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Seismic damage evaluation of existing low-rise RC buildings Suffered by the 2011 off the Pacific Coast of Tohoku Earthquake

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ABSTRACT

The 2011 off the Pacific coast of Tohoku Earthquake registered Seismic Intensity of upper 5 to 7 on the Japanese seven-stage seismic scale in Japan. Some of low-rise reinforced concrete (RC) buildings in Ibaraki Prefecture of Japan were severely damaged. They had old seismic performance criteria and were not retrofitted. The objective of this study is to investigate the structure performance and the damage levels of the existing low-rise RC buildings damaged by the earthquake, and to evaluate structural damage on the RC buildings more accurately by clarifying seismic behavior based on pushover analysis and seismic response analysis. The damage levels surveyed on the field work and quantified with residual ratio of seismic performance. Seismic performance is discussed by the seismic capacity index of structure in standard for seismic evaluation for existing RC buildings, results of pushover analysis and time history response analysis.

By comparing observed damage levels with results of seismic evaluation method, it was confirmed that the seismic performance was influenced by concrete strength decrease due to shoddy workmanship and concrete block wall. Shear strength of RC members with shoddy workmanship reduced significantly due to concrete strength decrease, so it is important to consider the defects such as shoddy workmanship. RC columns with spandrel wall of concrete-block are prone to shear failure, it was confirmed that they are dangerous and have poor seismic performance. Damage levels of RC columns on pushover analysis results show a good agreement with actual damage of the existing low-rise RC buildings. Based on these investigations, seismic performance and seismic behavior on the damaged RC buildings was estimated by time history response analysis.

1. INTRODUCTION

The 2011 off the Pacific coast of Tohoku Earthquake at 14:46 on March 11th, 2011(Mw 9.0) registered the maximum 7 on the Japanese seven-stage seismic scale at Tsukidate in Miyagi Prefecture. In Ibaraki Prefecture, the earthquake registered upper 5 to upper 6 and some of low-rise RC buildings had serious damage. These buildings had been built based on past seismic standard and had not been retrofitted.

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This study presents the investigation of selected two buildings (A) and (B) which were severely damaged in Ibaraki Prefecture. The damage levels measured on field work and quantified with residual ratio of seismic performance. Seismic performance is discussed by seismic capacity index of structure in Japanese standard for existing RC buildings. Pushover analysis and time response analysis are also carried out. By comparing the results, this study aims to clarify the serious damage of the buildings and confirm those applicability to the actual damage.

2. BUILDING (A)

The front view of the building (A) is shown in Fig.1 and the size and reinforcement of typical columns are shown in Table.1. First floor plan is shown in Fig.2 and framing elevation of line A is shown in Fig.3. The building (A) is a 3-story building constructed in 1965 and it has a 1-story building partially. It has 11 spans in longitudinal direction and 5 spans in 1st story and 2 spans in other story in lateral direction. It has an atrium in 1st story and an assembly hall which floor height is higher in 3rd story. The height of the 1st floor is 4.05m, 2nd is 3.60m, 3rd is 3.10m and 7.00m. The building is formed of rigid frame with the RC walls. Strong ground motion which had observed at distance of 150m in southern direction from this building and acceleration response spectrum are shown Fig.4. The maximum acceleration of ground motion is 967.4 cm/s^2 in north-south direction and 595.8 cm/s^2 in east-west direction. Acceleration response spectrum is distinguished in period 0.3 to 0.4 and 0.6 to 0.7 and value in north-south direction is higher.

Fig.1 View of building (A)

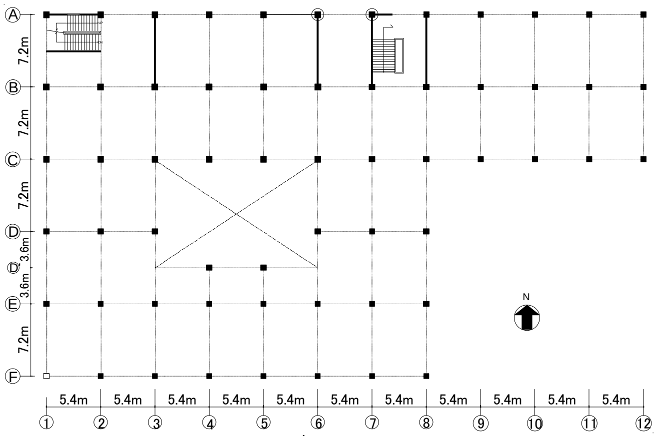


Fig.2 1st floor plan

Table.1 Typical column

Column	A-6	A-7
Dimensions (mm)	550×600	550×550
Main Bar	10-22φ	10-22φ
Hoop	9φ@250	

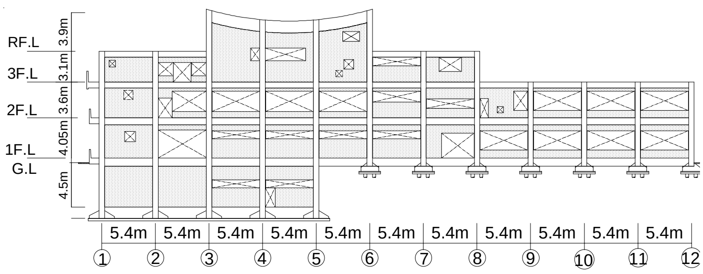


Fig.3 Frame elevation of line A

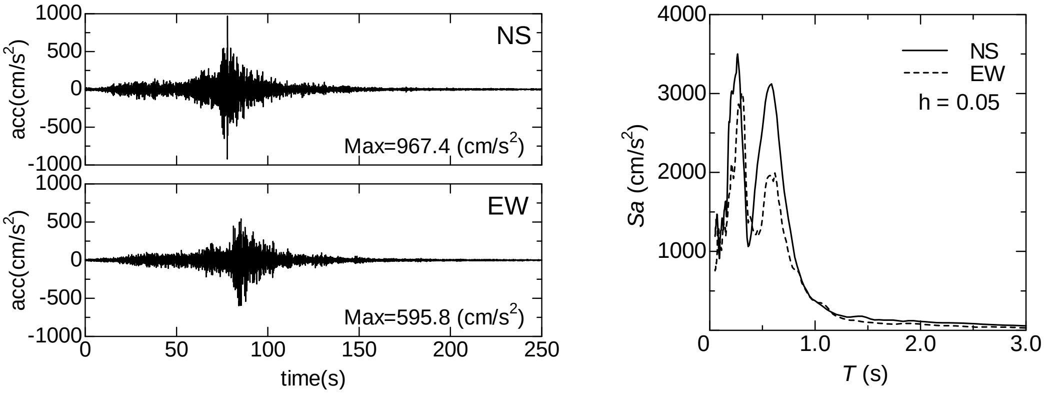


Fig.4 Strong ground motion and acceleration response spectrum

2.1 Observed damage

1st floor plan and damage levels in columns and walls are shown in Fig.5. The damage level of each column and wall are marked with Roman numbers. These damage evaluation are quantified based on "Guideline for Post-earthquake Damage Evaluation and Rehabilitation"[1)] with residual ratio of seismic performance. The building had many columns with severe shear failure in longitudinal direction. These shear failure of extremely short columns and walls are shown in Fig.6. Residual ratio of seismic performance are shown in Table.2. The longitudinal direction of 1st floor had most severe damage.

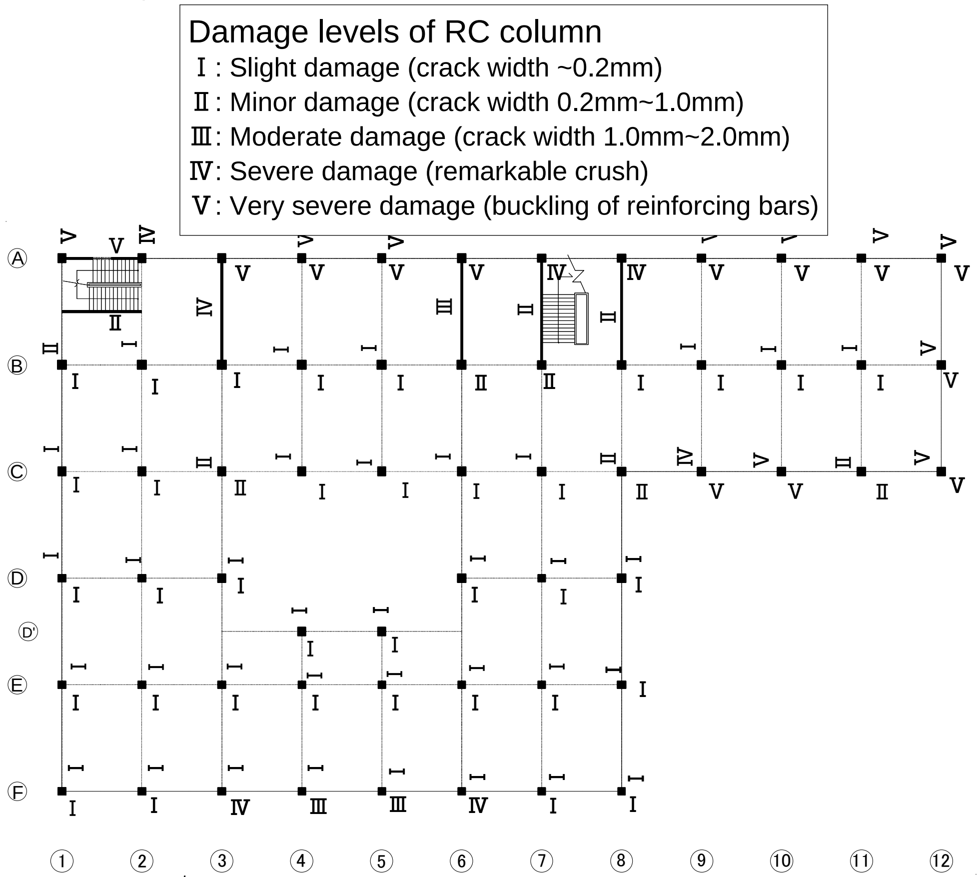


Fig.5 1st floor plan and damage levels in columns and walls

Shear failure of extremely short column and wall Axial failure Cracks of the wall

Fig.6 Damage of columns and walls

Table.2 Residual ratio of seismic performance

story	longitudinal		lateral	
	residual ratio *R* (%)	damage level	residual ratio *R* (%)	damage level
3	69.7	moderate	51.4	severe
2	69.4	moderate	80.6	minor
1	57.4	severe	58.6	severe

2.2 Seismic evaluation

Evaluation of Seismic capacity is based on the Japanese standard for seismic evaluation of existing RC buildings[2)]. This method evaluate with the index of structural seismic performance (I_S). This index calculated by Eq.1.

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

E_0 is a basic structural index which calculated by the evaluation of strength and ductility. Strength of building is evaluated by the index of strength (C), this index means base shear coefficient. Ductility is estimated by the index of ductility (F). The index F varies from 0.8 to 3.2, F = 0.8 means that the structural member is extremely brittle and F = 3.2 means that the structural member is most ductile. S_D is a reduction coefficient calculated by the shape of building, and T is also a reduction coefficient calculated by aged deterioration. C_{TU} is the index of cumulative index (C) evaluated at the ultimate limit state. Dimensions of members and reinforcement are set on the basis of field work and specifications. Concrete strength is set on material testing (see Fig.7 and Table.3). There are no information of the yield strength, it is set on 240N/mm^2 refer the standard for seismic evaluation. Space of hoops was planned 250mm on specification, but it seems to have few dispersion and the ends of bars are bent by angle of 90 degree. Therefore, the space of hoops is set on multiply 250mm by 4/3 based on the specification in Ibaraki Prefecture. The results of seismic capacity evaluation are shown in Table.4 and the relation between C_{TU} and F are shown in Fig.8. Failure types and F-indexes of structural members estimated by seismic evaluation and actual damage of 1st floor in longitudinal direction are shown Fig.9. Building (A) was built over 40years ago and it is assumed that it has degradation over time. However, this study aims to reveal the cause of structural damage, T is set on 1.00. Seismic capacity is evaluated by comparing (I_s) with seismic demand index of structure I_{SO} (=0.6), if the index of structural seismic performance (I_S) is higher than I_{SO}, the building is assessed provided to prevent major structural damage.

Fig.7 Sampling test piece

Table.3 Result of compression test of concrete

story	Concrete strength		
	average $\bar{X}$ (N/mm^2)	standard deviation σ (N/mm^2)	$\bar{X}-\sigma/2$ (N/mm^2)
3	24.5	5.98	21.5
2	19.1	5.84	16.2
1	15.7	3.64	13.9

Table.4 Results of seismic capacity evaluation

direction	story	F	C	E_0	S_D	T	I_S	$C_{TU} \cdot S_D$
X	3	1.00	2.11	1.41	0.93	1.00	1.31	1.30
	2	0.80	0.46	0.30	0.74	1.00	0.22	0.28
	1	0.80	0.33	0.26	0.93	1.00	0.24	0.30
Y	3	2.00	0.81	1.33	0.93	1.00	1.24	0.50
	2	1.00	0.66	0.53	0.74	1.00	0.39	0.39
	1	0.80	0.42	0.34	0.93	1.00	0.32	0.39

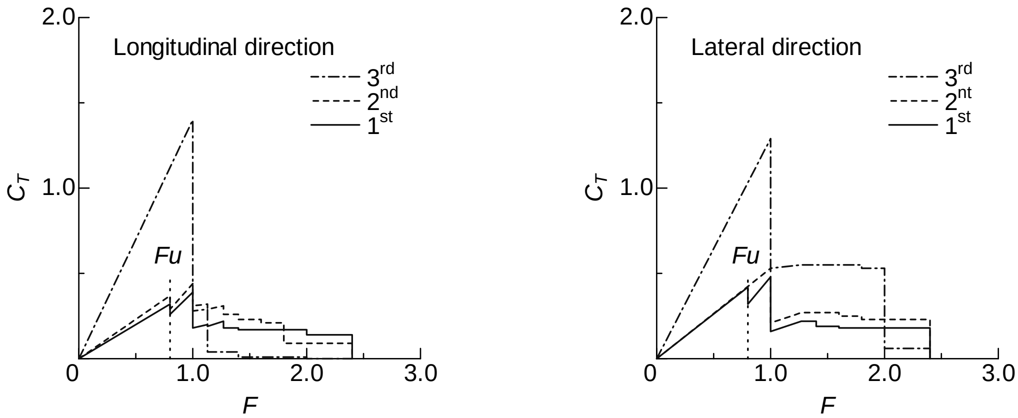


Fig.8 Evaluation of strength and ductility ($C_T - F$)

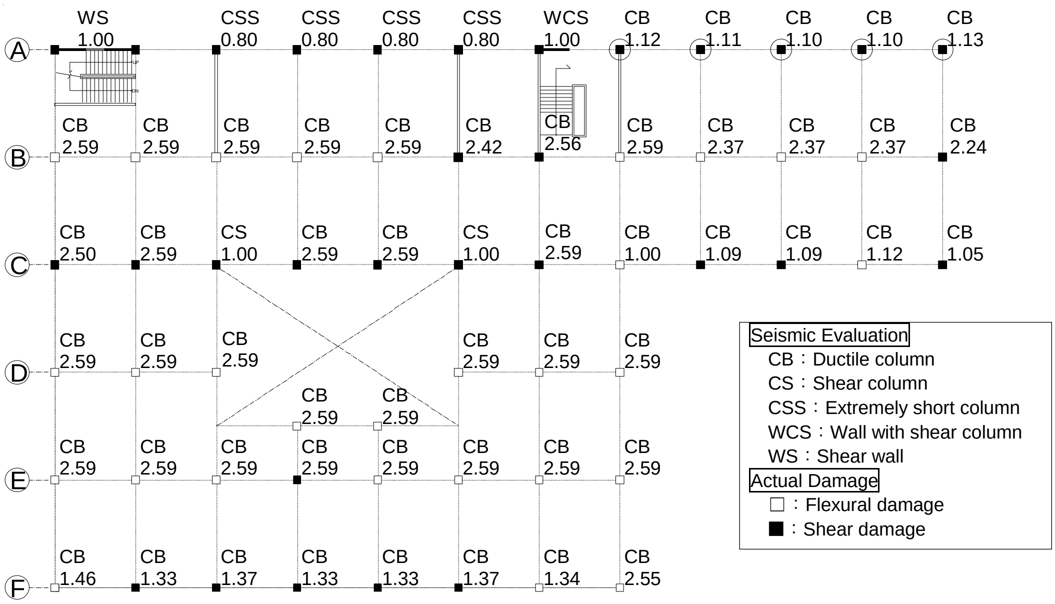


Fig.9 Failure types and F-indexes in the case of concrete strength 13.9N/mm^2

I_S of the longitudinal direction are especially low in 1st and 2nd floor. I_S in the 3rd floor is higher, it is cause of the concrete strength and the effect of RC wall. In 1st floor, I_S is lower than 0.3 and it almost agrees to the actual damage. F is estimated as 0.8 in 1st floor due to existing of the second-class prime elements which cannot resist axial load by serious damage and may cause to collapse of building. 1st floor has a number of shear columns and extremely short column. By comparison between failure types of seismic evaluation and actual damage, many estimated failure types of column differ from actual situation. For instance, several columns on the line A and F in Fig.9 showed shear failures, on the other hand, they are classified the ductile column. By the field survey and material test, some of shoddy workmanship on the line A are confirmed as shown Fig.10.

In consideration of hidden shoddy workmanship on the 1st story, the seismic capacity is re-evaluated. By decreasing concrete strength from 13.5N/mm^2 to 10.0N/mm^2 by 0.5N/mm^2, failure types of seismic evaluation and the actual damage are compared and compatibility rate is estimated. The rate is expressed by the number of members that fit the failure type divided by all of structural members. Concrete strength of 2nd and 3rd story are set on the value of the result of compression test. The relation between compatibility rate and concrete strength are shown in Fig.11. Compatibility rate of failure type increases as concrete strength decreases to 11.5 N/mm^2, the building has many shear failure columns on the east side. When concrete strength is 11.5N/mm^2, compatibility rate is the highest, and most failure type of columns on F line are shear failure type. Many of columns which failure types don't agree to actual damage are located on the C line. It seems that because some columns on the A line are collapsed in shear at first, torsion of structure and fluctuation of axial load affect failure types of columns on the C line.

Result of seismic evaluation in the case of concrete strength 11.5N/mm^2 is shown in Table.5 and the relation between C_T and F is shown in Fig.12. Failure types and F-indexes of the 1st floor on longitudinal direction are shown in Fig.13. Because compatibility between result of evaluation and actual damage improves by decrease of concrete strength, the influence of material degradation, loss of cross section, degradation of bonding by shoddy workmanship can be expressed by decreasing concrete compressive strength. From result of seismic evaluation when concrete strength is 11.5N/mm^2, reduction coefficient of concrete strength due to shoddy workmanship is estimated as 0.83. Considering some of hidden shear failure be caused by shoddy workmanship, seismic evaluation should be done by concrete strength reflected in internal damage and construction condition of members.

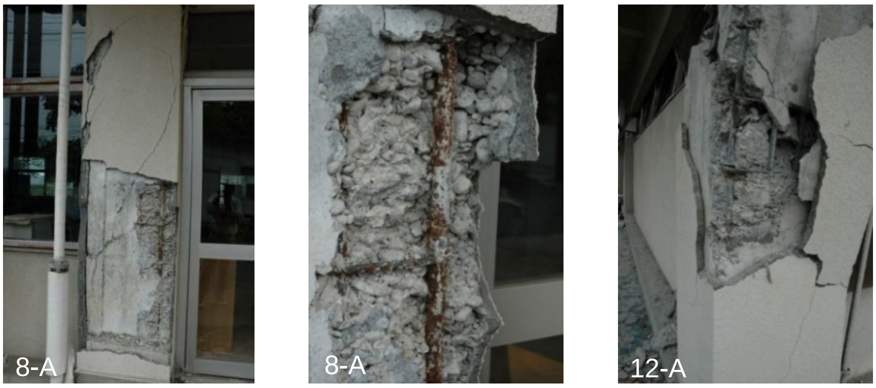


Fig10. Shoddy workmanship

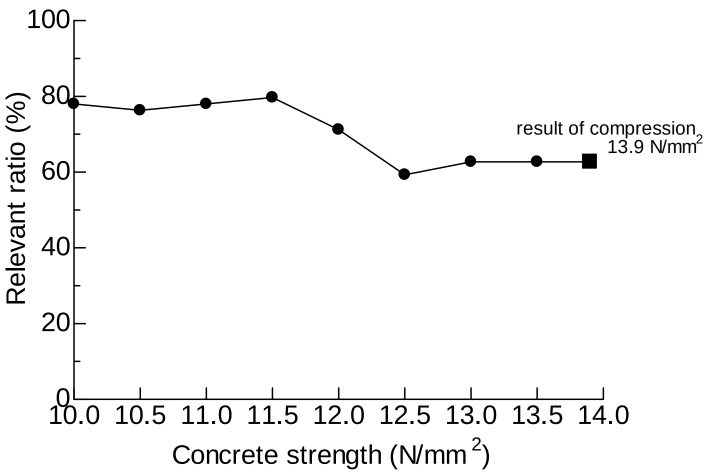


Fig.11 Relation between compatibility rate and concrete strength

Table.5 Result of seismic evaluation in the case of concrete strength 11.5N/mm^2

direction	story	F	C	E_0	S_D	T	I_s	$C_{TU} \cdot S_D$
X	3	1.00	1.69	1.13	0.93	1.00	1.05	1.04
	2	0.80	0.40	0.25	0.74	1.00	0.19	0.23
	1	0.80	0.29	0.23	0.93	1.00	0.21	0.27
Y	3	1.00	1.68	1.12	0.93	1.00	1.04	1.04
	2	1.00	0.55	0.44	0.74	1.00	0.33	0.33
	1	0.80	0.36	0.29	0.93	1.00	0.27	0.33

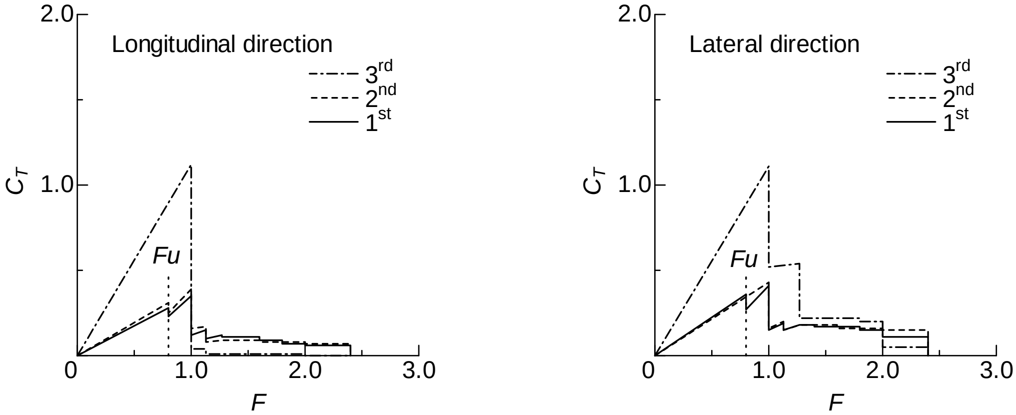


Fig.12 Evaluation of strength and ductility (C_T – F)

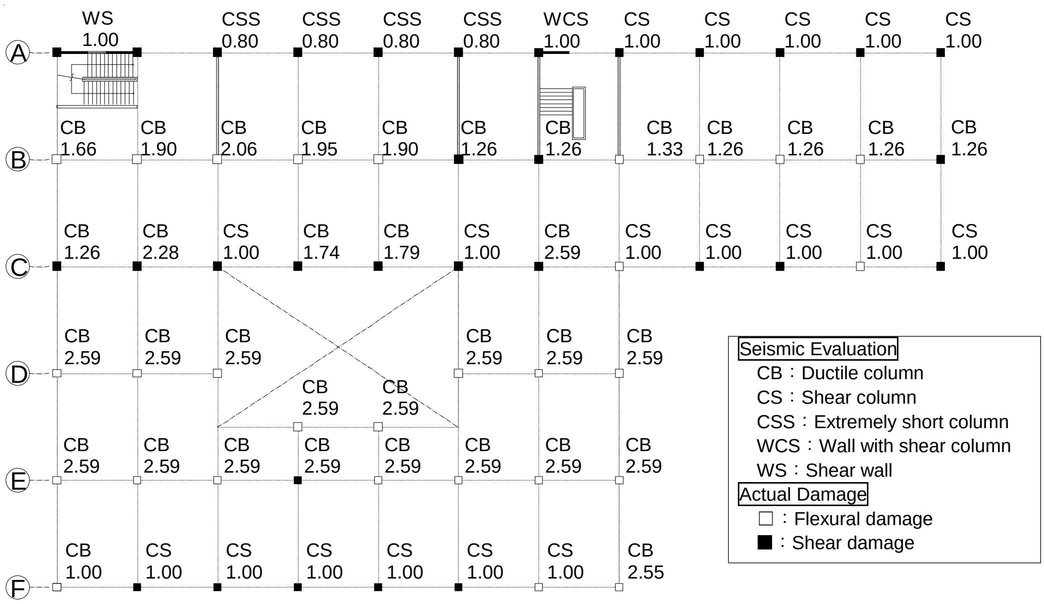


Fig.13 Failure types and F-indexes in the case of concrete strength 11.5N/mm^2

2.3 Pushover analysis

Pushover analysis is carried out on the longitudinal frames which damaged seriously by using elasto-plasticity analysis program called SNAP. Columns and beams are idealized by linear axial spring, nonlinear rotational springs at their ends, and shear spring in the middle of the member. Restoring force characteristics of rotational and shear springs are expressed by tri-linear model as shown in Fig.14. The hysteresis models are Takeda-model. As shear spring model, the model considering decline of shear force after shear failure is used. Stiffness of these springs decreases at cracking point and ultimate strength. Cracking moment is estimated by using AIJ standard[3]. Yield moment and shear capacity is estimated by using standard for seismic evaluation of existing reinforced concrete buildings. Rotation angle of member at the ultimate flexural strength of columns is assumed to be 0.01 rad. Drift angle at the ultimate deformation after shear failure is assumed to be 0.1 rad. Beam-column joint is assumed to be rigid. Pushover analysis is carried out until at least one column cannot resist shear force after shear failure. Result of compression test and decreased strength is used for concrete strength. Relations between story drift and story shear force are shown in Fig.15. Failure types of members on 1st story are shown in Fig16. In the case of concrete strength based on compression test, failure types of one wall and one column are shear failure and many other columns have flexural yielding. As a result, decline of story shear coefficient due to shear failure is not observed, and it is assumed that the each stories have sufficient ductile capacity. On the other hand, in the case of decreased concrete strength, many columns collapse in shear on the A and F line. The story shear coefficient is seriously decreased attendant on occurrence of shear failure. Failure types of columns match the actual damage.

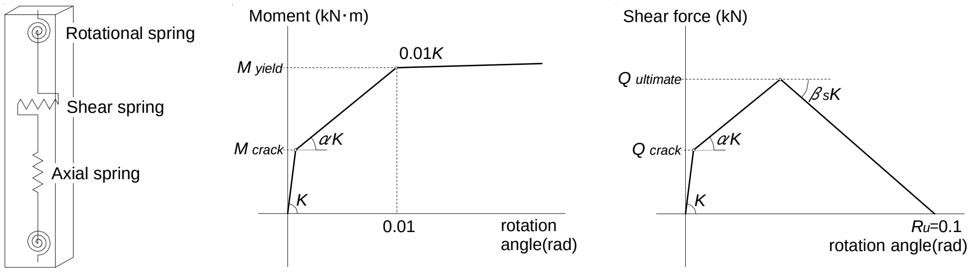


Fig.14 Restoring force characteristics of rotational and shear spring

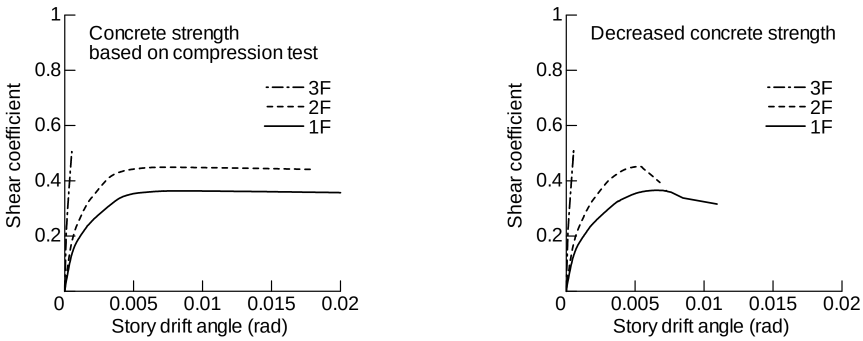
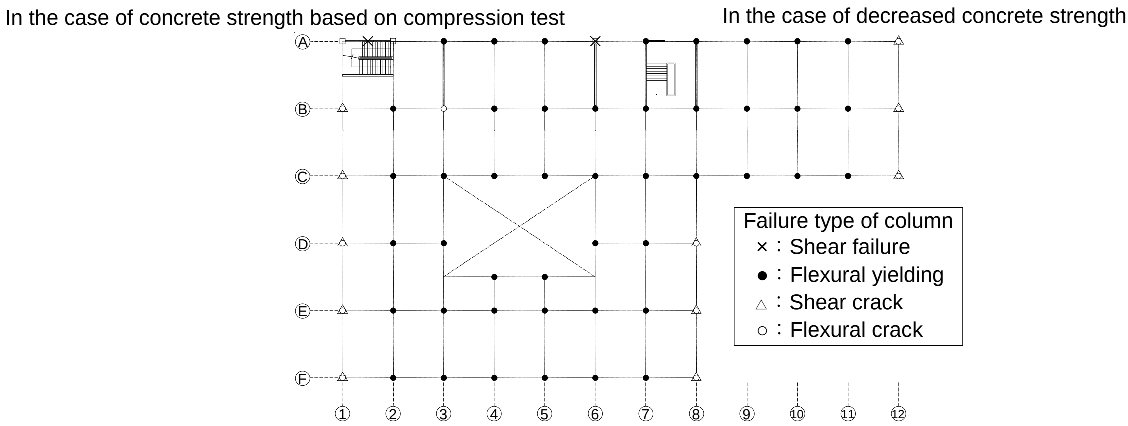


Fig.15 Relations between story drift and story shear coefficient



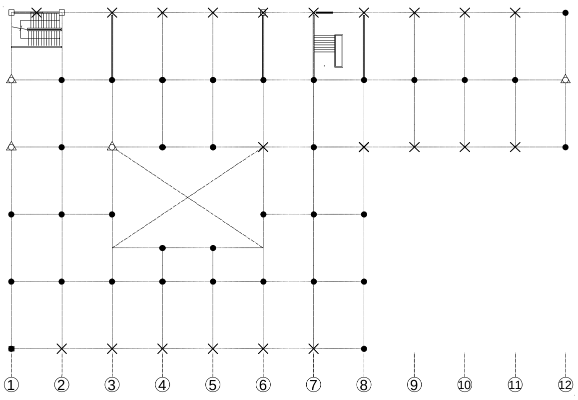


Fig.16 Failure types of members

2.4 Time history response analysis

Time history response analysis is carried out for MDF system using shear springs. The skeleton curves of each stories are fixed by the result of pushover analysis and hysteresis model as shown in Fig.17. Several parameters are set as shown in the figure referring to the result of previous experiment on column. Seismic wave used in the analysis is actual ground motion in east- west direction (Fig.4). Relations between story shear coefficients and story drift on the 1st story are shown in Fig.18. Time history responses of story deformation are shown in Fig.19. Maximum of story shear coefficient in the both cases is 0.4 when story drift is 1/250 rad. In the case of decreased concrete strength, story shear coefficient is seriously declined cause of existing many members with shear failure. As a result, shear capacity is assumed be lost and 1st story collapse at 85 sec registered maximum acceleration. In the case of concrete strength based on compression test, maximum story drift is about 71mm. Analysis result with decreased concrete strength basically accord with actual damage.

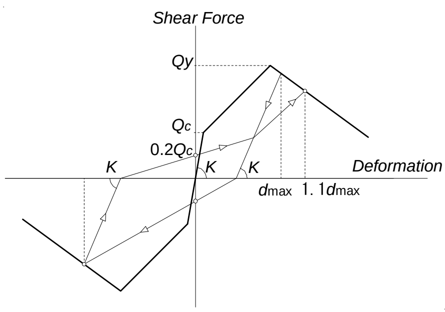


Fig17. Hysteresis model

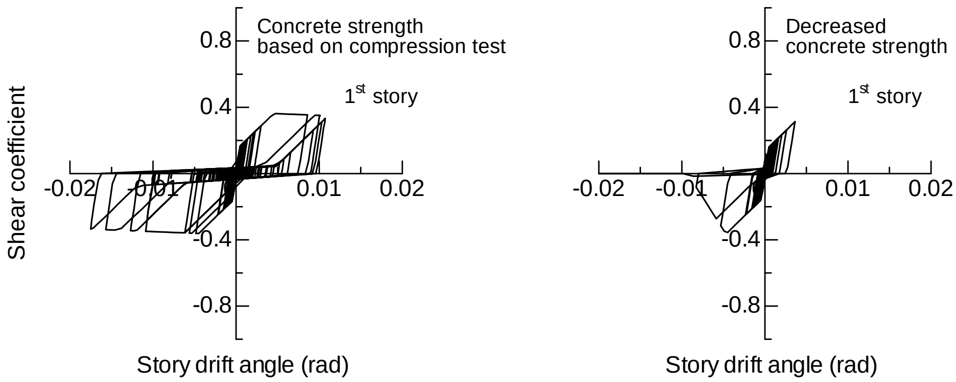


Fig.18 Relations between shear coefficients and story drift angle

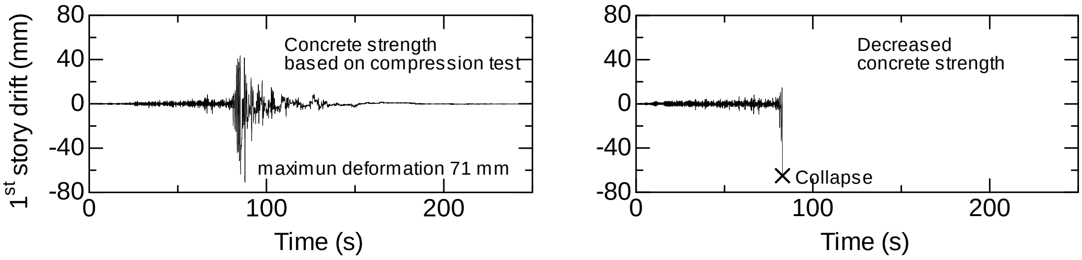


Fig.19 Time history 1st story drift

3. BUILDING (B)

Building (B) has two story as shown in Fig.20. The dimension and reinforcement of typical columns are shown in Table.6. The 1st floor plan is shown in Fig.21 and framing elevation of the A line is shown in Fig.22. The reinforcement arrangement and concrete strength used in several analysis is based on the field work and results of compression test as shown in Fig.23 and Table.7. Because there is no design information when the building was built, it is confirmed that the building has been built in 1964 approximately by interviews. It has 3 span on longitudinal direction and 1 span on lateral direction. The building is formed of rigid frame with the concrete-block walls. The height of 1st floor is 4.50m, 2nd is 3.40m. Strong ground motion which had observed at distance of 250m from building (B) and acceleration response spectrum are shown in Fig.24. The maximum acceleration are 524.7cm/s^2 in NS direction and 588.1cm/s^2 in EW direction. Acceleration response spectrum peaks at 0.4s and 1.0s in EW direction. It is guessed that ground motion in EW direction is stronger than one in NS direction.

Table.6 Columns on the 1st floor

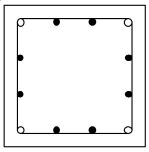
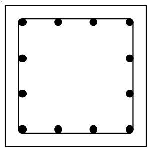
Column	A-1, A-4 B-1, B-4	A-2, A-3 B-2, B-3
Dimensions (mm)	 450×450	 450×450
Main Reinforcement	4-22φ 8-19φ	12-22φ
Hoop	9φ@250	

Fig.20 Building (B)

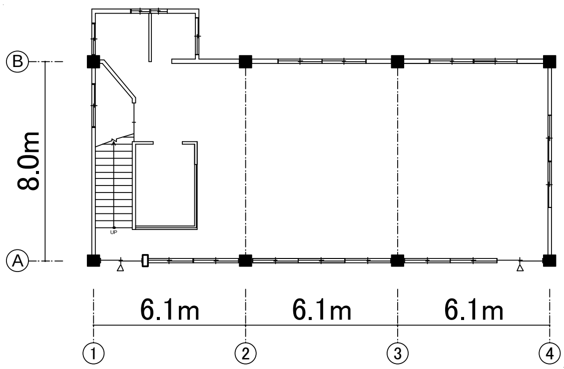


Fig.21 1st floor plan

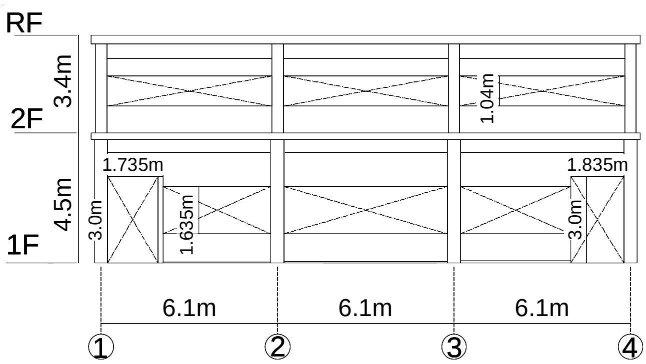


Fig.22 Framing elevation of line A

Fig.23 Survey of reinforcement

Table.7 Result of compression test of concrete

story	Strength of concrete		
	average $\bar{X}$ (N/mm^2)	standard deviation σ (N/mm^2)	$\bar{X}-\sigma/2$ (N/mm^2)
2	28.6	5.16	26.1
1	21.3	4.00	19.3

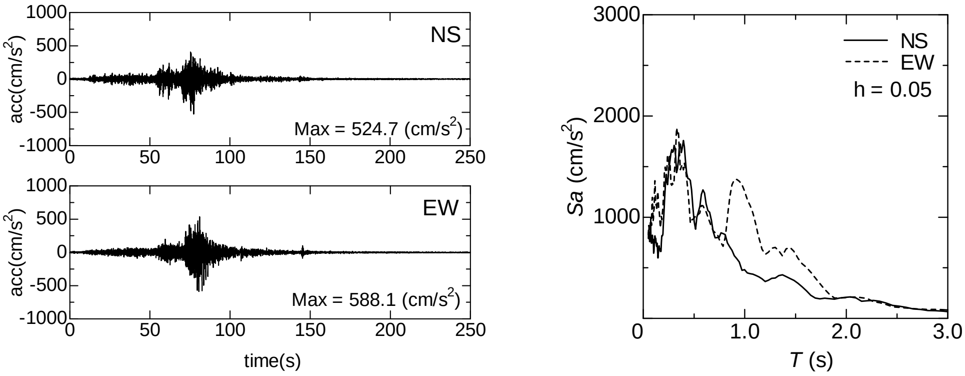


Fig.24 Strong ground motion and acceleration response spectrum

3.1 Observed damage

The floor plan and damage levels in columns are shown in Fig.25. Columns on the A line had serious damage specially as shown in Fig.26. It is observed that shear failure and falling of the concrete cover on two columns. These columns have spandrel walls of concrete-block. Other members are classified flexural columns which has no more than 1.0 mm. Concrete-block wall of the 4 line has falling of mortar and exposure of reinforcing bars. Some cracks were observed on a part of beams. There are seven flexural columns which were slight damaged and a shear column which were minor damaged on the 2nd floor. The 2nd floor has less damage than 1st floor. Residual ratio of seismic performance based on the Post-earthquake damage evaluation standard of Japan are shown in Table.8. Building (B) has moderate damage on the 1st story and minor damage on the longitudinal direction of 2nd story.

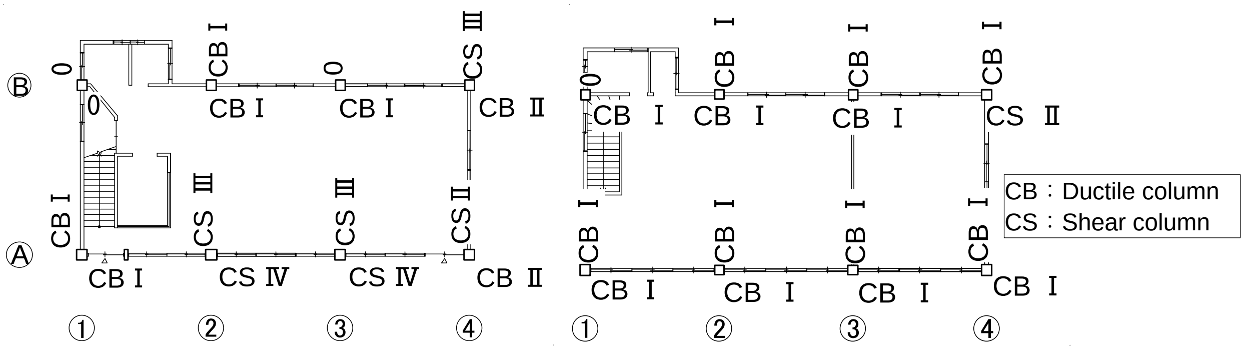


Fig.25 Floor plan and damage levels in columns


Fig.26 Shear failure of columns

Table.8 Residual ratio of seismic performance

story	longitudinal		lateral	
	residual ratio *R* (%)	damage level	residual ratio *R* (%)	damage level
2	90.6	minor	95.6	slight
1	66.9	moderate	67.5	moderate