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Seismic capacity of retrofitted SRC building damaged due to 2011 Great East Japan Earthquake

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Abstract. Civil Engineering building at Tohoku University, which is an instrumented nine-story composite structure, survived the 1978 Miyagi Oki earthquake with minor damage. In 2001, the building was assessed and strengthened following Japanese standards (JBDPA 2001). The main actions through the strengthening process were 1) addition of steel braces to one facade, 2) replacement of concrete and reinforcement in the webs of two exterior shear walls, and 3) jacketing of short beams. In 2011, the building survived without collapse another strong earthquake: Great East Japan earthquake. The linear-response spectra computed from base-acceleration records obtained in 2011 were nearly identical to those computed from records obtained in 1978 in the transverse direction. Nevertheless, the damage caused by the 2011 motion in that direction was so severe that the building had to be evacuated and demolished. A plausible explanation of such damage and its mechanism is presented, compared and discussed with the seismic evaluation results and capacity spectrum method.
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Keywords: 2011 Great East Japan; 1978 Miyagi Oki; Seismic retrofit; Capacity spectrum method

1 INTRODUCTION

In Japan, seismic evaluation and strengthening have been widely applied to existing buildings especially since the 1995 Kobe Earthquake. Many existing buildings designed before 1981 according to old seismic code in Miyagi Prefecture were evaluated, and many of the buildings assessed as being vulnerable were retrofitted before the 2011 East Japan earthquake. Most retrofitted buildings performed well against the strong shaking by the 2011 East Japan earthquake (Maeda 2012). The seismic retrofits reduced mostly the damage to buildings in the affected area. However, some of the retrofitted buildings suffered moderate or severe structural damage. One of the severely damaged was the building of Civil Engineering (referred as "CE building" in this paper), which was located on the Aobayama campus (Engineering campus) of Tohoku University, Sendai. The locations of the building, Sendai, and the epicentre of the earthquake are shown in Figure 1.

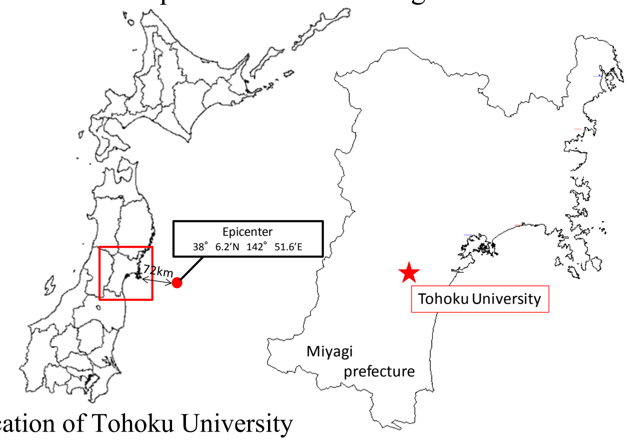


Figure 1. Location of Tohoku University

The CE building is a 9-story composite structure constructed in 1968. Accelerometers were installed in 1968 on the 1st and 9th floors by Building Research Institute (BRI). Acceleration records were obtained from a number of earthquakes including the 1978 Miyagi Oki earthquake. In the 1978 earthquake, the building suffered minor damage as reported by Shiga and Shibata (Shiga et al. 1981). The damage was repaired and the building continued the operation until 2001, when the building was strengthened.

Despite the strengthening done in 2001, the 2011Great East Japan earthquake caused severe damage to the building and lead to demolition. In this earthquake, acceleration records were obtained again on the 1st and 9th floors. This fact and the availability of 1) previous records, 2) detailed reports of the damage caused by previous earthquakes, and 3) complete construction and retrofit drawings, make the building a special case attracting many researchers (Motosaka 2012, and so on).

This paper describes 1) the configuration of the building, 2) the results of the seismic evaluation that trigger the retrofit work done in 2001, and 3) damage caused by both the 1978 and 2011. Plausible explanations for the great difference in damage are studied using a simple model.

2 BUILDING CONFIGURATION

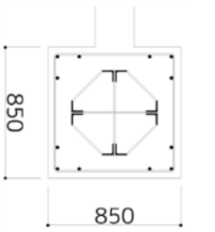
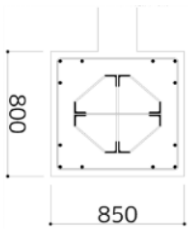
2.1 Outline of building

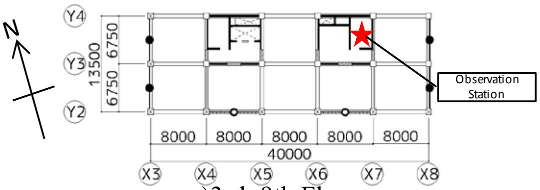
Figures 2a) and 2b) show the North and East elevations of the CE building. The building was a 9-story steel/concrete composite structure. The lower two stories were larger in the plan than the upper stories (Figure 2b), Figure 3). The typical floor plan for the 3rd to 9th floors is shown in Figure 3a), and the floor plan of the 1st and 2nd floors is shown in Figure 3b). The locations of the accelerometers are indicated by the stars in Figure 3.

(a)from North (b)from East

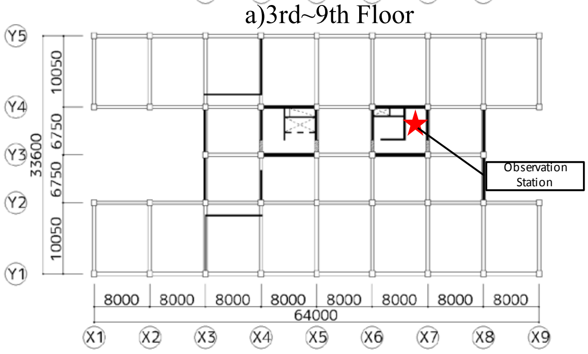
Figure 2. General view of the building

Table 1. Typical column list

Floor	1F	2・3F	4・5・6F	7・8・9F
Dimension (mm)	 850 × 850		 800 × 850	
Steel	8Ls-75X75X12	8Ls-75X75X9	8Ls-75X75X6	8Ls-65X65X6
Main bar	12-D25	12-D22	12-D19	
Hoop	Center: 9Φ@300 Top&Bottom: 9Φ@150			



a)3rd~9th Floor



b) 1st&2nd Floor

Figure 3. Plan drawing

In the transverse direction, the main lateral-load resisting system consisted of shear walls along lines X3, X8, and the C-shaped shear walls of the staircases. As shown in Figure 3a), the structure was symmetrical about its transverse axis. Forces in the longitudinal direction were resisted by the staircase walls and two reinforced concrete shear walls located along the longitudinal axis (Y3). But the structure was not symmetric about axis Y3. The asymmetry was caused by the staircase walls, which were not aligned with this axis.

Table 1 shows typical column dimensions and reinforcing details. Each column had eight vertical structural steel angles "tied" together with discrete horizontal steel plates. These plates did not form continuous webs or continuous flanges. Instead, they simply provided discrete supports for the vertical angles. These angles were embedded in concrete reinforced with twelve deformed vertical bars and widely spaced small-diameter hoops made with 9mm plain round bar (Table 1).

The material of the steel angles was reported to meet Japanese standard SS400, which specifies a nominal yield stress of σ_{sy}=235N/mm^2. The deformed vertical reinforcing bars were reported to meet Japanese standard SD345, which specifies a nominal yield stress of σ_y=345N/mm^2. Ties were made from smooth round bars meeting standard SR235 (σ_y=235N/mm^2). The specified strength of concrete was Fc = 21N/mm^2.

2.2 Seismic evaluation

Tohoku University applied the JBDPA (Japan Building Disaster Prevention Association) standard, Japanese Standard for Seismic Capacity Evaluation of Existing Reinforced Concrete Building, (JBDPA 2001) to the building in order to evaluate the seismic capacity and to decide seismic retrofit scheme. I_s-index which represents the seismic performance of the structure can be calculated by Eq.(1) at each story and each direction according to the standard. E_0 is a basic structural index calculated by Eq.(2).

$$I_s = E_0 \times S_D \times T \qquad (1)$$

$$E_0 = 1/A_i \times C \times F \qquad (2)$$

C-index is strength index that denotes the lateral strength of the buildings in terms of story shear coefficient which is the story shear normalized by weight of the building sustained by the story. *F*-index denotes the ductility index of the building ranging from 1.0 (brittle) to 3.5 (very ductile) in case of SRC building, depending on the sectional properties such as bar arrangement, member proportion, shear-to-flexural-strength ratio etc. *Ai* is the distribution of lateral forces along the height of the building is based on the *Ai* distribution (see Eq.(3)) prescribed in the AIJ provision (AIJ 1999).

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right) \times \frac{2T}{1+3T} \qquad (3)$$

where: α_i is ratio between the total weight supported by story *i* to the total weight of building, *T* is the 1st natural period

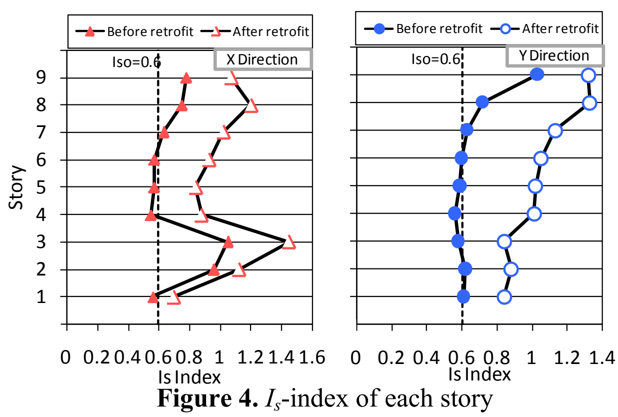
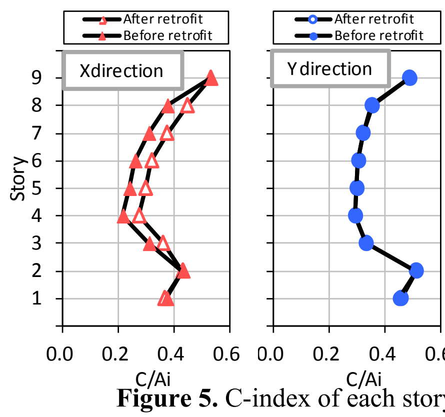


Figure 4. I_s-index of each story



Mass (ton)	Ai
1074	2.47
846	2.06
850	1.84
854	1.69
868	1.57
882	1.48
944	1.39
2800	1.18
3024	1.00

Figure 5. C-index of each story

S_D and T are reduction factors to modify E_0 considering of structural irregularity and deterioration after construction, respectively. The Seismic Evaluation Standard recommends as the demand criterion that I_s-Index higher than 0.6 should be provided to prevent major structural damage or collapse.Figure 4 shows the I_s-index of all stories for both before and after retrofit. This building was retrofitted because I_s-index was less than 0.6 for most stories. It should be noted that the F index (ductility index) used here was 3.5 assuming the building has very ductile flexural walls.

Figure 5 shows the C-index /Ai. 1st and 2nd stories have higher lateral strength (C/Ai=0.4~0.5) than upper stories in both directions. In the Y direction, 3rd-8th stories have approximately the same C/Ai value (=about 0.3). C/Ai =0.3 is the minimum requirement for RC buildings in the current seismic code in Japan

2.3 Scheme of seismic retrofit

According to the seismic evaluation results, seismic capacity I_s-index was insufficient to requirement of I_s=0.6 and was classified as a retrofit candidate. In 2001, the building was seismically retrofitted by installing framed steel braces in the longitudinal direction, replacement of RC shear walls in transverse direction (frame X3 and X8) from the 3rd to 9th story, and jacketing of adjacent short beams with steel plates to avoid shear failure, as shown in Figure 6. The main scheme of the seismic retrofit was to increase I_s-index to meet the criteria of $I_s \geqq 0.6$. The retrofit plan was to increase the ductility of the transverse direction by replacing the exterior shear walls with new exterior shear wall.

As for the longitudinal direction, steel braces were installed to increase lateral strength and also to reduce the torsion vibration induced by the irregularity of the structure in this direction.

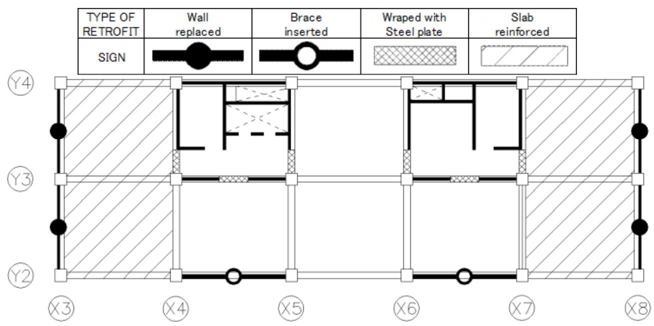


Figure 6. Part of retrofit plan

The reinforcing details of the shear panels replaced in walls X3 and X8 shown in Figure 7. It is important to emphasize that development length of post installed anchors were 110mm according to the regulation for seismic retrofit design. The length of anchor met the guidelines for seismic Retrofit of Existing Reinforced Concrete Building (JBDPA 2001). However, the tensile strength of anchor is conditioned on cone shaped fracture, as can be seen in Figure 13.

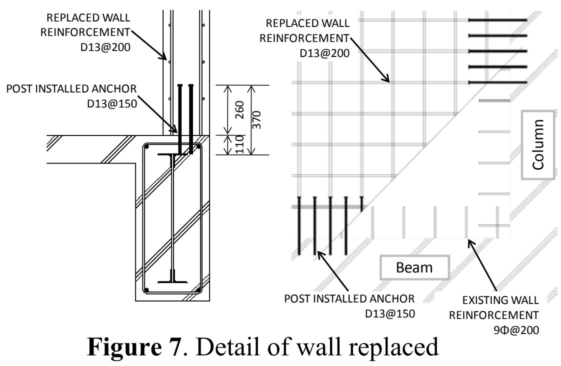


Figure 7. Detail of wall replaced

3 EARTHQUAKE DAMAGES

3.1 1978 Miyagi Oki earthquake

Detailed damage survey was conducted after the 1978 Miyagi Oki earthquake by professors Shiga and Shibata, Tohoku University (Shiga et al. 1981). According to their report, small shear cracks and flexural cracks were observed in exterior shear walls(see Figure 8), adjacent beams and a few columns on the third and fourth floor. Typical wall and short beams crack patterns are shown in Figure 9 and 10. The maximum width of shear cracks in shear walls and that of flexural cracks in columns and beams are reported about as 1.0mm. The width of shear cracks in the adjacent beams with openings was reported about as 1.5mm. Therefore, the structural damage of the building by the 1978 earthquake, was considered to be fairly minor. As can be seen in Figure 10, exterior shear wall successfully sustained lateral force and contributed as shear resisting elements, because shear cracks were relatively uniformly distributed in the wall panels.

a)All b) 3rd Floor

Figure 8. Damaged wall in 1978 Miyagi Oki

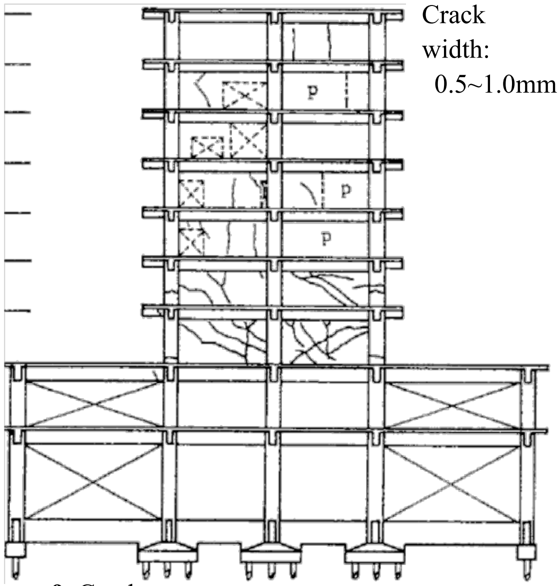


Figure 9. Crack patterns after1978Miyagi Oki earthquake (Shiga et al 1981)

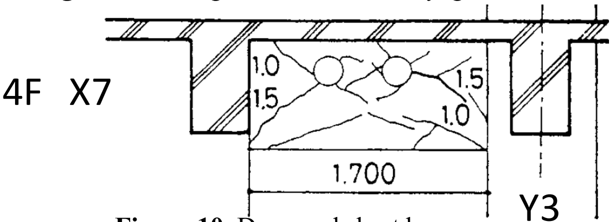


Figure 10. Damaged short beams (Shiga et al 1981)

3.2 2011 Great East Japan earthquake

The authors carried out detailed damage surveys of the building after the 2011 earthquake. The most severe damage occurred in the 3rd story. Four corner boundary columns in exterior walls X3 and X8 (which were intervened in 2001) fractured as shown in Figure 12a). A close-up of the disintegrated near their bases, and the steel angles and reinforcing bars embedded in them either buckled on the base of column X8-Y2 is shown in Figure 11.

Figure 12 shows cracks observed on the exterior walls in the transverse direction. A horizontal separation was formed at the base of the third story, between the post-installed structural wall panels (shown in Figure 12b)) and the beam supporting them. The post-installed anchors that were supposed to prevent this separation appeared to have been pulled out of the beam as shown in the close-up in Figure 12c) and Figure 13.

Along axes X4 to X7, shear cracks with widths of up to 1.7mm were observed on “wing walls” and “stems” forming the staircases in all stories. Spalling of concrete and a large horizontal crack were observed at the 3rd floor level (Figure 12d), e)). The damage and cracks of the wing walls indicate the interior C-shape walls remained integral and were effective in resisting lateral loads.

Though the bases of exterior boundary columns disintegrated, the webs of the exterior walls had little (or imperceptible) cracking in the 2nd to 3rd stories, and thin cracks (0.1~0.2mm) elsewhere. Comparing the damage caused to these walls by the Earthquakes of 2011 (Figure 12) and 1978 (Figure 9) suggests that there was a radical change in the mechanism through which these walls resisted lateral loads. It seems that in 1978 the exterior walls acted as units (composed of webs and columns connected to integrally), while in 2011 the webs did not seem to contribute much to lateral resistance. It is assumed that the buckling of the steel triggered the disintegration of the concrete at column bases, because the exterior columns along axis Y3 exhibited bar buckling and spalling of concrete despite the fact they resisted with no compression at peak displacements.

Figure 11. X8-Y2 column after remove the crush concrete

Figure 13. Zoom viewing of wall anchor

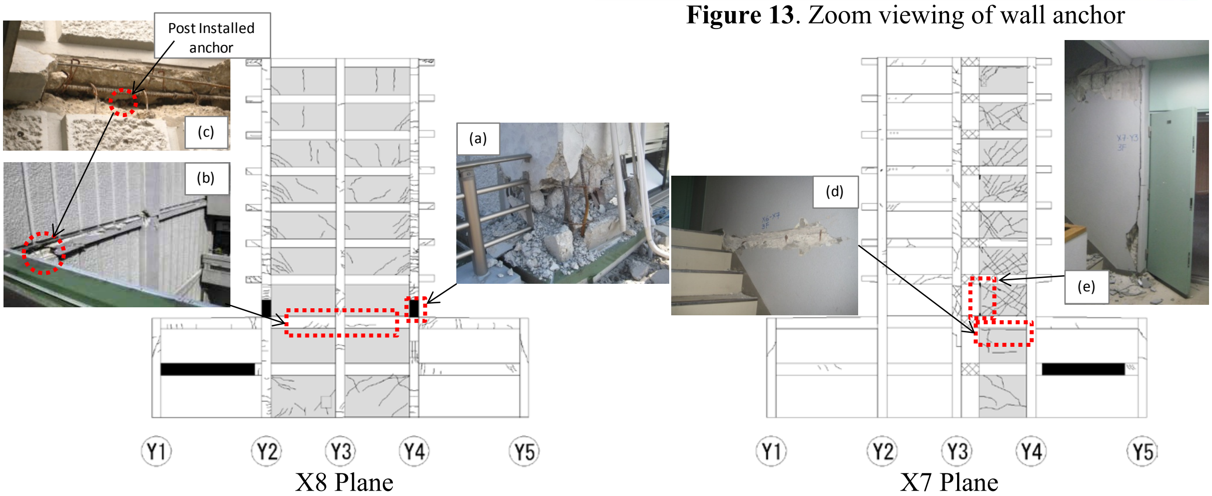


Figure 12. Crack patterns after 2011Great East Japan earthquake

4 OBSERVED EARTHQUAKE RECORDS AND BUILDING RESPONSE

4.1 Acceleration response spectrum

Figure 14 shows acceleration response spectrum computed for both records (1978 and 2011) using a damping coefficient of 5%. In the NS direction, the two spectra are strikingly similar. The same is true for displacement spectra. It is reasonable to conclude that these two earthquakes produced comparable demands on the building in the NS direction.

However, even though the demands appear to have been similar (according to two separate pieces of evidence), the observed damage was completely different in 1978 and 2011, especially in NS direction.

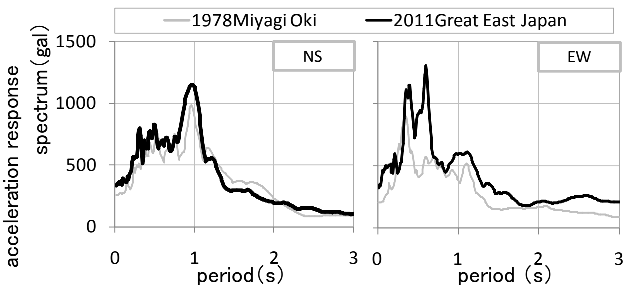


Figure 14. 5% damping acceleration response spectrum

4.2 Acceleration time history

The building has experienced many previous earthquakes. Table 2 shows accelerations during some of previous major earthquakes. The largest one is Miyagi Oki earthquake 1978 (Magnitude 7.4) and 2011 Great East Japan earthquake (Magnitude 9.0).

The maximum observed accelerations on the 1st floor in 1978 Miyagi Oki earthquake and 2011 Great East Japan earthquake were 258gal and 333gal, respectively. The maximum observed acceleration on the 9th floor were 1040gal and 908gal, respectively.

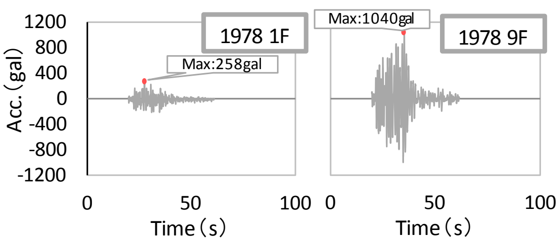
Comparing the two seismic records, one of the main characteristics of 2011 Great East Japan earthquake is long duration of about 180 seconds. Comparison of acceleration time history on 1st and 9th floor of the 1978 and 2011 earthquakes is shown in Figure 15.

Figure 16 shows the partial time history of 9th floor acceleration obtained in 1978 and 2011 around the maximum response. The waveform is just smooth because this building dominates the 1st mode in 1978 Miyagi Oki earthquake. Also, the response of 9th floor is stable after the maximum displacement at (4).

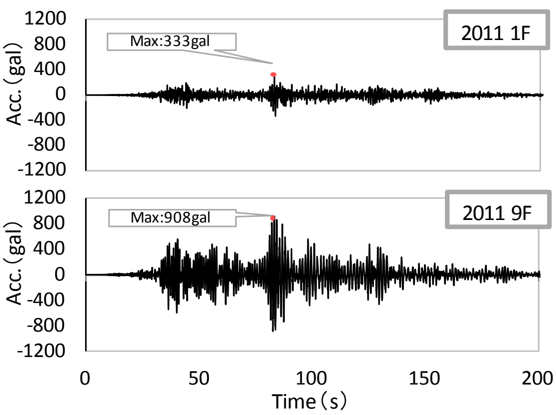
Similarly, the waveform before maximum response((1)',(2)') in 2011Great East Japan earthquake is quite smooth. However, after the maximum response((4)'~(6)'), the waveform shows irregularity such as sudden drop and unstable.

Table 2. Observed earthquake(PGA over 150gal)

y/m/d	Magnitude	1F maximum Acc.(gal)	9F maximum Acc.(gal)
1978/2/20	6.7	170	421
1978/6/12 (Miyagi Oki)	7.4	258	1040
1998/9/15	5.2	451	379
2011/3/11(East Japan)	9.0	333	908

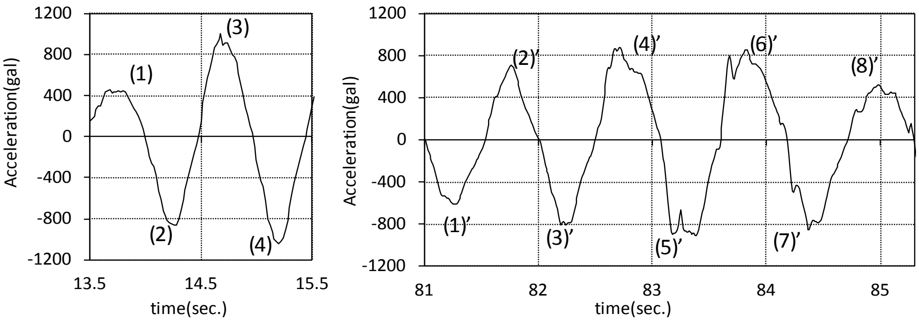


a) 1978Miyagi Oki



b) 2011Great East Japan

Figure15. Observed acceleration time history at NS direction



a)1978Miyagi Oki b) 2011Great East Japan

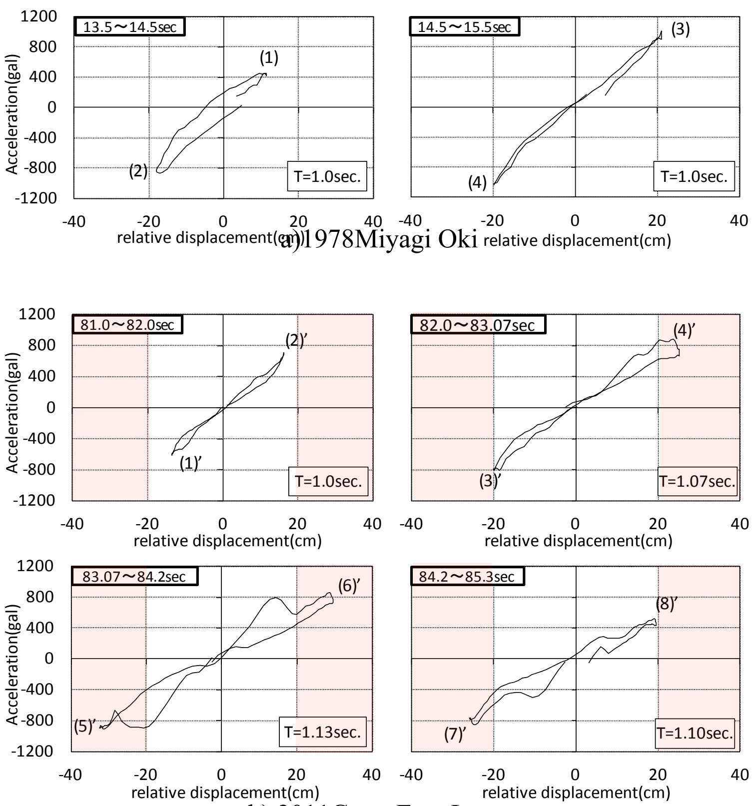
Figure 16. Zoom viewing of acceleration time history

4.3 Dynamic hysteresis loop

Figure 17 shows the dynamic hysteresis loop of the building. The vertical axis denotes the absolute acceleration of 9th floor, and the horizontal axis denotes the relative displacement of the building calculated by double integration of observed base acceleration.

In case of 1978 Miyagi Oki earthquake(Figure 17a)), the hysteresis loop from 13.5sec. to 14.5 sec. is fat loop having the large hysteresis area. However, the hysteresis loop form 14.5sec. to 15.5sec., when the maximum displacement 20.0cm was observed , is similar to linear elastic behavior without remarkable energy dissipation.

On the other hand, Figure 17b) shows the dynamic hysteresis loop of 2011 Great East Japan earthquake. The hysteresis loop before the maximum displacement of 1978 Miyagi Oki earthquake draw the inverse "S" shaped loop, which is similar to 1978 Miyagi Oki earthquake. However, the acceleration reached the ceiling about 800gal at peak (3)' and sudden dropped at peak (4)' to (6)'.



a)1978Miyagi Oki

b) 2011Great East Japan

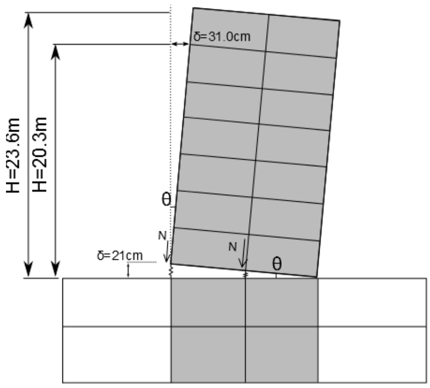
Figure 17. Dynamic hysteresis loop

After that, the hysteresis loop drew unstable and origin oriented shape. Also at the sa,me time, the relative displacement exceeded the maximum displacement, 20.0cm, observed in 1978 Miyagi Oki earthquake. From these facts, it is assumed that boundary columns fractured at peak (4)' to (6)' where the displacement exceeds the experienced maximum response displacement.

5 ESTIMATINON OF THE RESPONSE USING CAPACITY SPECTRUM METHOD

5.1 Estimation of the failure mechanism

The mechanisms of failure for the main structural elements (the walls) were dominated by flexure. Eventually, the ground motion caused the steel angles and reinforcing bars in boundary columns in axes X3 and X8 to buckle and fracture, and the anchor rods meant to fasten the webs to the beams were pulled out. At that point, the response of the exterior walls approached the response of a rocking block. This rocking behavior was inferred by Motosaka (2012) from wave analysis. If the lateral displacement on the 9th floor reached 31 cm, the uplift must have approached 21 cm . Walls and frames along column lines X4 to X7 are not suspected to have uplifted because their vertical reinforcement was continuous and well anchored.



X3 and X8

Figure 18. collapse mechanism

5.2 Result of the estimated response

Two-dimensional push-over analyses were conducted using the computer program SNAP (version6, 2012). The analyses referred exclusively to the transverse direction. The structure is modelled from 3rd to 9th story. The 1st and 2nd story were assumed stiff enough with many walls and were assumed as the base of the upper stories. Two cases were considered; 1978 model and retrofitted model of 2011. The shear versus displacement relation of each story was converted to a single degree of freedom system expressed in spectral acceleration and displacement (Sa-Sd) relations using the procedure of the performance based method used in Japan(AIJ 2009). Eq.(4), Eq.(5), Eq.(6) and Figure 19 shows the concept of conversion to SDOF.

$$M = \frac{\left(\sum_{i=1}^{N} m_i.\delta_i\right)^2}{\sum_{i=1}^{N} m_i.\delta_i^2} \qquad (4)$$

$$S_a = \frac{Q_1}{M} \qquad (5)$$

$$S_d = \frac{\sum_{i=1}^{N} m_i.\delta_i^2}{\sum_{i=1}^{N} m_i.\delta_i} \qquad (6)$$

where: m_i=lumped mass in the i-th story, Q_1=base shear and δ_i=horizontal displacement at i-th story

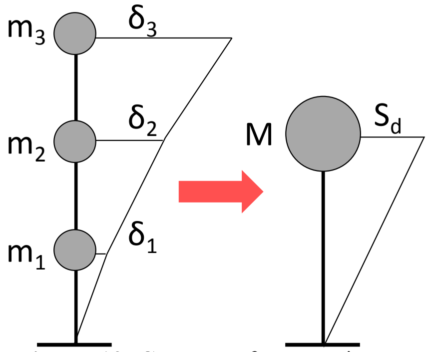
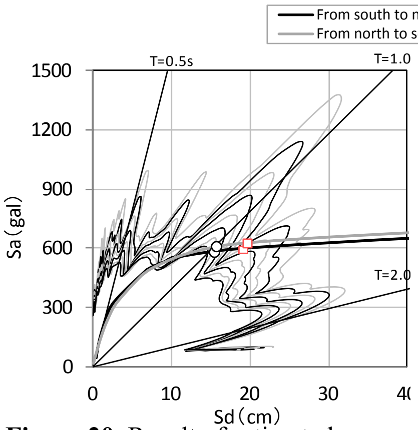


Figure 19. Concept of conversion to SDOF

The capacity spectra (Sa-Sd) of building are plotted with response spectra of the 1978 earthquake and 2011 earthquake with 3% to 10% damping. (see Figure 20 and 21). The damping to estimate the response is calculated by Eq.(7).

$$h = h_e + 0.2 \times (1 - 1/\sqrt{\mu}) \qquad (7)$$

where: h_e=elastic damping factor, μ=ductility factor of the building



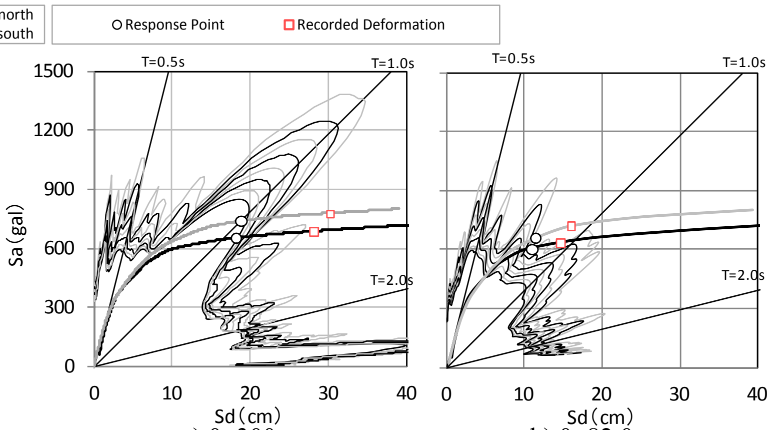


Figure 20. Result of estimated building response (1978 Miyagi Oki)

a) 0~300sec. b) 0~82.0sec.

Figure 21. Result of estimated building response (2011 Great East Japan)

The elastic damping factor is 0.03 according to experiment of Man-power excitation by Fujihashi(Fujihashi 1996).

The maximum horizontal displacement was evaluated by double integration of acceleration records observed on the 9th floor. The maximum recorded displacements at the 9th floor for the two earthquake cases (1978 EQ and 2011 EQ) are converted to the spectral displacement Sd and plotted in Figure 20 and 21 employing the assumed equivalent height of the SDOF. Specifically, the 9th floor recorded displacement is converted to the equivalent height displacement by assuming the distribution of displacement as inverted triangle from 3rd to 9th(see Figure 22) and the equivalent height is assumed at 0.8H (=18.8m).

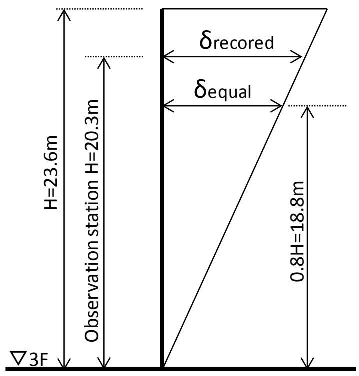


Figure 22. Displacement at equivalent height

The estimation results of response displacement and equivalent period agrees well with observed value on the case of 1978 Miyagi Oki earthquake(see Figure 20). However, there is big difference on the case of 2011 Great East Japan earthquake. As described at the Chapter 3, failure pattern of the building in 2011 was completely different from those in 1978, namely post installed anchors in wall panels pulled out and boundary columns were crushed.

As described at the Capter4, fracture of column is consider to occur around 82.0sec. Therefore structural characteristics are considered to change, and the acceleration records could be divided into 2 parts at the time 82.0sec. Then, the response analysis from 0sec. to 82.0sec. at 2011Great East Japan earthquake was carried out. Thereby, estimated response displacement and equivalent period are in good agreement with observed one. These almost have similar accuracy with the case of 1978 Miyagi Oki earthquake. Accordingly, the rocking movement from 3rd to 9th story by the fracture of boundary columns causes the difference between estimated and observed response at 2011 Great East Japan earthquake.

6 CONCLUSIONS

1) CE building at Tohoku university showed good performance for saving human lives on 2011 Great East Japan earthquake
2) Insufficient development and pull out of the vertical reinforcement in a shear wall panel caused concentrations of rotation at the 3rd floor level which led to buckling and the fracture of the vertical reinforcement in exterior wall boundary elements. This damage resulted in the inevitable evacuation and demolition of the structure.
3) The waveform of 9th floor acceleration got unstable and suddenly dropped after reaching the maximum response in 2011 Great East Japan. These facts mean that the boundary columns were fractured.
4) Discrepancy between estimated and observed response in 2011 is larger than that in 1978. However, estimated response on 2011 Great East Japan during the time 0~82.0sec have similar accuracy with that of 1978 Miyagi Oki earthquake. Accordingly, the rocking movement from 3rd to 9th story by the fracture of boundary columns causes the difference between estimated and observed response at 2011 Great East Japan earthquake.

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