- Technical Paper-

STUDY OF LOW-RISE RC BUILDINGS WITH RELATIVELY HIGH SEISMIC CAPACITY DAMAGED BY GREAT EAST JAPAN EARTHQUAKE 2011

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ABSTRACT

This paper investigates the damage of several low-rise RC buildings caused by the Great East Japan Earthquake in Sendai City. The Selected building are evaluated to have high seismic capacity, index I_s > 0.7, using Japanese Standard for Seismic Evaluation of Existing RC Buildings. Causes for the damaged are discussed. Moreover, pushover analysis was carried out to those buildings to check its applicability to predict the actual damage. In general, pushover analysis predicted well the damage level, but there were some differences in plastic hinge locations when compared to the actual damage.

keywords : Great East Japan earthquake, Seismic evaluation, existing RC buildings, damage, pushover analysis

1. INTRODUCTION

The Mw 9 Great East Japan Earthquake on the 11th of March 2011 had generated significant ground shaking in the western Pacific Ocean with its epicenter about 72 km east of the Oshika Peninsula of Tohoku, Japan. The PGA exceeded 1000 cm/s^2 in several locations and the maximum recorded acceleration was 2699 cm/s^2 obtained by National Research Institute for Earth Science and Disaster Prevention (NIED) at station MYG004 N-S direction [1].Although RC buildings preformed well and damage is not greater than previous earthquakes such as 1995 Kobe and 2004 Nigata Chuetsu Earthquake, some buildings with relatively high seismic capacity, I_s index greater than 0.7, were evaluated to have a moderate and severe damage.

This study presents the investigation of selected buildings which were evaluated to have relatively high seismic capacity, but had moderate and severe damage induced by ground motion of the Great East Japan Earthquake 2011.The selected buildings were chosen from Tohoku University`s post earthquake damage survey and school investigation of reinforced concrete building structures performed by RC committee of the Architectural Institute of Japan.

This paper is divided into two main sections. First the study of lecture-room RC building of 2-stories constructed in 1966 located in Tohoku University engineering campus which was severely damaged is presented. This building will be referred to as N Lecture (Fig.1). N Lecture is compared to another lecture-room building similar in its structural system and standing next to it but the latter was slightly damaged. This building will be referred to as S Lecture.

Fig.1 N Lecture building

Secondly, the study of 3 storied RC building of an elementary school in Sendai city constructed in 1974 is presented (Fig.2). The building is divided by expansion joint into west side and east side. Seismic evaluation was carried out to both sides. According to the seismic evaluation, the East side building needed to be retrofitted and the West side was evaluated to have enough seismic capacity and no retrofitting was needed. The East side building, which had already seismically retrofitted suffered only minor damage in its structural members. On the other hand, the West side building was heavily damaged.

Fig.2 North view of H school building

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2. CASE STUDY NO.1

As mentioned above, two lecture-room buildings, N Lecture and S Lecture, were investigated and compared. Both buildings are identical in plan, span, members' sizes and reinforcement. Structural system in longitudinal direction is moment frame. However S Lecture building have extra one shear wall in its longitudinal direction. The height of the 1st floor is also different. The height of the 1st floor of S Lecture building is 5.22 m and the height of the 1st floor of N Lecture building is 4.02m. Both buildings have nonstructural partial height concrete wall attached against some of its columns. Therefore, the clear height of columns is also different from a column to another.

2.1 Observed damage

The N Lecture building had a severe shear failure in many of its columns in the 1st story in the longitudinal direction (see Fig.3). The damage to columns progressed much due to the 7th of April aftershock earthquake (see Fig.4). Less damage in the 2nd story but shear cracks were also noticed. Typical column size and its reinforcement is shown in Fig.5. First floor plan with damage classes in columns in the longitudinal direction are shown in Fig 6. Two columns were damaged by previous earthquake and strengthened by FRB sheets jacketing are marked as (? unknown) in Fig. 6. It is marked unknown because damage to concrete was invisible by the FRP jacket. The details of this repair were unavailable. However, the transverse direction was slightly damaged.

Fig.3 After 11th of March earthquake

Fig.4 After 7th of April aftershock

The damage levels of each column are marked with Roman numbers based on the "Post-earthquake damage evaluation standards of Japan."[2].

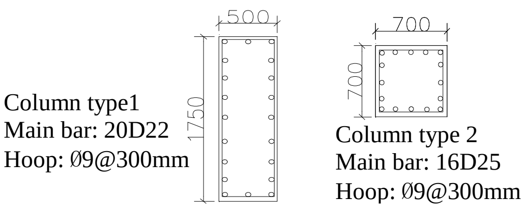


Fig.5 Typical column size and reinforcement

The S lecture building had slight damage. However, Small shear cracks from width of 0.2mm~1 mm were noticed in the longitudinal shear wall. No cracks were seen in columns of 1st story.

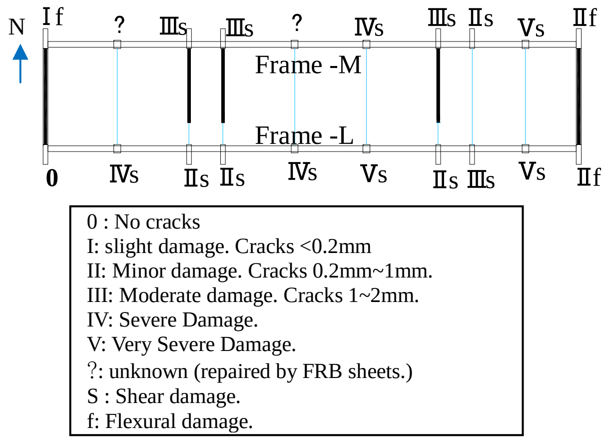


Fig6. 1st floor plan and damaged observed in longitudinal direction For N lecture building

2.2 Seismic evaluation results

The Japanese standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (JBPDA)[3] were applied to the 1st story for the longitudinal direction of the both buildings and the results of the second level procedure are shown in Table 1.

I_s Index is calculated by Eq.1:

$$I_s = E_0 \times S_D \times T \tag{1}$$

E_0 is a basic structural index calculated by Eq.2:

$$E_0 = \phi \times C \times F \tag{2}$$

Only overview of the seismic evaluation method is explained here. C-Index is strength index that denotes the lateral strength of the members in terms of shear force coefficient, namely the shear normalized by the weight of the building sustained by the story. F-Index denotes the ductility index of the member ranging from 0.8 (extremely brittle) to 3.2 (most ductile). Ø is story index that is a modification factor to allow for the mode shape of the response along the building height. S_D and T are reduction factors to modify E_0 in consideration of structural irregularity and deterioration after construction, respectively. C_{TU} is the cumulative strength index evaluated at the ultimate state of a story. The Seismic Evaluation Standard recommends as the demand criterion that Is-Index higher than 0.6 should be provided to prevent major structural damage or collapse.

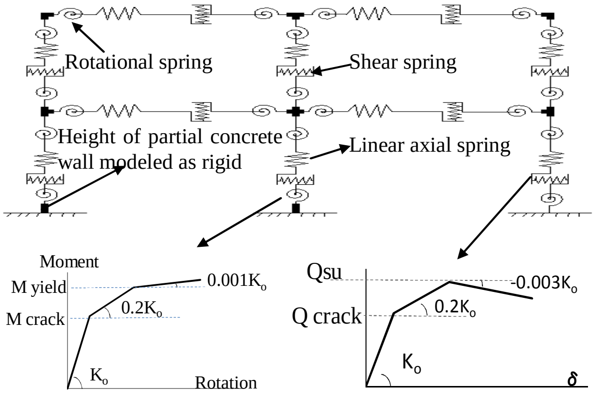
Table 1 Seismic capacity in Second level procedure of 1st story longitudinal direction

	C (groups)	F (groups)	E_0	S_D	$C_{TU}\cdot S_D$	T	I_s
Lecture N building	0.16	1	0.86	0.975	0.72	0.9	**0.755**
	0.46	1.14					
	0.35	1.9					
Lecture S building	0.37	1	0.9	0.95	0.76	0.9	**0.77**
	0.32	1.23					
	0.29	2.5					

I_s index values for both building are about the same. Since I_s> 0.6, both buildings were considered to have sufficient seismic capacity and no retrofitting was needed.

2.3 Pushover analysis

Two-dimensional pushover analysis using computer program SNAP is carried out for the longitudinal frames. Beams and columns are idealized by two nonlinear rotational springs at their ends, nonlinear shear spring in the middle and linear axial spring. A tri-linear relation is used for rotational and shear springs (see Fig.7). Cracking and yield moment of rotational spring and shear spring are estimated using AIJ standard [4]. The contribution of slab and hanging partial walls to the beams strength were ignored. The beam-column connection is assumed to be rigid. The part of the column attached to the partial concrete wall is modeled as rigid part. The shearwall is replaced with an equivalent brace model suggested in [5]. The distribution of lateral forces in the pushover analysis is based on the Ai distribution prescribed in the provision [6]. The pushover analysis is carried till the story drift reaches the maximum story drift which is assumed to be 1/100. The shear versus displacement relation of each story is reduced to equivalent single degree of freedom and expressed in spectral acceleration and displacement (Sa-Sd) relations using procedures in Japanese performance-based seismic design [7].



Rotational spring backbone curve

Shear spring backbone curve

Fig.7 Member modeling and backbone curve of nonlinear springs used in pushover analysis

Strong ground motion observation station of Tohoku University engineering campus (THU) is located at distance of 250m from the investigated buildings as shown in Fig 8. The response spectra for THU EW are plotted against pushover curves which represents the capacity of the buildings for both buildings in Fig.9.

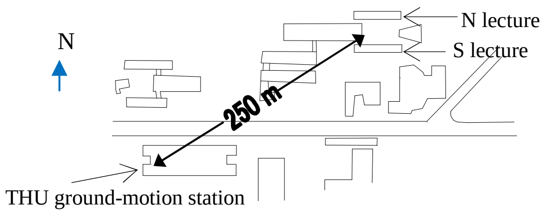


Fig. 8 Engineering campus of Tohoku University

The response spectra curve of THU EW has low values and sharp peaks at short periods as shown in Fig.9 and intersects the capacity curve of both buildings at low Sa values. If the capacity method is used, the seismic response for both buildings was expected in the elastic region and that contradicts the actual damage observed in the buildings. Therefore anticipated seismic response was chosen at the point where the actual damage observed relatively matched the damage calculated by pushover analysis which was at relative story drift angle of about 1/200rad in the 1st story for both buildings.

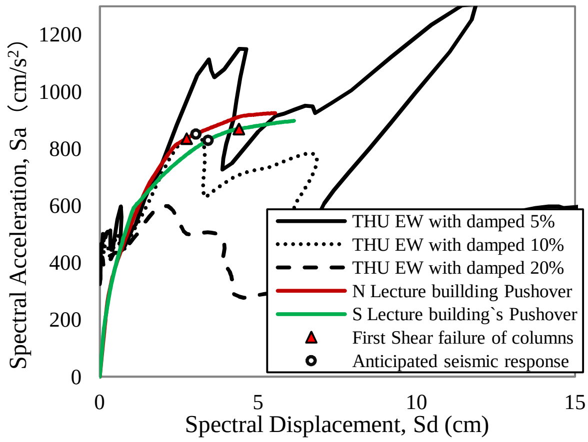


Fig.9 THU EW response spectrum

Fig. 10 shows yielded hinge locations in longitudinal direction of one frame of S lecture building at story drift of 1/200rad. The Shear wall has yielded and hinges are formed in two beams and two columns. However, as for the actual damage for S Lecture building, only shear cracks was noticed at the shear wall, no cracks was observed in other members. In the other hand, at the same story drift angle, three columns of the N Lecture building had failed in shear (Fig 11) and many columns were about to fail in shear.

Plastic hinges were expected in some beams as shown in the Fig. 10 and Fig. 11 which were not noticed in the actual damage investigation.

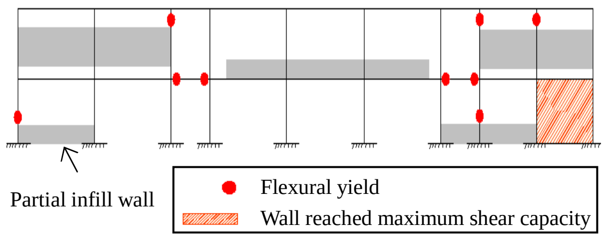


Fig.10 Damage predicted by pushover analysis for frame M of the S lecture at 1st story drift 0.5%

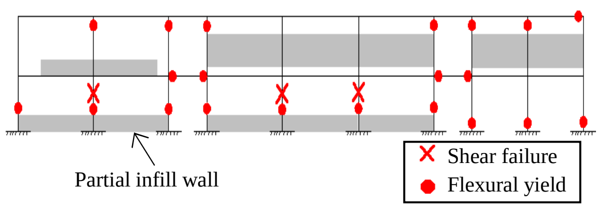


Fig.11 Damage predicted by pushover analysis for frame M of the N lecture at 1st story drift 0.5%

2.4 Discussion

When comparing damage predicted by pushover analysis with the observed damage, the pushover analysis showed good estimation of the damage level and location of shear failure and plastic hinges.

As shown in Table.1 the I_s index and $C_{TU}.S_D$ index for both of the buildings are almost the same, but the N lecture building had a greater damage than expected in the seismic evaluation. This is thought to be of the poor construction of columns. The cover around the hoops is so thin in some cases 0.5 cm (see Fig.12). Moreover, the ends of bars should be hooked by bending of at least of 135°, but in this case it was bent by angle of 90^0 as shown in Fig.13. Therefore, the bond between the concrete and hoops is weak and the hoops didn't reach its maximum yielding point and slipped. The hoops are not really helping shear strength. The N Lecture building depends in its seismic capacity mainly on shear columns. In the other hand, the S Lecture building depends mainly on the wall for its seismic capacity. The shear capacity for the columns didn't reach its maximum strength since most of the seismic load was carried first by the wall which was not affected by the poor detailing of the hoops.

Fig12. Cover of 0.5cm

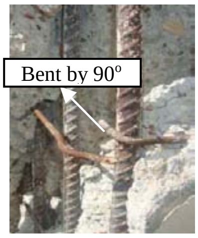


Fig13. 90^o hooks

3. CASE STUDY NO.2

3 storied RC building of an elementary school in Sendai city constructed in 1974 is studied. The building is divided by expansion joint into West side building (W) and East side building (E) as shown in Fig. 14. Total floor area for building (E) is 2542 m^2.Total floor area for building (W) is 3348 m^2.

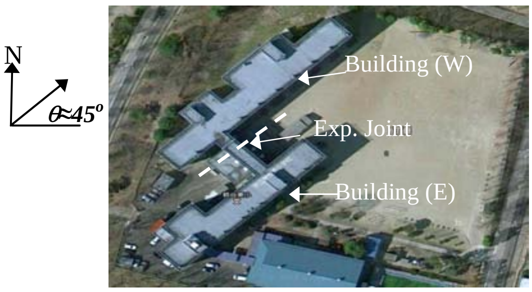


Fig14. Bird's Eye View of H elementary school

Seismic evaluation was carried out to both sides. According to the seismic evaluation, building (E) needed to be retrofitted and the building (W) was evaluated to have enough seismic capacity and no retrofitting was needed. Building (E) was retrofitted by adding framed steel braces in the 1^{st} and 2^{nd} floor and shear walls.

3.1 Observed damage

The longitudinal direction of building (W) had shear failure in many of its columns (see Fig.15) and shear cracks in wing walls (see Fig.16). Cracks in slab and beams were also observed. The transverse direction had slight damage. Damage was concentrated in the 1^{st} floor of building (W).

As for building (E), minor damage was concentrated in 3^{rd} floor. Flexural and shear Cracks of less than 1mm in some columns in the 3^{rd} story was noticed. As for 1^{st} and 2^{nd} floor, no damage was noticed. This could be because steel braces for retrofitting was only added to 1^{st} and 2^{nd} floors.

Fig15. Shear failure in column. Building (W)

Fig.16 Shear Crack in side walls. Building (W)

Typical story plan and its damage to the longitudinal direction of building (W) are shown in Fig. 17. Typical member`s size and reinforcement are shown in Fig.18

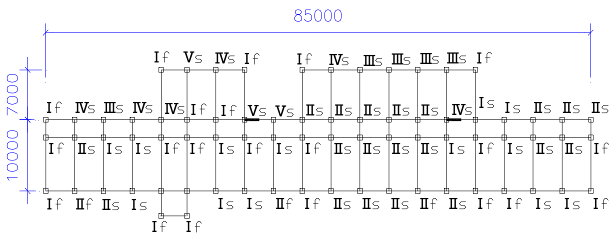


Fig17. 1^{st} floor plan and damaged observed in longitudinal direction of building (W)

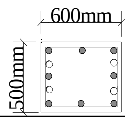
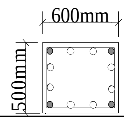
Typical column	Building (E) (1^{st} floor)	Building (W) (1^{st} floor)
Dimensions	600mm, 500mm 	600mm, 500mm 
Main Reinf.	8D22, 4D19	4D22, 8D19
Hoop	Ø9, Ø13@100mm	Ø9, Ø13@100mm

Fig18. Typical member size and reinforcement

3.2 Seismic evaluation results

The seismic evaluation for building (E) before it was retrofitted is shown in Table 2. The first and second columns in the table represent C and F indexes for different groups of members. The third and fourth columns of the table represent the F index and cumulative C index at the point where I_s index chosen for the 1^{st} story.

Table 2 Second level screening 1st story longitudinal direction building (E) before retrofit

C (groups)	F (groups)	C	F	E_o	SD	T	$C_{TU}\cdot S_D$	I_s
0.21	0.8	**0.66**	**1**	0.66	0.879	0.98	0.58	**0.57**
0.44	1							
0.02	1.4							
0.04	1.6							
0.07	1.8							
0.14	2.8							
0.01	3.2							

Since I_s index is less than 0.7, which is the criteria for schools in Japan, it was retrofitted. The seismic evaluation after retrofit for building (E) is shown in Table 3. The seismic evaluation for building (W) is shown in Table4.

Table 3 Second level screening 1st story longitudinal direction building (E) after retrofit

C (groups)	F (groups)	C	F	E_o	SD	T	$C_{TU}\cdot S_D$	I_s
0.17	0.8	**0.88**	**1**	0.87	0.879	0.98	0.77	**0.75**
0.56	1							
0.02	1.4							
0.02	1.6							
0.06	1.8							
0.14	2							
0.13	2.8							
0.01	3.2							

Table 4 Second level screening 1st story longitudinal direction building (W)

C (groups)	F (groups)	C	F	E_o	SD	T	$C_{TU}\cdot S_D$	I_s
0.09	0.8	**0.50**	**1.75**	0.87	0.93	0.98	0.46	**0.80**
0.03	1							
0.07	1.75							
0.09	2							
0.18	2.25							
0.15	2.6							
0.02	3.2							

3.3 Pushover analysis

Description of the pushover analysis as mentioned in previous section is used. The seismic response is calculated using a bilinear idealization of pushover curves and procedures in Japanese performance-based seismic design.

Two Strong ground motion observation stations are located at some distance from the school building. (See Fig. 19).

Fig19. Strong motion observation stations

The building is oriented at an angle of 45° from North (see Fig.14). The response acceleration at an angle of 45° from north will be referred as NW in this paper. Response spectra for NW of K-net MYG013 [1] and NW of JMA Sendai [8] are plotted against pushover curve in Fig.21 and Fig. 22 respectively.

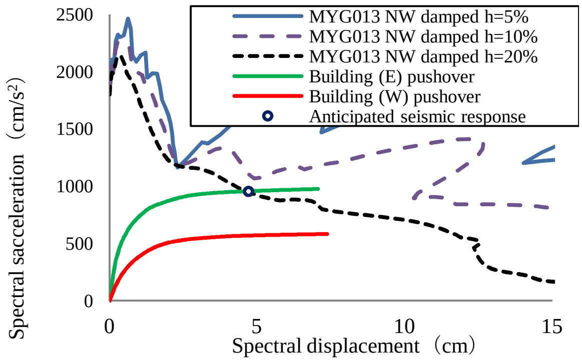


Fig.20MYG013 NW response spectrum and pushover curves

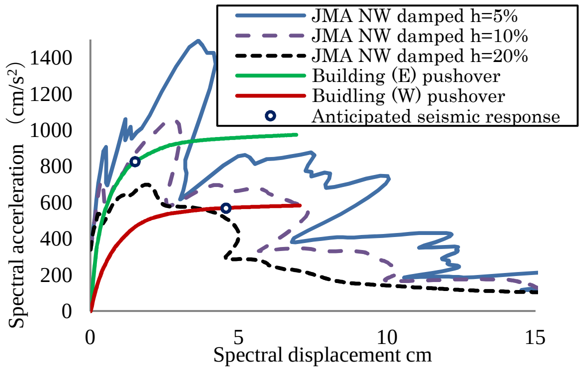


Fig. 21 JMA NW response spectrum and pushover curves

Comparison between pushover analysis results for first story for both buildings and seismic evaluation results is shown in Fig.22 and Fig.23. Base shear coefficient is the lateral shear force at the base of building normalized by the weight of the building. The F index is converted to lateral displacement as follows: F=0.8 is equivalent to Inter-story drift of 1/500, F=1 is equivalent to Inter-story drift of 1/250, F=1.27 is equivalent to Inter-story drift of 1/150 and for F>1.27 the Eq.3 is used.

$$F=\frac{\sqrt{2\cdot Rmu/Ry-1}}{0.75\cdot(1+0.05\cdot Rmu/Ry-1)} \qquad (3)$$

Where;

Ry : Yield deformation in terms of inter-story, which in principle shall be taken as Ry = 1/150.

Rmu : Inter-story drift angle at the ultimate deformation capacity.

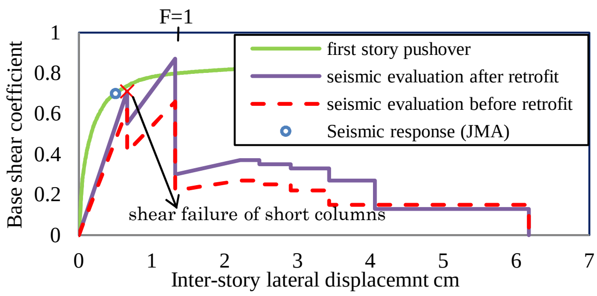


Fig22. Comparison between 1st story pushover and seismic evaluation results of building (E)

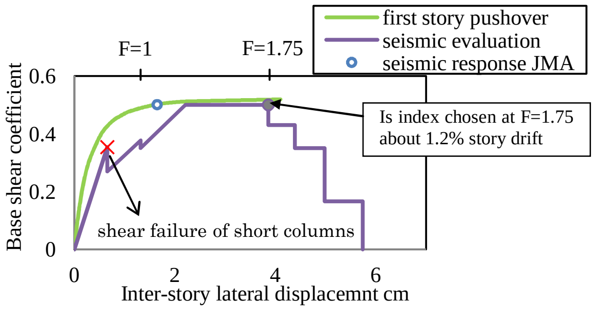


Fig23. Comparison between 1st story pushover and seismic evaluation results of building (W)

3.4 Discussion

I_s index for both buildings are chosen at a ductility greater than F=0.8, which means that the failure of extremely short columns was allowed in the seismic evaluation, since axial loads could be redistributed to other columns and the building will not collapse. However, short column failed only in building (W) and didn't in building (E). This is due to several reasons; The C_{TU} . S_D for 1st story at ductility index F = 0.8 for building (E) is about twice of building (W), as shown in Table 5. It is thought that building (E) had a story drift just less than F=0.8. This is shown using seismic response of JMA NW which is just before the shear failure of short columns (see Fig. 22). Therefore short columns didn't reach their maximum allowable ductility and escaped shear failure. In the other hand, The I_s value for building (W) depends on ductility (F=1.75) to reach the criteria of I_s > 0.7. The anticipated seismic response using either JMA NW or MYG013NW spectrum are greater than F=1 (see Fig.23). At this seismic response, all short and shear columns had shear failure.

Table 5. $C_{TU} \cdot S_D$ at F=0.8

	$C_{TU} \cdot S_D$ at F=0.8
Building A	0.62
Building B	0.33

According to structural drawings, slits are inserted between columns and interior infill concrete walls in building (W). However, these slits weren't inserted properly. Therefore, columns which were assumed to have flexural failure in the seismic evaluation had actually failed in shear (see Fig.15). This resulted in greater damage than expected.

Two Response spectra, MYG013 KNET and JMA Sendai, are used for analysis. Using MYG013 response spectra, Fig. 20, it is demonstrated that more significant structural damage would occur in building (E) and a higher probability of collapse for building (W). In the other hand, the damage expected using JMA Sendai Spectra, Fig. 21, relatively matches the actual damage.

The actual damage for building (W) matches the pushover analysis results in some columns and doesn't in others columns. This is because plastic hinges were expected to occur in beams and not in columns, but this wasn't the case of the actual damage. This could be due to the contribution of the slab and the contribution of hanging concrete walls to the beam`s strength.

The shear failure of short and shear columns in building (W) were expected in a major earthquake using the seismic evaluation. This building wasn't retrofitted since it's judged that there is no threat to life safety. However, the school could not use this building after the earthquake and repairing expenses would be relatively high if compared to the retrofitting expenses for a better performance. If the school administration was informed of possible consequences about the function of its building and repairing costs after an earthquake, they might be willing to pay additional expenses for higher performance. This case raises two issues; the function-ability of the building after the earthquake and the lack of communication between the structural engineer and owner.

4. CONCLUSION

The pushover analysis predicted well the damage`s level and location in S and N buildings. In the other hand, for H school building there were some differences in damage locations. Some plastic hinges were expected to occur in beams and not in columns. In general, capacity spectrum method predicted well the level of damage.

A study of selected buildings was presented. As for the lecture-room buildings in Tohoku University, seismic evaluation method couldn't explain why the two buildings are different in actual damage. Poor detailing of hoops is thought to be the main cause of greater damage than expected. As for the H school building, the shear failure of short and shear columns in building (W) were expected in a major earthquake using the seismic evaluation. This case raises the problem of the function-ability of the building after earthquakes.

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