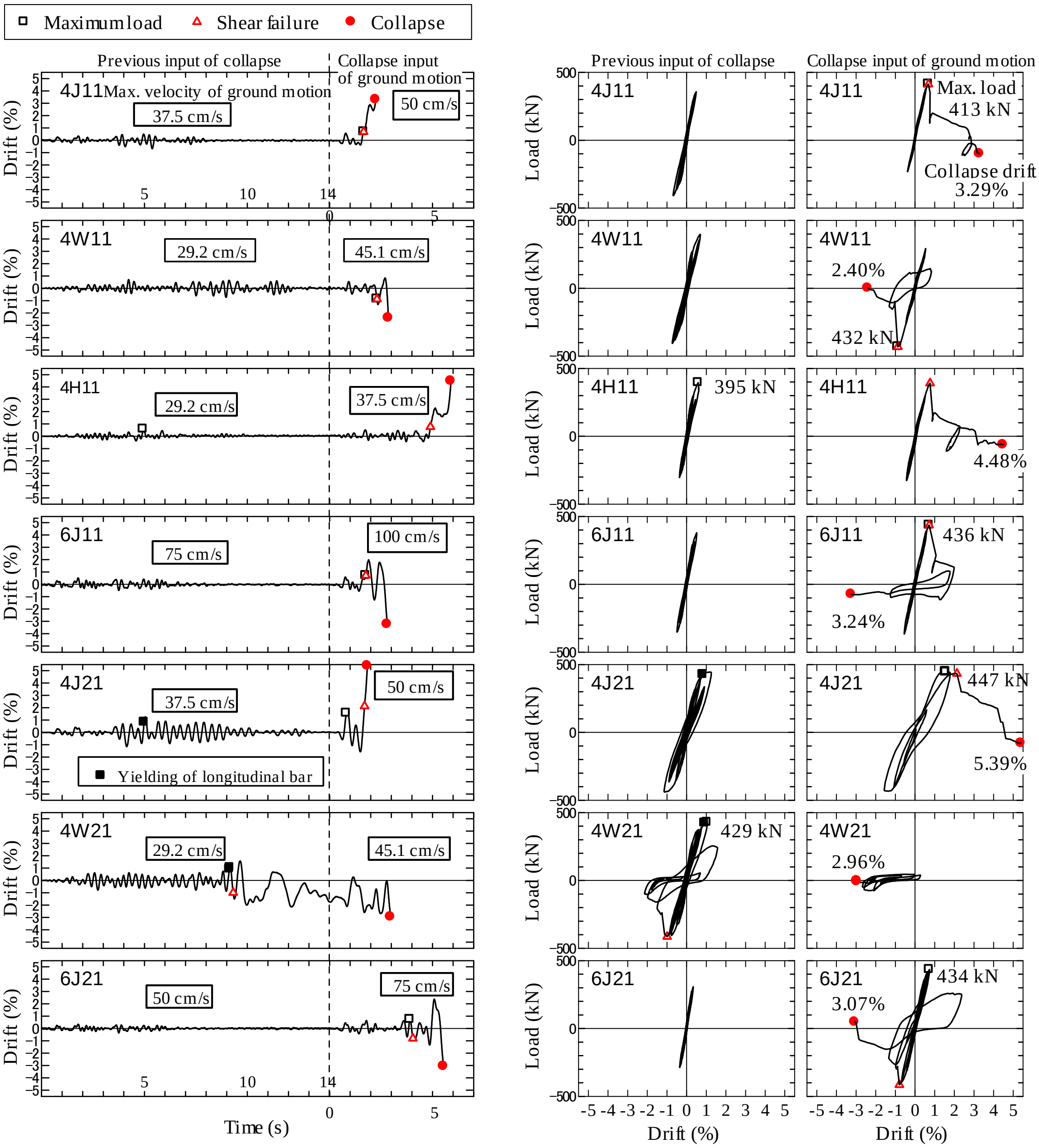


Figure 6. Time history of drift and lateral load vs. drift (series 2)

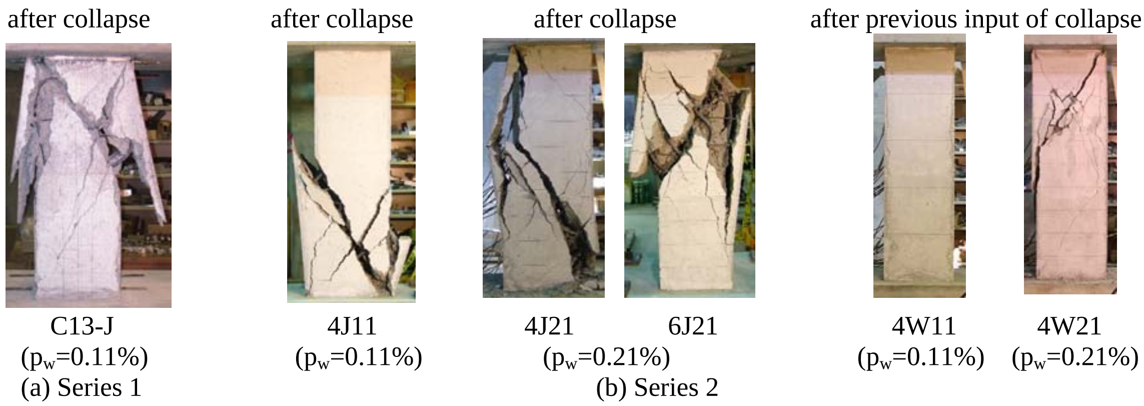


Figure 7. Damage states

3.2. Collapse drift

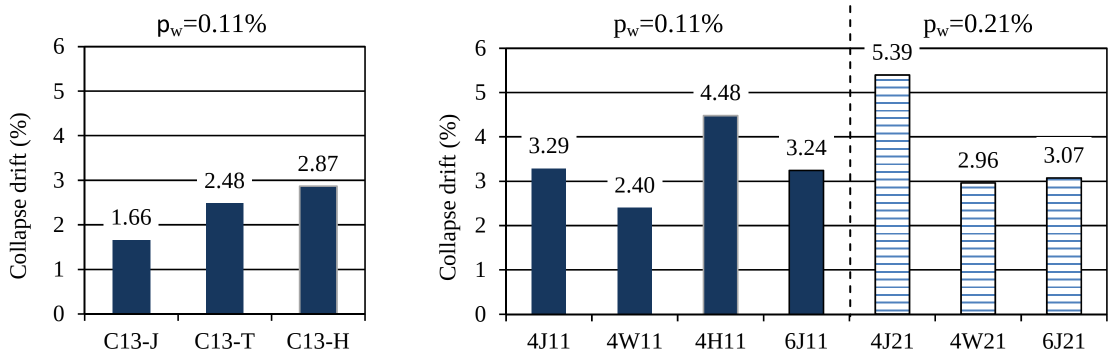
The collapse drifts are compared in Figure 8 and described below.

p_w=0.11%

For series 1 (C13-J, C13-T, C13-H), the collapse drift ranges from 1.66% to 2.87%. For series 2 (4J11, 4W11, 4H11, 6J11), the collapse drift ranges from 2.40% to 4.48%. These results indicate that the collapse drifts of columns, which have a small hoop ratio and fail in shear, are roughly the same irrespective of the loading history of the various earthquakes.

p_w=0.21%

For series 2 (4J21, 4W21, 6J21), the collapse drift ranges from 2.96% to 5.39%. The value for 4J21 was much larger than those of 4W21 and 6J21. This result could be due to the difference of the shear mode, as stated before.



(a) Series 1 (b) Series 2

Figure 8. Collapse drift

3.3. Necessary levels of ground motion that induce collapse

The relation between the I_S value and the necessary levels of ground motion that induce collapse is stated below.

Series 1

For the buildings whose I_S values were 0.49 (C13-J, C13-T, C13-H), the maximum velocity of ground motions that induce collapse ranged from 40 to 70 cm/s (maximum acceleration: 434 to 594 cm/s^2).

Series 2

For the buildings whose I_S values were 0.41 (4J11, 4W11, 4H11, 4J21, 4W21), the maximum velocity of ground motions that induce collapse ranged from 37.5 to 50 cm/s (maximum acceleration: 411 to 513 cm/s^2).

For the buildings whose I_S values were 0.62 (6J11, 6J21), the maximum velocity of ground motions that induce collapse ranged from 75 to 100 cm/s (maximum acceleration: 743 to 990 cm/s^2).

These results indicate that buildings whose I_S values were 0.41 or 0.49 collapsed due to ground motions at the current design level intensity in Japan. In contrast, buildings whose I_S value was 0.61 did not collapse until they suffered a more severe earthquake than one at the design level intensity. Thus, old (before 1971) buildings in Japan with I_S values less than 0.6 are in high danger of collapsing. Strengthening those buildings before future earthquakes is of the utmost urgency.

3.4. Ground acceleration when a column collapses

As shown in Figure 5, the lateral drift of C13-H increased gradually after a large portion of the ground acceleration was applied, and collapse occurred when the ground acceleration was small. This result indicates that collapse can occur even when the ground acceleration is small.

3.5. Drift shift

Figures 5 and 6 indicate that the lateral drift tended to shift to the direction where the collapse occurred. To study this phenomenon, a new ratio was defined as the ratio of the maximum drift in the opposite direction of collapse to the collapse drift. The ratio, expressed in percent, is hereinafter called the 'drift-shift ratio'. The calculation method of the drift-shift ratio is shown in Figure 9. The smaller the drift-shift ratio is, the greater the drift shift is. The small drift-shift ratio indicates that the load vs. drift relation is monotonic on one side. The drift-shift ratios are shown in Table 4 and Figure 10. For seven specimens, except C13-J, 4W21, and 6J21, the drift-shift ratios ranged from 10% to 37%; in short, lateral drifts tended to one direction. In contrast, for C13-J, 4W21, and 6J21, the drift-shift ratios ranged from 53% to 84%; in short, a pronounced shift of drift did not occur. Overall, for RC columns with brittle failure modes whose lateral load deteriorates sharply, the lateral drift tends to shift to one direction. This result indicates that when static tests are conducted, using monotonic loading as a loading pass is more appropriate than using cyclic loading for shear-failing brittle columns.

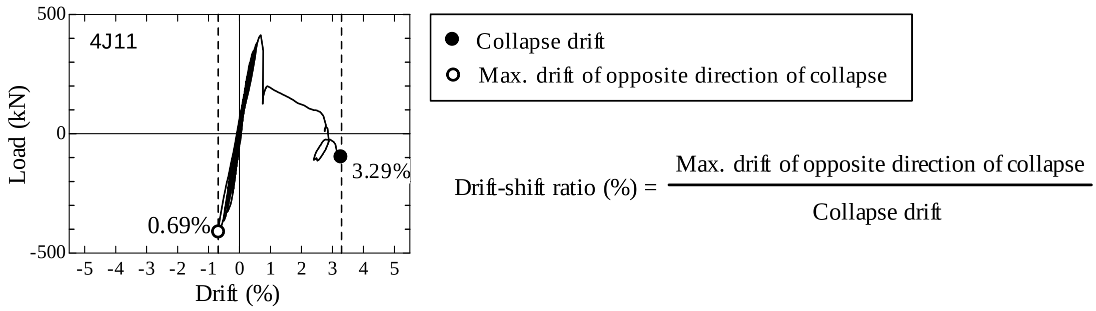


Figure 9. Evaluation of drift-shift ratio

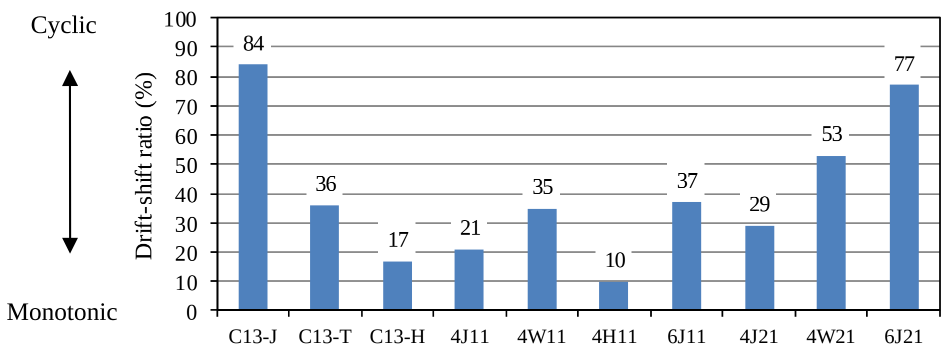


Figure 10. Drift-shift ratio

3.6. Time between shear failure and collapse

The time between shear failure and collapse (the time from the open triangle mark to the solid circle mark in Figures 5 and 6) is considered. A person can recognize the damage of a building only after the shear failure of a column appears as a wide crack. Thus, the time between shear failure and collapse equals the time that a person in a building needs to evacuate to outside of the building. The times between shear failure and collapse are shown in Table 4 and Figure 11. As shown, 4W21 was excluded because the shear failure and collapse occurred in different ground motion inputs. The times ranged from 0.59 to 4.97 seconds. This result indicates that the times are too short for evacuation.

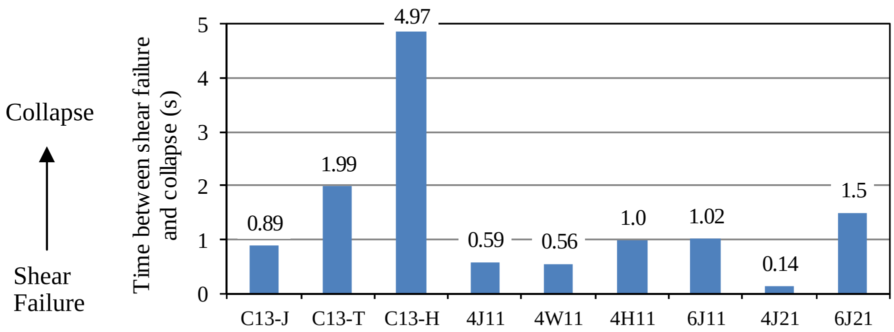


Figure 11. Time between shear failure and collapse

3.7. Axial deformation at collapse vs. collapse drift relations

Lateral drift vs. axial deformation relations for 4J11 and 4W21 are shown in Figure 12. The axial deformations are divided by the column height. The axial deformation increased rapidly after shear failure. The axial deformations at collapse are shown in Table 4. Similar behaviour was observed for the other specimens. Axial deformation in the collapse vs. collapse drift relations are shown in Figure 13. The average ratio of the axial deformation at collapse to the collapse drift is 0.30 irrespective of failure mode and the loading history of the various earthquakes, as shown by the fitted line in the figure. The axial deformation at collapse increases linearly with the increase of the collapse drift.

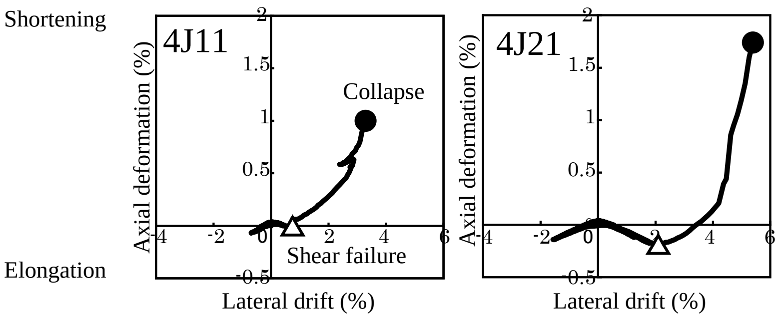


Figure 12. Lateral drift vs. axial deformation relations (specimens 4J11 and 4J21)

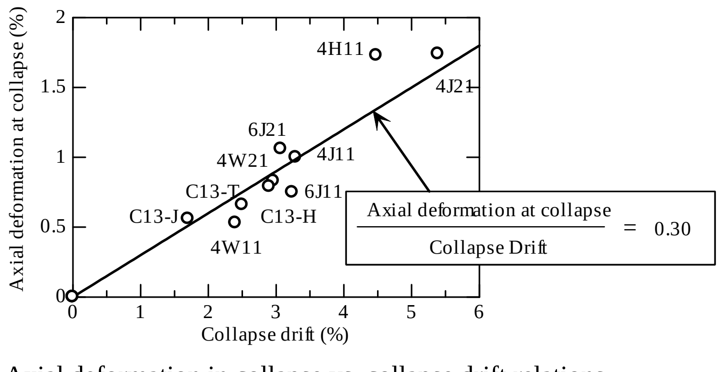


Figure 13. Axial deformation in collapse vs. collapse drift relations

4. CONCLUSIONS

The dynamic responses of old (before 1971) three-story RC buildings simulated by a single column line were studied by the substructure pseudo-dynamic test method. The tests were conducted until collapse for specimens simulating first-story columns failing in shear or shear after flexural yielding. The major findings from the study are as follows.

1) Buildings whose I_S values are 0.41 or 0.49 collapsed from ground motions at the current design level intensity in Japan (maximum velocity: 37.5 to 70 cm/s). In contrast, buildings whose I_S values were 0.61 did not collapse until they suffered a more severe earthquake than one at the design level (maximum velocity: 75 to 100 cm/s). Thus, old buildings in Japan with I_S values less than 0.6 are in a high danger of collapse, Strengthening those buildings before future earthquakes is of the utmost urgency.
2) The collapse drifts of columns that have a small hoop ratio and fail in shear are approximately the same, irrespective of the loading history of the various earthquakes.
3) For RC columns with brittle failure modes whose lateral loads deteriorate sharply, the lateral drift tends to shift to one direction.
4) For one specimen, collapse occurred when the ground acceleration was small. Thus, collapse can occur even if the ground acceleration is small.
5) The time interval between shear failure and collapse was too short for individuals to evacuate to the outside, out of the building.

APPENDIX

Seismic capacity index, I_S, is given by the following equation.

$$I_S = E_0 \cdot S_D \cdot T \tag{1}$$

where S_D is the configuration index, assumed as 1.0 for this study; T is the time index, assumed as 1.0 for this study; and E_0 is determined by the following equation.

$$E_0 = \frac{n+1}{n+i} \cdot C \cdot F \tag{2}$$

where n is the number of stories, and i is the story to be studied. Index C is defined as the strength of a column divided by the total weight of the floors above the column, whereas index F is determined according to the deformability of the column. The F value is calculated as 1.0 for the columns in this study.

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