ANALYSIS OF DETERIORATION BEHAVIOR OF COLUMNS USING MULTI-SPRING MODEL AND APPLICATION TO FRAME MODEL ANALYSIS

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Abstract

In order to simulate the deterioration behavior of building columns during a large earthquake by numerical analysis, it is necessary to use a column member deterioration model. On the other hand, considering the calculation cost of nonlinear response analysis, it is desirable that the member model be as simple as possible. Therefore, in this paper, the multi-spring model (hereinafter referred to as MS model) was adopted as a column member model that can evaluate the response when bending moment and axial force act simultaneously. And the simulation analyses of member tests of Rectangular Hollow Section (RHS) columns and concrete-filled RHS columns (hereinafter referred to as CFT columns) were conducted.

For the RHS column, a simulation analysis of a cyclic loading test with a constant axial force (30% of the yield axial force) was conducted. The spring element of the MS model incorporates the hysteresis model proposed by Ishida et al. (2014), which considers deterioration caused by local buckling. As a result, the experimental result was able to accurately be simulated for the strength and deterioration behavior in the horizontal direction and the progress of axial displacement after local buckling. This simulation confirmed the validity of the Ishida et al. (2014).

For the CFT column, a simulation analysis of a cyclic loading test with a constant axial force (30% of the yield axial force) was conducted. The hysteresis characteristics proposed by Ishida et al. (2014) was used for RHS, and the hysteresis characteristics combined with Fafitis and Shah (1985) and Maekawa et al. (2003), which considers deterioration associated with tensile crack and compression fracture, was used for concrete. As a result, the experimental result was able to accurately be simulated the strength and deterioration behavior in the horizontal direction and the progress of the axial displacement due to local buckling of RHS and compression fracture of concrete. Therefore, MS model is applicable to CFT column.

In addition, MS model was incorporated into the 3D frame analysis program. The developed analysis program was used to analyze the seismic response of a three-dimensional frame model for input ground motion three times as large as Japanese design motion. It was shown that this program can confirm the behavior of the building including column deterioration.

In the future, in order to clarify the collapse behavior of buildings, seismic response analysis for more large earthquakes will be conducted.

Keywords: RHS column, CFT column, Multi-Spring Model, Deterioration behavior, Frame model

1. Introduction

For high-rise buildings in Japan, time history response analysis is performed against design ground motions, and it is confirmed that the buildings and structural members satisfy the seismic safety criterion. In design, a strict deformation limit was set to limit the drift to 1/100 rad or less, and a limit is also set for the plasticity of the member. Therefore, except for the yield of the member, deterioration of characteristics such as local buckling and fracture are not considered.

On the other hand, in recent years in Japan, earthquakes exceeding the design ground motion have occurred frequently, such as the 2011 off the Pacific coast of Tohoku Earthquake and the 2016 Kumamoto Earthquake. Therefore, there is an increasing demand for accurately grasping the behavior of the building and for verifying the safety against unexpected large earthquakes. However, the member model in design cannot consider deterioration as described above, and is not suitable for the purpose of evaluating response to a large input.

In order to simulate the collapse behavior of a building against a large earthquake by numerical analysis, it is necessary to incorporate a model that can consider the deterioration behavior into a three-dimensional frame analysis model. Also, in consideration of the design, it is desirable that the member model be as simple as possible for the analysis of the three-dimensional frame model.

A member model that takes into account beam-end's local buckling and fracture of the lower flange of a beam has been proposed in Ref.[1]. It has shown that shaking table test [2] is simulated well using the member model.

On the other hand, a member model for simulating the column deterioration behavior against a large earthquake is not sufficient. For the local buckling of Rectangular Hollow Section (RHS) columns, a member model using Multi-Spring model [3][4] (hereinafter referred to as MS model) is proposed in Ref.[5]. However, no MS model applicable to CFT columns which can simulate deterioration behavior such as fracture of concrete under compression or crack under tension has been proposed.

In this paper, first, the simulation analysis of the cyclic loading test of the welded RHS column is shown, and the validity of the Ref.[5] is verified. Next, simulation analysis of cyclic loading test of CFT column is shown using the Ref.[5] model and the hysteresis model of concrete proposed in this paper. From the above, it is shown that the MS model is suitable for simulating the deterioration behavior of RHS columns and CFT columns.

Finally, a three-dimensional frame model is created, and using the developed MS model, static cyclic displacement control loading analysis is performed to confirm the whole building behavior and column behavior. Further, a time history response analysis is performed and the behavior of the whole building including the deterioration of columns is shown.

2. Multi-Spring Model (MS Model)

2.1 Overview of MS model

The Multi-Spring model is a model proposed by Lai et al (1984) [3], as shown in Fig. 1. This model consists of MS elements at both ends and line element connecting them, and is intended to represent the behavior of bending and axial deformation of column (for example [4]). In this paper, local buckling of RHS column and deterioration of concrete are represented by springs (MS elements) at the ends of the MS model which are given nonlinear characteristics.

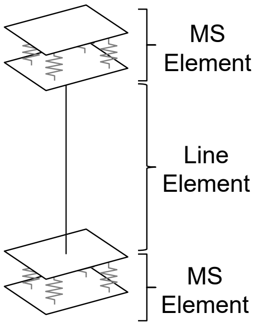


Fig. 1 – Multi-Spring model (called MS model)

2.2 Hysteresis model for MS element

For steel springs, the hysteresis characteristics proposed by Ishida et al. (2014) [5] as shown in Fig. 2 (a), which considers deterioration associated with local buckling, was used.

For concrete springs, the hysteresis characteristics combined with Fafitis and Shah (1985) [3] and Maekawa et al. (2003) [4] as shown in Fig. 2 (b), which considers deterioration caused by tensile crack and compression fracture, was used. On the tensile side, state D represents a constant stress, state E represents a degradation region, and state G is directed to the origin when unloaded. The compressive region is described in detail as below.

A) Elastic region: $0.2\,\sigma_c/E \le \varepsilon \le \varepsilon_t$

$$\sigma = \varepsilon E \tag{1}$$

B) Loading condition: $\varepsilon_c \le \varepsilon \le 0.2\sigma_c/E$

$$\sigma = 0.2\sigma_c + 0.8\sigma_c\left(1-\left(1-\frac{\varepsilon-0.2\sigma_c/E}{\varepsilon_c}\right)^{\varepsilon_c E/\sigma_c}\right) \tag{2}$$

C) Loading condition: $\varepsilon \le \varepsilon_c$

$$\sigma = \sigma_c \times \exp\left(-k\left(-\varepsilon+\varepsilon_c\right)^{1.15}\right) \tag{3}$$

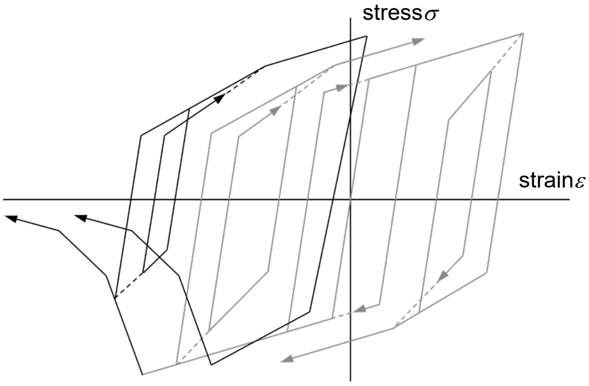
F) Unloading condition

$$\sigma = K_0 E\left(\varepsilon-\varepsilon_p\right)\left(K_0^2+\left(\frac{\sigma_{old}}{K_0 E\left(\varepsilon_{old}-\varepsilon_p\right)}-K_0^2\right)\left(\frac{\varepsilon-\varepsilon_p}{\varepsilon_{old}-\varepsilon_p}\right)^2\right) \tag{4}$$

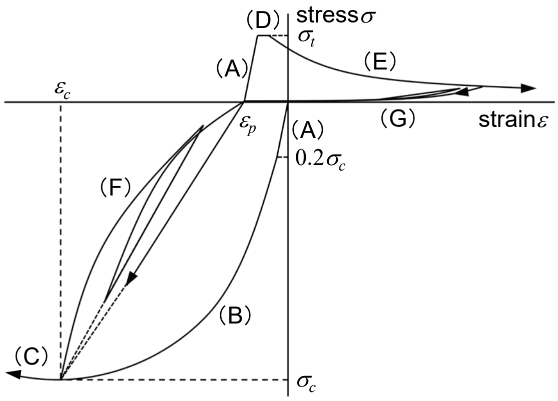
$$K_0 = \exp\left(-0.73\frac{\varepsilon_{\max}}{\varepsilon_c}\left(1-\exp\left(-1.25\frac{\varepsilon_{\max}}{\varepsilon_c}\right)\right)\right) \tag{5}$$

$$\varepsilon_p = \left(\frac{\varepsilon_{\max}}{\varepsilon_c}-\frac{20}{7}\left(1-\exp\left(-0.35\frac{\varepsilon_{\max}}{\varepsilon_c}\right)\right)\right)\varepsilon_c \tag{6}$$

ε :	Strain	ε_c :	Compressive strain
σ :	Stress	ε_{max} :	Maximum strain
σ_c :	Compressive strength	k :	Deterioration factor
E :	Elastic stiffness	old :	Before 1 step



(a) Steel



(b) Concrete

Fig. 2 – Hysteresis model for the spring in MS element

3. Simulation analysis of cyclic loading test of RHS column

3.1 Overview of test

For the RHS column, a simulation analysis of the stub column test and the cyclic loading test [8] with a constant axial force (30% of the yield axial force) was conducted. The specimen is a 787.5 mm long cantilever. In this test, local buckling was seen at an angle of 0.023 rad and cracks of RHS column were detected at an angle of 0.057 rad.

3.2 Properties of MS model

The spring layout of the MS element is shown in Fig. 4. In order to determine the skeleton curve applied to the spring, a simulation analysis of a stub column test was performed. As a result, the strength and second stiffness were determined as the values that fit the relationship between the axial stress and the axial strain as shown in Fig. 5. The properties of MS model are shown in Table 1. In the analysis of cyclic loading test (next section), the ratio of the MS element length to the member length was set to 0.05 in accordance with Ref.[5].

Fig. 3 – Result of cyclic loading test of RHS-column

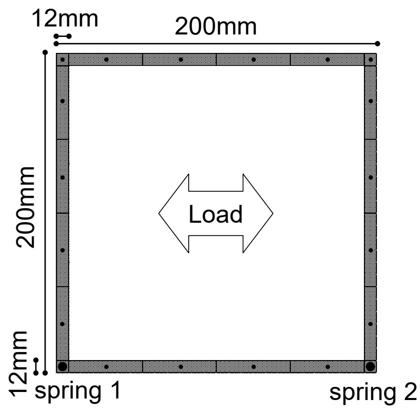


Fig. 4 – Spring layout of MS element
(RHS-column test, 20 steel springs)

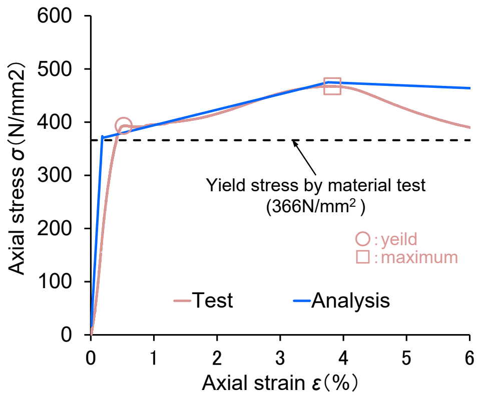


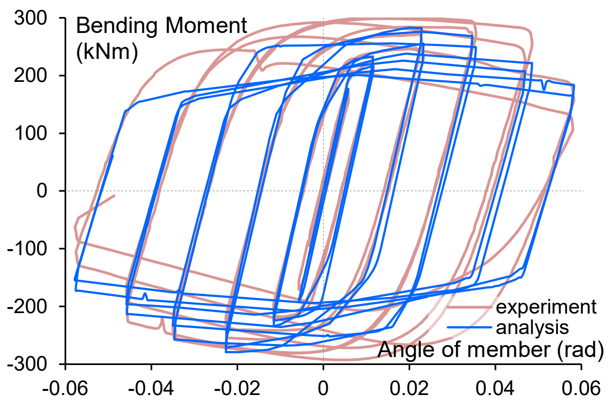
Fig. 5 – Relationship between axial stress and axial strain of stub column test

Table 1– Properties of MS model (RHS-column test)

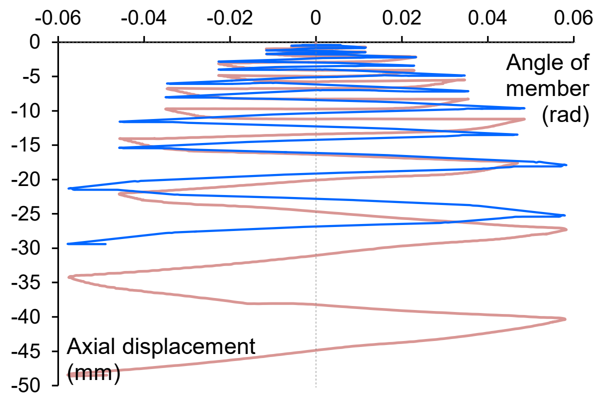
	Parameter	Value	Comment
RHS	Yield stress	366 N/mm^2	From material test
	Strength (local buckling)	475.8 N/mm^2	From stub column test
	Second stiffness	2870 N/mm^2	ditto
Other	Ratio of MS element	0.05	Ishida (2014) [5]

3.3 Simulation analysis

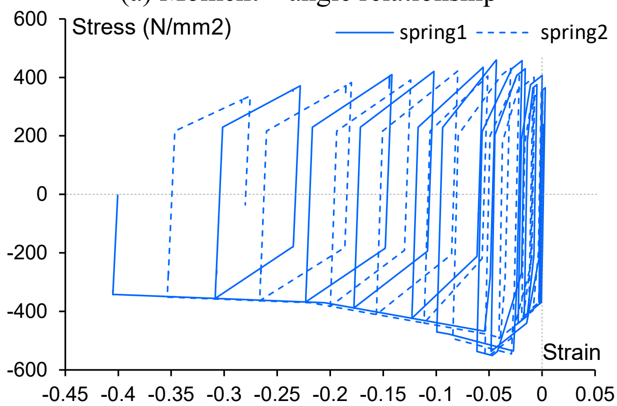
The results of simulation analysis are shown in Fig. 6. The analysis was able to simulate the strength and deterioration behavior. Therefore, the validity of this MS model was shown. However, the evaluation result of the axial displacement was slightly smaller. Seeing the stress-strain relationship of spring, the deterioration was very large.



(a) Moment – angle relationship



(b) Axial displacement – angle relationship



(c) Stress – strain relationship (spring in MS element)

Fig. 6 – Comparison between experiment and analysis

4. Simulation analysis of cyclic loading test of CFT column

4.1 Overview of test

For the CFT column, a simulation analysis of the cyclic loading test [8] with a constant axial force (30% of the yield axial force) was conducted. The specimen is a 2240 mm long column fixed at both ends. In this cyclic loading test, local buckling was seen at an angle of 0.02 rad and cracks of RHS column were detected at an angle of 0.033 rad.

4.2 Properties of MS model

The spring layout of the MS element is shown in Fig. 7. The list of properties of MS model is shown in Table 2. The material properties of RHS was obtained from the paper [7] [8] which analyzed the statistical data of the stub column test. The material properties of concrete was based on the results of material experiments, and the deterioration factor *k* and compressive strain were determined by parametric study. However, the compressive strain had little impact on the analysis result. The ratio of the MS element length to the member length was set to 0.125 with reference to the damaged region observed in the experiment.

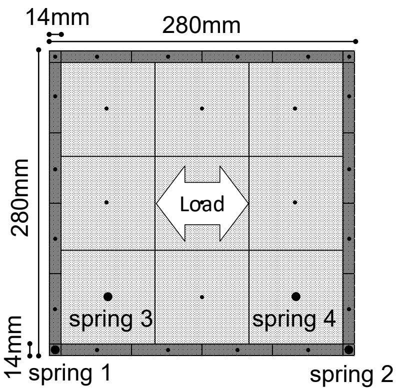


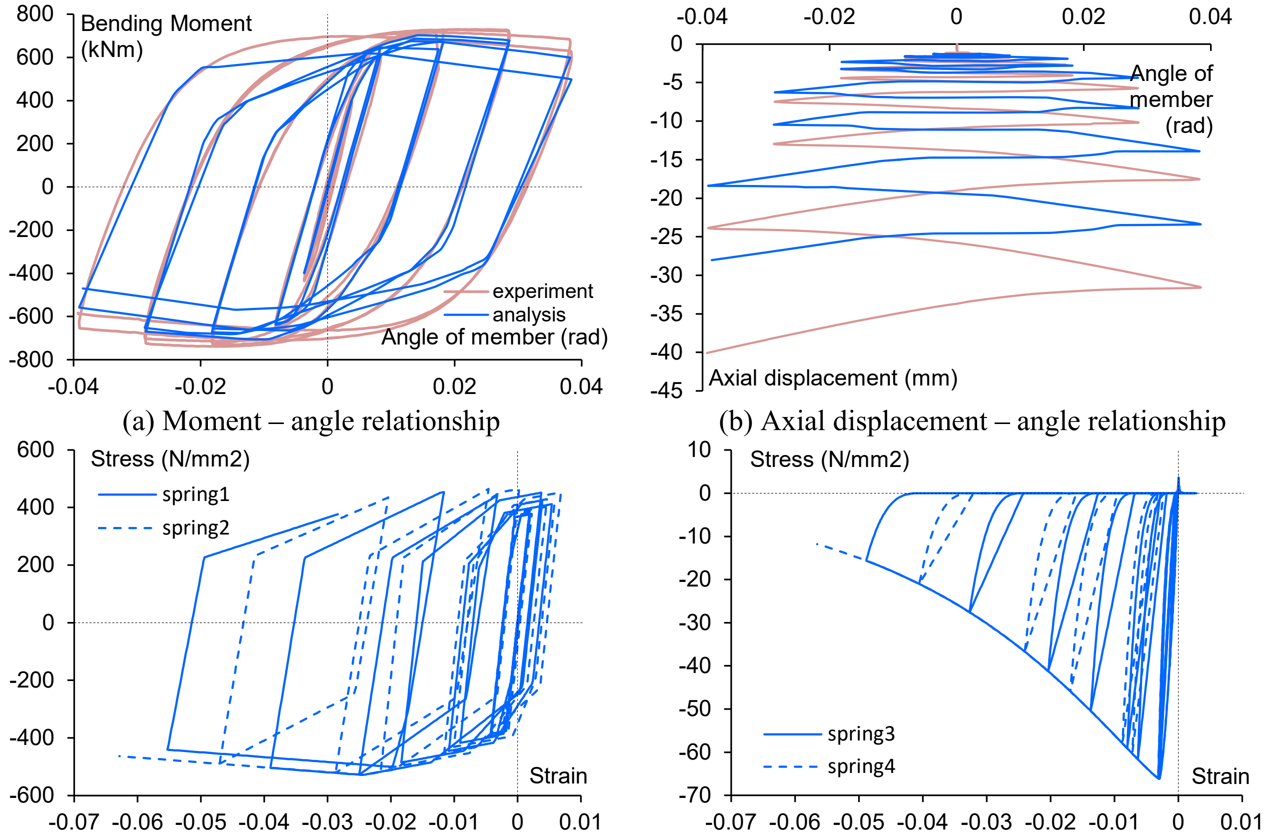
Fig. 7 – Spring layout of MS element (CFT-column test, 20 steel springs and 9 concrete springs)

Table 2– Properties of MS model (CFT-column test)

	Parameter	Value	Comment
RHS	Yield stress	376 N/mm^2	From material testing
	Strength (local buckling)	475.8 N/mm^2	Kato (1987) [10]
	Second stiffness	0.020	Yamada (1993) [11]
Con-crete	Young modulus	32770 N/mm^2	From material testing
	Compression strength	66.2 N/mm^2	ditto
	Deterioration factor *k*	50	Parametric study
	Compressive strain	0.003	ditto
Other	Ratio of MS element	0.125 (=width of RHS)	Large strain region

4.3 Simulation analysis

The results of simulation analysis are shown in Fig. 8. The analysis was able to simulate the strength and deterioration behavior. Therefore, the validity of the proposed concrete model was shown. However, the evaluation result of the axial displacement was slightly smaller. Seeing the stress-strain relationship of spring, the deterioration of concrete was very large.



(a) Moment – angle relationship

(b) Axial displacement – angle relationship

(c) Stress – strain relationship (spring of RHS in MS element)

(d) Stress – strain relationship (spring of concrete in MS element)

Fig. 8 – Comparison between experiment and analysis

5. Numerical analysis of a three-dimensional frame model

5.1 Modelling of building

The analysis target is a real building in Japan with 9 stories high above the ground. This building is a moment-resisting frame which has CFT columns and I-section steel beams. Plan and typical member dimension on the 1st floor is shown in Fig. 9.

The analysis model was created as follows. A three-dimensional frame model was used for the analysis. Beams were modelled by beam elements, and connection panels were modelled by pure shear panel models. Columns were modelled by MS model. The stress at local buckling of the RHS was set in accordance with Ref.[10], and the plastic deformation capacity was set in accordance with Ref.[11]. The Deterioration factor k of the concrete was set to 50. Beams were modelled by a multi-component parallel model [1]. Plastic stiffness was assumed to be 3% of the elastic stiffness in the bending moment / rotation angle relationship. Hysteresis model for the beam ends was modelled by a deterioration type in consideration of the lower-flange fracture. C, which represents the deformation capacity of the fracture at the flange under the beam end, was set to $C = 5.6$, which is equivalent to the detail with no weld access hole. The P-Δ effect was considered with the additional shear force corresponding to the story drift. The stiffness proportional damping coefficient was set to 2% of critical damping in the first mode response.

5.2 Result of analysis

5.2.1 Modal analysis

The modal properties are shown in Table 3. The natural period of the 1st mode is 1.57 seconds, and vibrating to X-direction. The natural period of the 2nd mode is 1.56 seconds, and vibrating to Y-direction.

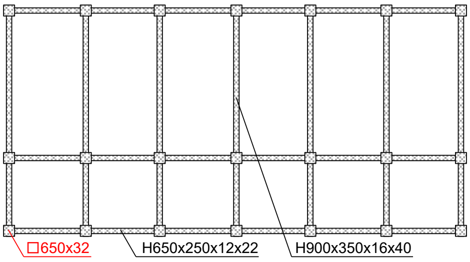
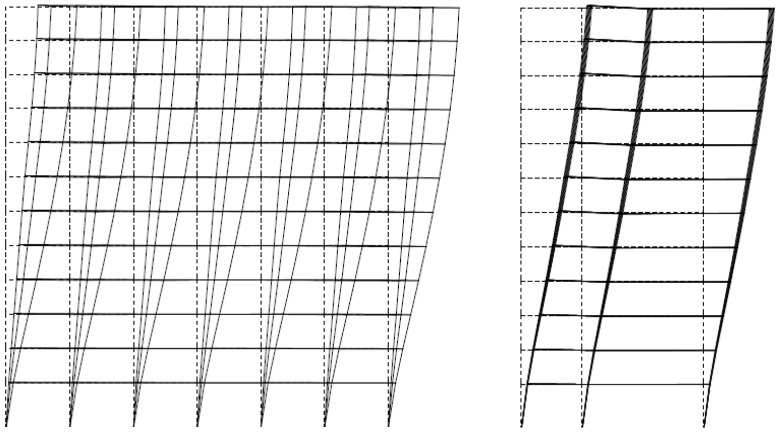


Fig. 9 – Plan and typical member dimension on the 1st floor

Table 3– Modal properties

Mode	Period (s)	Comment
1	1.57	X direction
2	1.56	Y direction

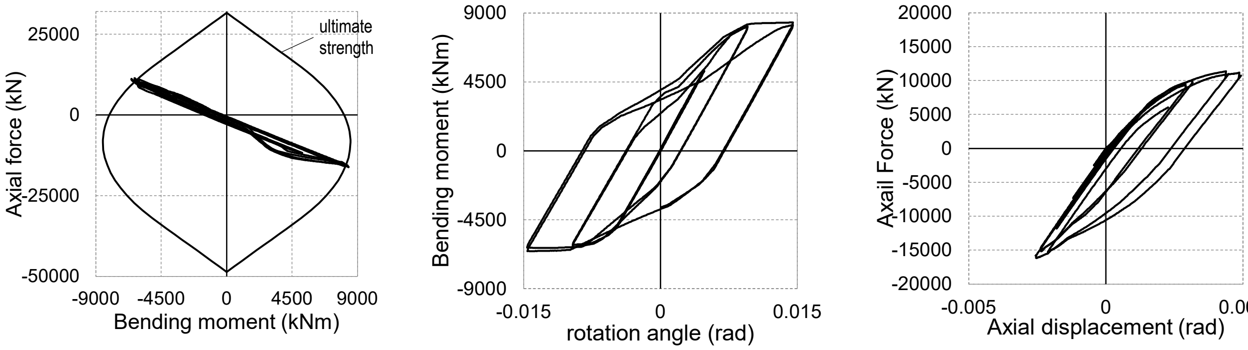


(a) X direction (b) Y direction

Fig. 10 – Mode shapes in both principal directions

5.2.2 Static cyclic loading analysis

In accordance with the mode shape, static cyclic displacement control loading was performed so that the story drift of the 1st floor were 1/400 rad, 1/200 rad, 1/100 rad, and 1/66 rad to the Y direction. The response of the corner column (see red one in Fig. 9) is shown in Fig. 11. MS model is useful because it corresponded well to the strength calculated by Ref. [12] as shown in Fig. 11 (a). And the yield stress differ between positive and negative, because of the varying axial force, as shown in Fig. 11 (b).



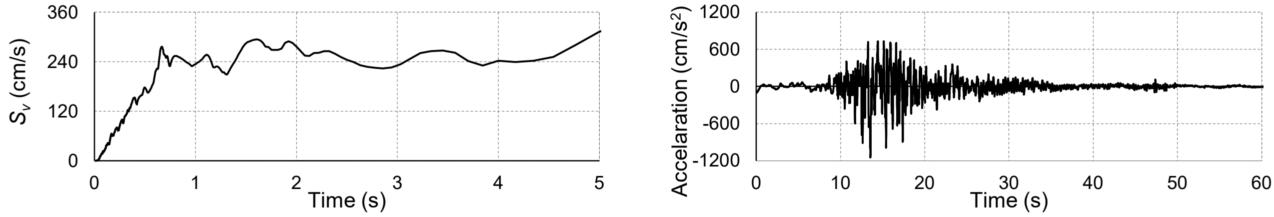
(a) Axial force – moment relationship (b) Moment – angle relationship (c) Axial force – axial displacement relationship

Fig. 11 – Response of column (red one in Fig. 9)

5.2.3 Time history analysis

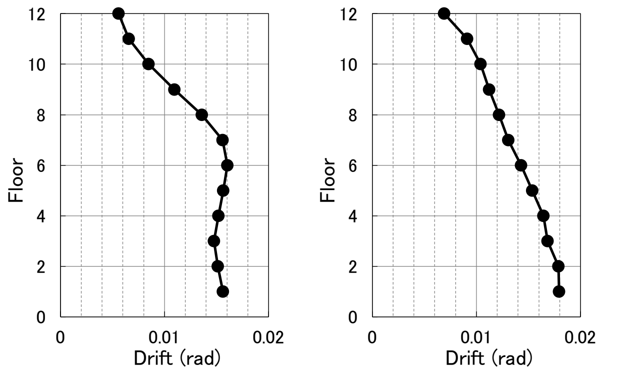
The input ground motion is 3 times as large as the Japanese design ground motion (return period is about 500 years) as shown in Fig. 12. 3 times was used to ensure the concrete reaches the compression strength. In addition, the ground motion was inputted in a 45-degree direction to increase the damage to the columns.

The responses of the building are shown in Fig. 13 and Fig. 14, which have a very large story drift. The column (see red one in Fig. 9) response is shown in Fig. 15. The MS model can take into account the combination of varying axial force and bi-directional bending, and can evaluate complex responses, so it corresponded well to the strength (calculated by Ref. [12]) as shown in Fig. 15 (a).



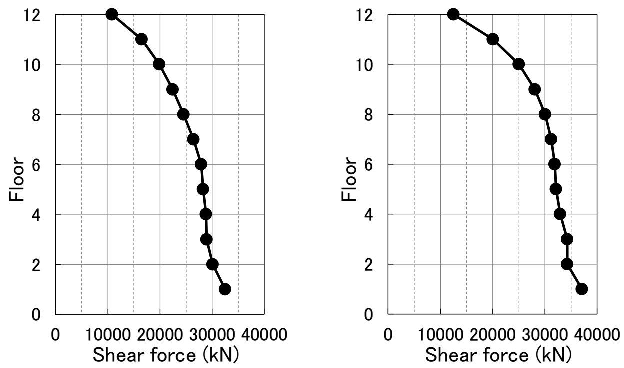
(a) Spectral Velocity (5% damping) (b) Time history

Fig. 12 – Ground motion (3 times of Japanese design ground motion)



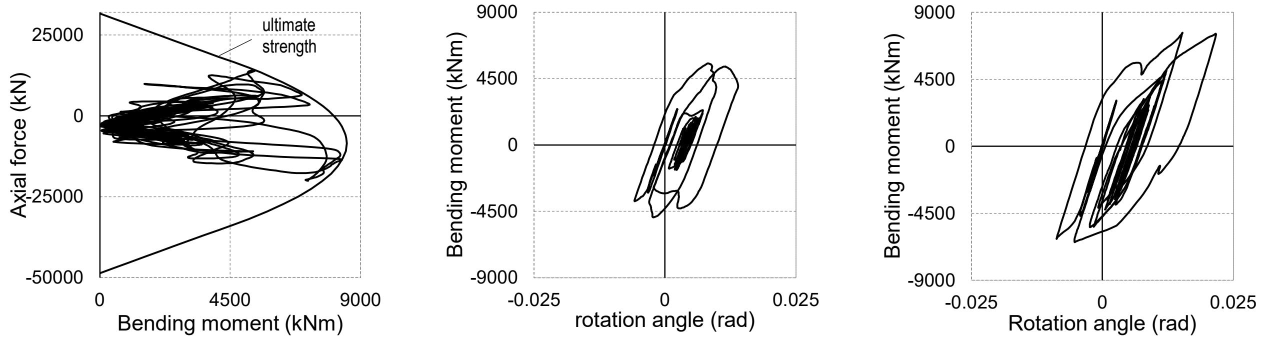
(a) X direction (b) Y direction

Fig. 13 – Story drift



(a) X direction (b) Y direction

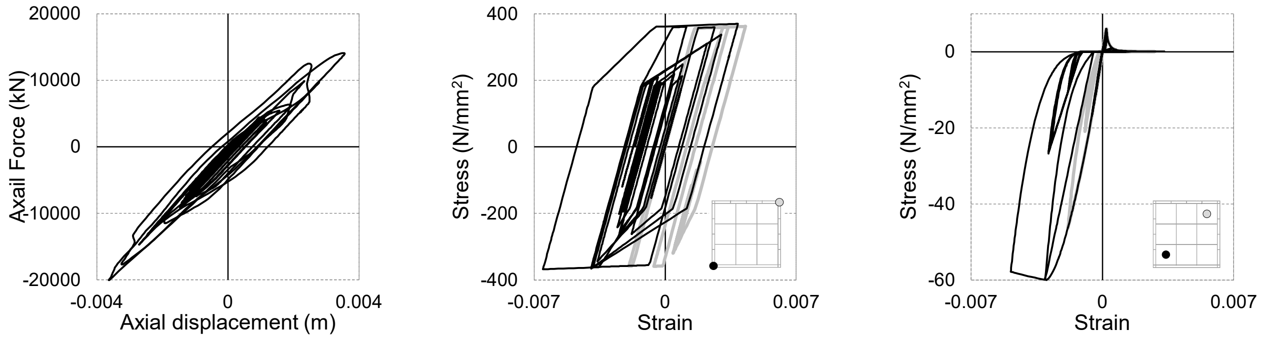
Fig. 14 – Shear force



(a) Axial force – moment (45 degree direction) relationship

(b) Moment – angle relationship (X direction)

(c) Moment – angle relationship (Y direction)



(d) Axial force – axial displacement relationship

(e) Stress – strain relationship (Two corner steel spring of MS model)

(f) Stress – strain relationship (Two corner concrete spring of MS model)

Fig. 15 – Dynamic response of column (red one in Fig. 9)

6. Conclusions

In order to simulate the deterioration behavior of a column by numerical analysis using the multi-spring (MS) model, nonlinear hysteresis rule that considers the deterioration behavior was applied to the spring element of the MS model. Using MS model, simulation analyses of a RHS column and a CFT column were performed, and it was shown that the deterioration behavior of the columns was able to be simulated by considering local buckling and concrete deterioration. In addition, the three-dimensional frame analysis was performed using the developed analysis program with MS model, and the behavior of the building was confirmed. In the future, in order to clarify the collapse behavior of buildings, seismic response analysis for larger earthquakes will be conducted.

7. References

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