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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 Is > 0.7, using Japanese Standard for Seismic Evaluation of Existing RC Buildings. Causes of the damage are discussed. Moreover, pushover analysis was carried out to those buildings. In general, pushover analysis predicted well the damage level, but there were some differences in plastic hinge locations when compared to the actual damage.

Key Words: Great East Japan earthquake, seismic evaluation, existing RC buildings, structural damage, pushover analysis.

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 gal in several locations and the maximum recorded acceleration was 2699 gal in Miyagi prefecture obtained from National Research Institute for Earth Science and Disaster Prevention (NIED) at station MYG004 N-S direction. 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.

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. 1 N Lecture building

Fig. 2 North view of H school building

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 (see figure 3 and figure 4). 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 as shown in Fig.3 and Fig.4. 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. Typical column size and its reinforcement is shown in Fig.5.

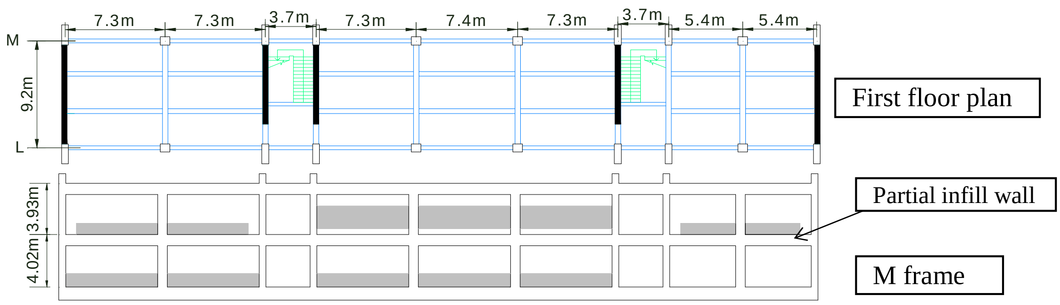


Fig.3 N Lecture building plan and elevation

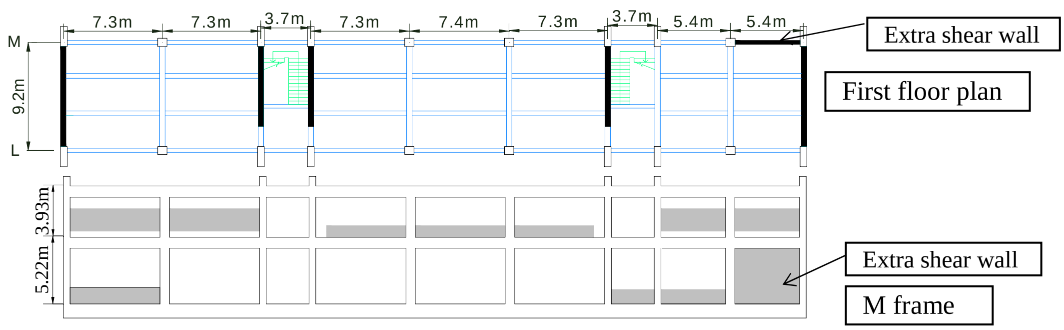
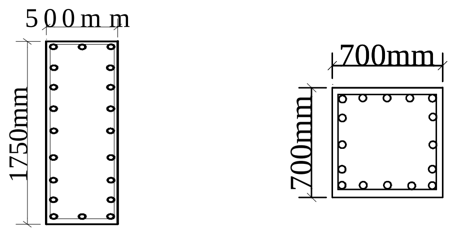


Fig.4 S Lecture building plan and elevation

Column type1
Main bar: 20D22
Stirrups: Ø9@300mm



Column type 2
Main bar: 16D25
Stirrups: Ø9@300mm

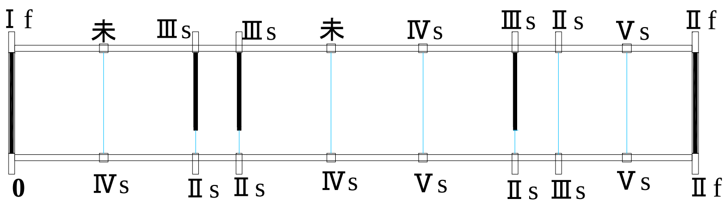
Fig.5 Typical column size and reinforcement

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.6). The damage to columns progressed much due to the 7th of April aftershock earthquake (see Fig.7). Less damage in the 2nd story but shear cracks were also noticed. First floor plan with damage classes of columns in the longitudinal direction are classified based on the "Post-earthquake damage evaluation standards of Japan" (JBDPA 2001a) and shown in Fig 8. Two columns were damaged by previous earthquake and strengthened by FRB sheets jacketing are marked as (未 unknown) in Fig. 8. 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.6 After 11th of March earthquake

Fig.7 After 7th of April aftershock earthquake



0 : No cracks
I: slight damage. Cracks <0.2mm
II: Minor damage. Cracks 0.2mm~1mm.
III: Moderate damage. Cracks 1~2mm.
IV: Severe Damage.
V: Very Severe Damage.
未: unknown (repaired by FRB sheets.)
S : Shear damage.
f: Flexural damage.

Fig8. 1st floor plan and damaged observed in longitudinal direction For N lecture building

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.

Seismic evaluation results

The Japanese standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (JBDPA

2001b) was 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**.** Is-Index is calculated by Eq.(1) .

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

E_0 is a basic structural index calculated by Eq.(2).

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

C-Index is strength index that denotes the lateral strength of the buildings in terms of shear force coefficient. F-Index denotes the ductility index of the building ranging from 0.8 (extremely brittle) to 3.2 (most ductile), depending on the sectional properties such as bar arrangement, member proportion, shear-to-flexural-strength ratio etc. Ø is story index that is a modification factor to allow for the mode shape of the response along the building height. SD and T are reduction factors to modify E_0 in consideration of structural irregularity and deterioration after construction, respectively. 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. This criterion is based on the correlation study from the past earthquake damage and the calculated indices for the damaged buildings. Past experiences of the big earthquakes reported that buildings with Is-Indices higher than 0.6 escaped severe damage or collapse.

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

	C (groups)	F (groups)	E_0	S_D	$C_{TU}.S_D$	T	Is
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> 0.7, both buildings were considered to have sufficient seismic capacity and no retrofitting was needed.

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. The stiffness after yielding is taken as 1/1000 of the elastic stiffness. Cracking and yield moment of rotational spring and shear spring are estimated using AIJ standard (AIJ standard 1999). The contribution of slab and hanging partial walls to the beams strength were ignored. The beam-column connection is assumed to be rigid. The shearwall is modeled with an equivalent brace model suggested in (Aoyama 1987). The distribution of lateral forces in the pushover analysis is based on the Ai distribution prescribed in the provision (Ministry of Land, Infrastructure and Transport Notification 1980). 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 (M. Midorikawa 2003).

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 9. The response spectra for THU EW are plotted against pushover curves which represents the capacity of the buildings for both buildings in Fig.10.

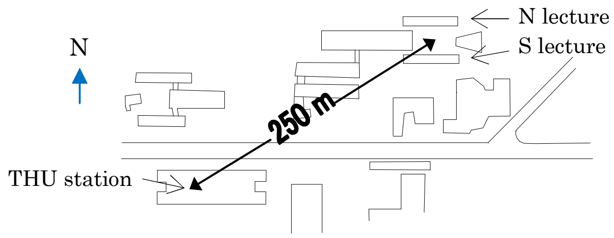


Fig. 9 Engineering campus of Tohoku University

The response spectra curve of THU EW has low values and sharp peaks at short periods as shown in Fig.10 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 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. Due to large spacing between stirrups, 300mm, brittle failure and rapid degradation of shear resistance is estimated and shown as the green dotted line in Fig.8.

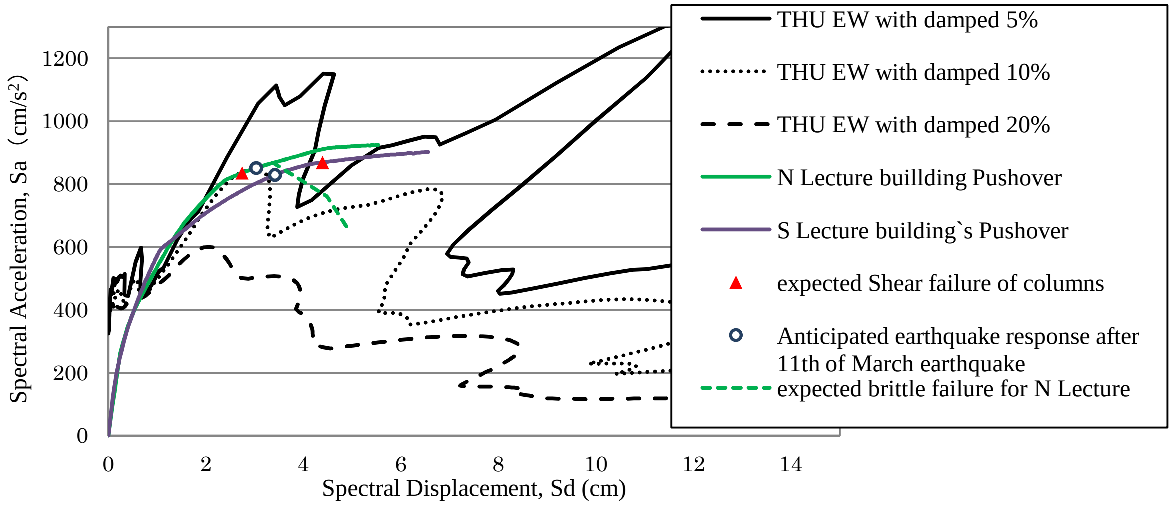


Fig.10 THU EW response spectrum

Fig. 11 shows yielded hinge locations in frame in longitudinal direction 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 (see Fig 12) and many columns were about to fail in shear and had plastic hinges.

Plastic hinges were expected in some beams as shown in the Fig. 11 and Fig. 12 which were not noticed in the actual damage investigation. This could be due to the contribution of slab which was not considered in the calculations of the beams strength capacity.

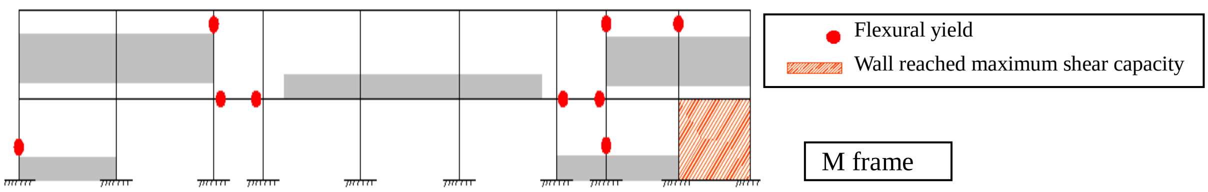


Fig.11· Damage predicted using pushover analysis for S lecture at 1st story drift angle of 1/200rad

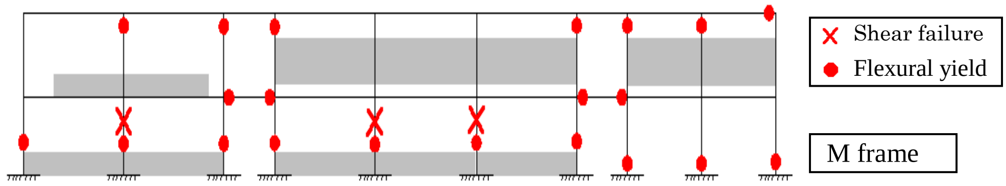


Fig.12 Damage predicted using pushover analysis for N lecture at 1st story drift angle of 1/200rad

Comparison of base shear-displacement curve in the first story for both buildings between pushover analysis results and seismic evaluation results is shown in Fig.13 and Fig.14. 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.

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

Where; Ry : Yield deformation in terms of inter-story, which in principle shall be taken as Ry = 1/150 and Rmu : Inter-story drift angle at the ultimate deformation capacity.

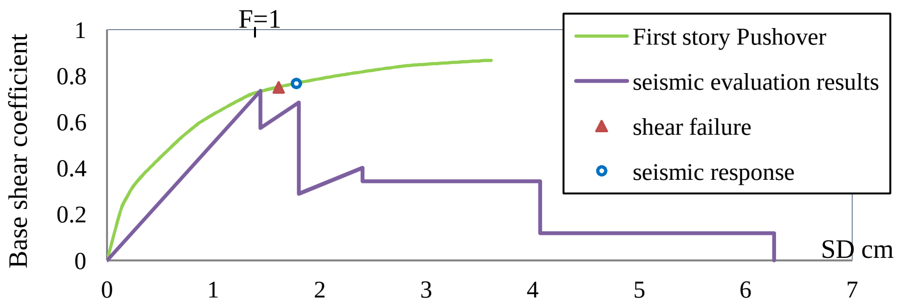


Fig13. 1st story pushover and seismic evaluation results of N Lecture Building

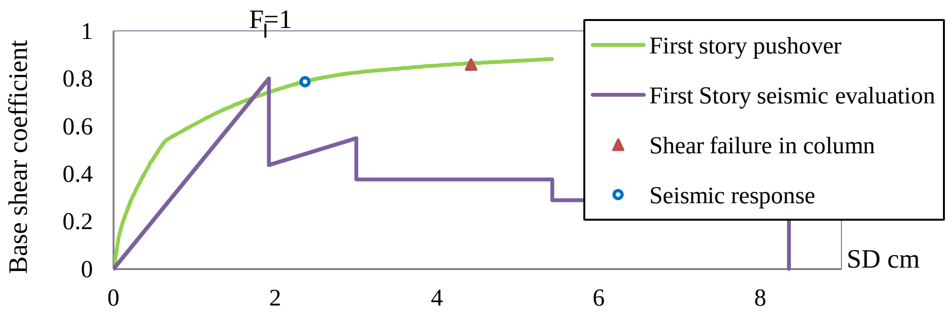


Fig14. 1st story pushover and seismic evaluation results of S Lecture Building

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 Is index and $C_{TU}.S_D$ index which represent the base shear 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 stirrups is so thin in some cases 0.5 cm (see Fig.15). Moreover, the ends of bars should be hooked by bending through 180º in mild-steel bars and 135º in deformed bar. But in this case it was bent by angle of 90^0 as shown in Fig.16. Therefore, the bond between the concrete and stirrups is weak and the stirrups didn't reach its maximum yielding point and slipped. The stirrups 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 by the wall which was not affected by the poor detailing of the stirrups.

Fig15. Cover of 0.5cm

Fig16. 90^o hooks

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 building (A) and building (B) as shown in Fig. 17. Average floor area for building (A) is 857.6m^2 and total floor area of 2542 m^2. Average floor area for building (B) is 1116m^2 and total floor area of 3348 m^2.

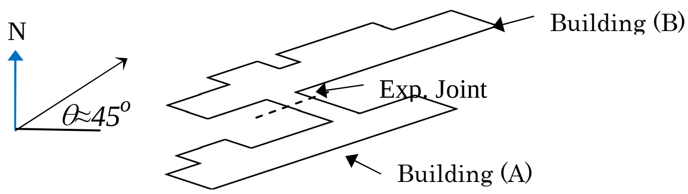


Fig17. H elementary school

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

Observed damage

The longitudinal direction of building (B) had shear failure in many of its columns (see Fig.18) and shear cracks in some wing walls (see Fig.19). Cracks in slab and beams were also observed. The transverse direction had slight damage. Damage was concentrated in the 1st floor of building (B).

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

Fig18. Shear failure in column. Building (B)

Fig.19 Shear Crack in side walls. Building (B)

Typical story plan and its damage to the longitudinal direction of building (B) are shown in Fig. 20. Typical member`s size and reinforcement is shown in Fig. 21.

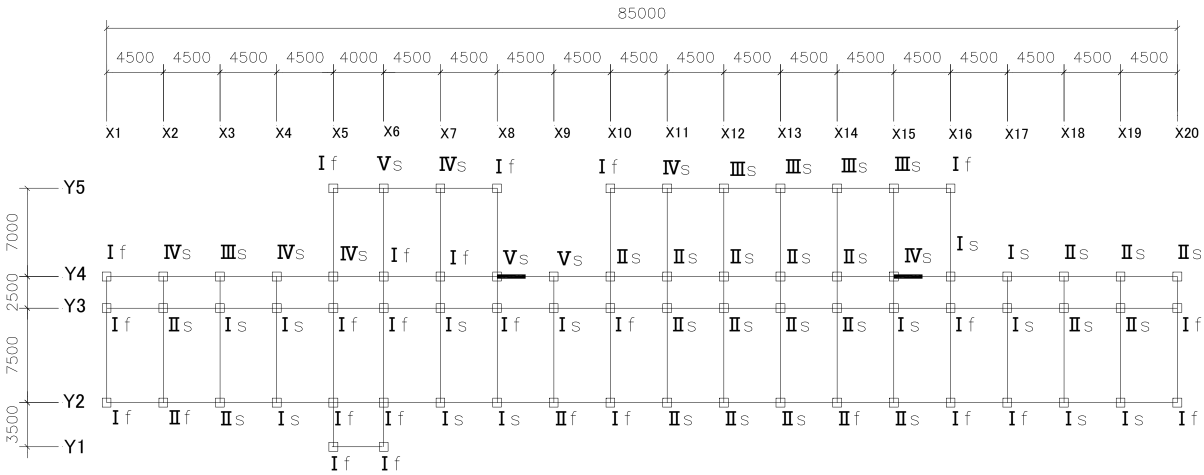
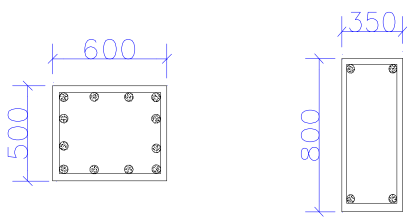


Fig. 20 1st floor plan and damaged observed in longitudinal direction of building (B)

Typical column:
Main bar: 4D22, 8D19
Stirrups: Ø9@100mm



Typical beam:
Main bar: 2D22
Stirrups: Ø9@150mm

Fig. 21 Typical member size and reinforcement

Seismic evaluation results

The seismic evaluation for building (A) before it was retrofitted is shown in Table 2. Since I_s index <0.7, which is the criteria in Japan, it was retrofitted. The seismic evaluation after retrofit for building (A) is shown in Table 3. The seismic evaluation after retrofit for building (B) is shown in Table4.

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

C (groups)	F (groups)	C	F	E_o	SD	T	CT.SD	Is
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							

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

C (groups)	F (groups)	C	F	E_o	SD	T	CT.SD	Is
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 (B)

C (groups)	F (groups)	C	F	E_o	SD	T	CT.SD	Is
0.09	0.8	0.50	1.75	0.87	0.93	0.98	0.461	0.80
0.03	1							
0.07	1.75							
0.09	2							
0.18	2.25							
0.15	2.6							
0.02	3.2							

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. 22).

Fig22. Strong motion observation stations

The building is oriented at an angle of 45° from North (see Fig.17). The response acceleration at an angle of 45° form north will be referred as NW in this paper. Response spectra for NW of K-net MYG013 (NIED) and NW of JMA Sendai (JMA) are plotted against pushover curve in Fig.23 and Fig.24 respectively.

According to structural drawings, slits are inserted between columns and interior infill concrete walls. However, these slits weren't inserted properly. Columns which were assumed to have flexural failure had actually failed in shear (see Fig. 18). Therefore, two cases are assumed for building (B); with slits and without slits.

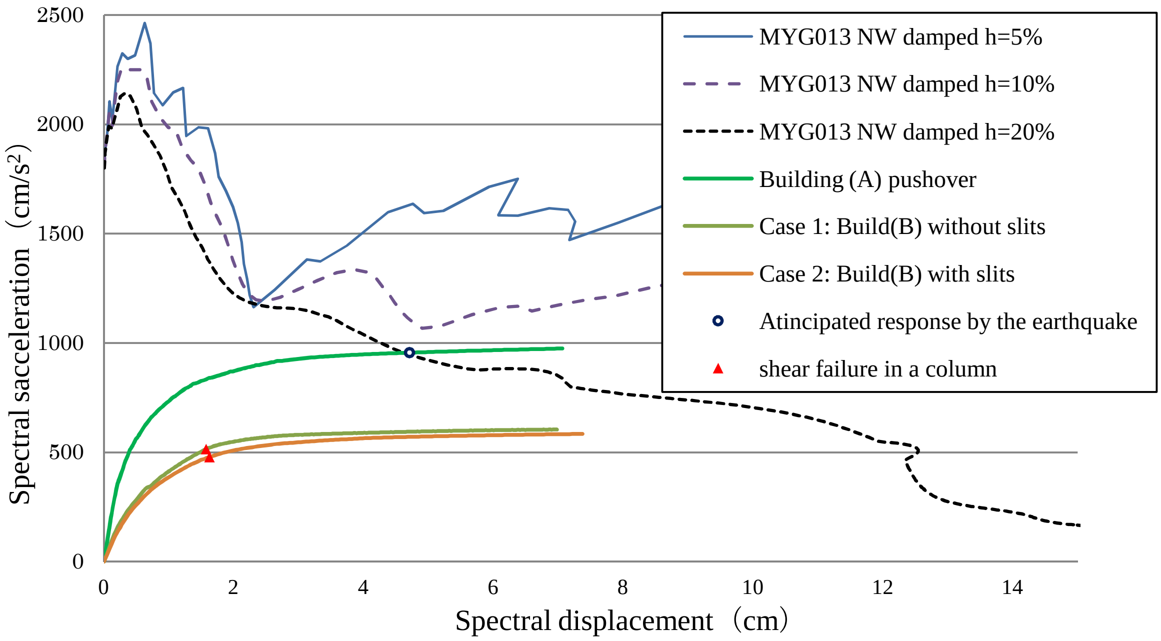


Fig.23 MYG013 NW response spectrum and pushover curves

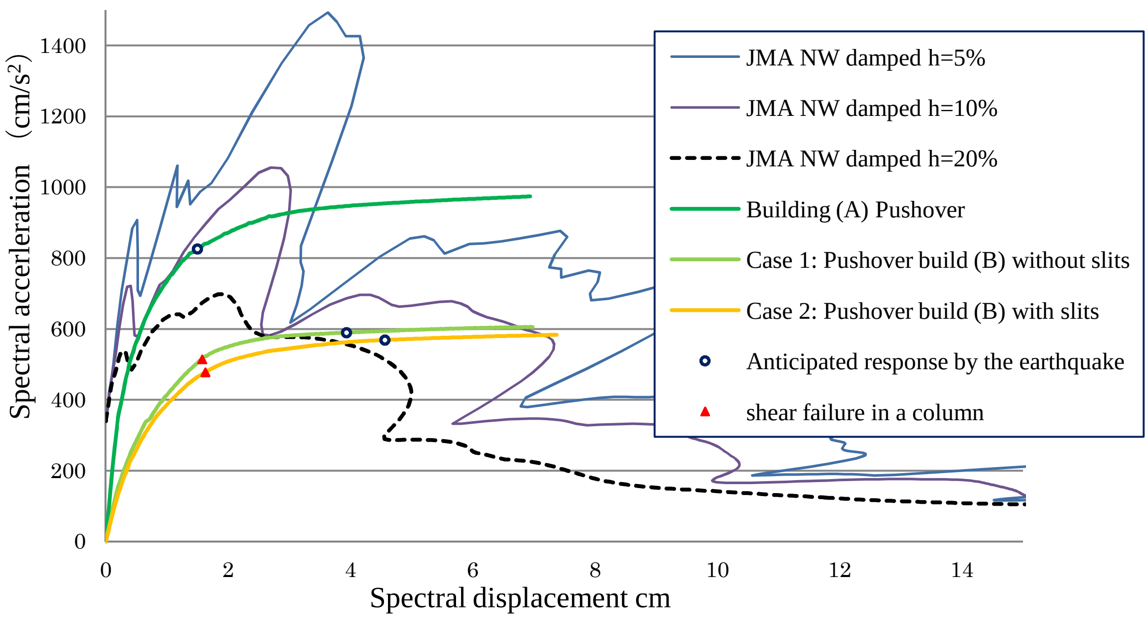


Fig. 24 JMA Sendai NW response spectrum and pushover curves

Comparison between pushover analysis results for first story for both buildings and seismic evaluation results is shown in Fig.25 and Fig.26

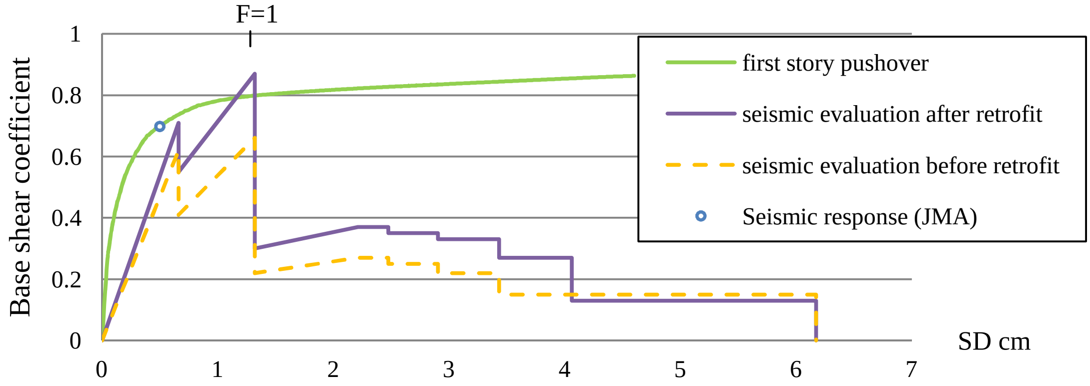


Fig. 25 1st story pushover and seismic evaluation results building A

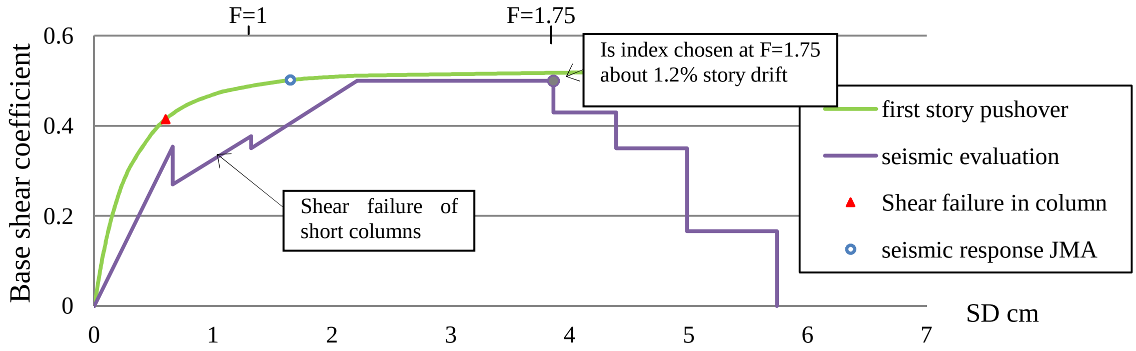


Fig. 26 1st story pushover and seismic evaluation results building B

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 (B) and didn't in building (A). This is due to several reasons; The CT.SD for 1st story at ductility index F = 0.8 for building (A) is about twice of building (B), Table 5. It is thought that building (A) had a story drift just less than F=0.8. This is shown in Fig. 23 using anticipated seismic response of JMA NW spectrum. Therefore short columns didn't reach their maximum allowable ductility and thus shear failure didn't occur. In the other hand, The I_s value for building (B) 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.24). Therefore all short and shear columns have collapsed.

Table 5 CT.SD at F=0.8

	CT.SD 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 (B). However, these slits weren't inserted properly. Therefore, columns that were expected to have flexural failure, had shear failure. This resulted in greater damage than expected.

Two Response spectra, MYG013 KNET and JMA Sendai, are used for analysis. Using MYG013 response spectra, Fig. 21, it is demonstrated that more significant structural damage would occur in building (A) and a higher probability of collapse for building (B).In the other hand, the damage expected using JMA Sendai Spectra, Fig. 22, relatively matches the actual damage.

The actual damage for building B matches the pushover analysis results in some columns and doesn't in others columns. Some columns which actually failed in shear didn't in the pushover analysis. 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 (B) 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.

CONCLUSIONS

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. Plastic hinges were expected to occur in beams and not in columns, but this wasn't the case in the actual damage. In general, capacity spectrum method predicted well the level of damage in the H school building.

As for the lecture room buildings in Tohoku University, it is concluded that the main cause for greater damage than expected by the seismic evaluation method was because of the poor construction observed in detailing the columns. As for the H school building, the shear failure of short and shear columns in building (B) were expected in a major earthquake using the seismic evaluation. The slits weren't inserted properly. This resulted in greater damage than expected. This case raises the problem of the function-ability of the building after earthquakes.

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