10CUEE CONFERENCE PROCEEDINGS
10th International Conference on Urban Earthquake Engineering
March 1-2, 2013, Tokyo Institute of Technology, Tokyo, Japan

PERFORMANCE OF THE BUILDING OF THE FACULTY OF ENGINEERING AT TOHOKU UNIVERSITY DURING THE GREAT EAST JAPAN EARTHQUAKE OF 2011

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Abstract: This paper presents a study of a retrofitted SRC building which was damaged by 2011 Great East Japan Earthquake. The building was severely damaged in 2011 Great East Japan earthquake. However, this building also suffered from minor damage due to the 1978 Miyagi Oki earthquake. In 2001, This building was seismically evaluated using Japanese seismic evaluation standard (JBDPA) and it was retrofitted based on this seismic evaluation. It was retrofitted by installing framed steel braces, replacement of shear walls and jacketing of adjacent beams with steel plates. Characteristics and crack Figures of the structural damage is presented. The actual damage observed was different from estimated damage of the seismic evaluation. Plausible explanation of such damage and its mechanism is presented, compared and discussed with the seismic evaluation results, virtual work method and 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 that were designed before1981in Miyagi Prefecture were evaluated, and buildings deemed to be vulnerable were retrofitted before the 2011 East Japan earthquake. Most retrofitted buildings performed well during the 2011 East Japan Earthquake. The retrofits helped to limit damage to buildings in the affected area. However, some of the retrofitted buildings suffered moderate or severe structural damage. One of the severely damaged building was the building of the Faculty of Engineering and 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 epicenter of earthquake are shown in Figure 1.

The CE building is a 9-story composite structure constructed in 1968. Accelerometers were installed in 1968 in the 1st and 9th floors by Building Research Institute (BRI). Acceleration records were obtained for a number of earthquakes including the 1978 Miyagi-ken 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 was used without disruptions until 2001, when the building went through

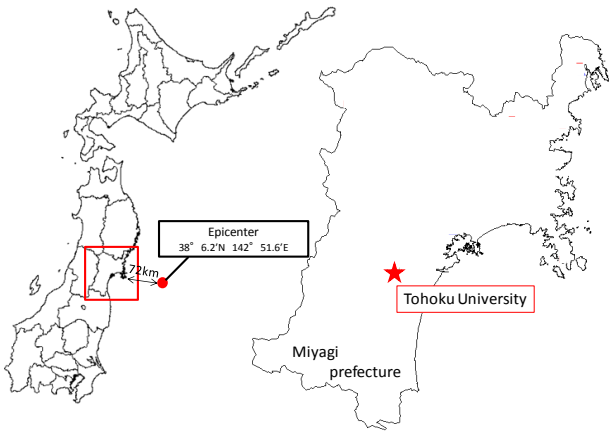


Figure 1 Location of Tohoku University

(a)from North (b)from East

Figure 2 General view of this building

major strengthening work.

In 2011, and despite the strengthening done in 2001, the Great East Japan Earthquake caused severe damage to the building leading to its evacuation and demolition. In this earthquake, acceleration records were obtained again on the 1st and 9th floor. This fact, and the availability of 1) previous records, 2) detailed accounts of the damage caused by previous earthquakes, and 3) complete construction and retrofit drawings make the building a special case attracting the attention of many researchers (Motosaka 2012, and Kimura et al 2012).

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. BUILDIN CONFIGRATION

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 building. The first two stories were larger in 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 3 b). The locations of the accelerometers are indicated by the stars in Figure 3.

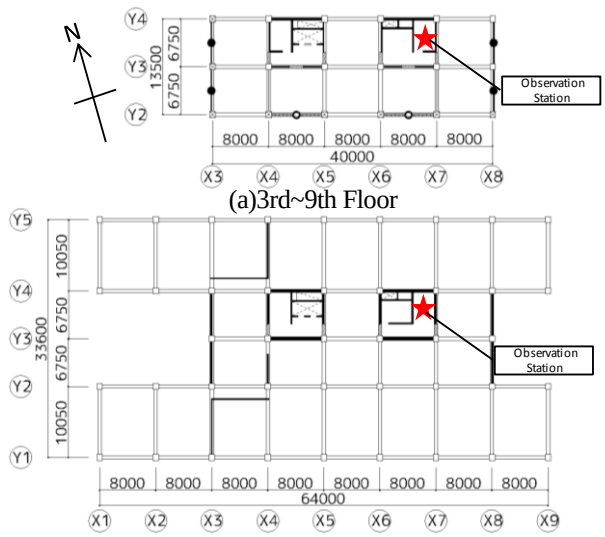
In the transverse direction, the main lateral-load resisting system consisted of shear walls along lines X3, X8, and the C-shaped shear walls of the staircases. As shown in Figure 3a), the structure was symmetrical about its transverse axis. Forces in the longitudinal direction were resisted by the staircase walls and two 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 by discrete horizontal steel plates. These plates did not form continuous webs or continuous flanges. Instead, they simply provided discrete supports for the vertical angles (Figure 20a)). These angles were embedded in concrete reinforced with twelve deformed vertical bars and widely spaced small-diameter ties made with 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

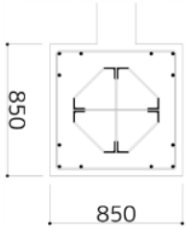
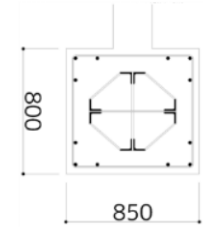


(a)3rd~9th Floor

(b) 1st&2nd Floor

Figure 3 Plan drawing

Table 1 Typical Column list

Floor	1F	2・3F	4・5・6F	7・8・9F
Dimension (mm)				
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			

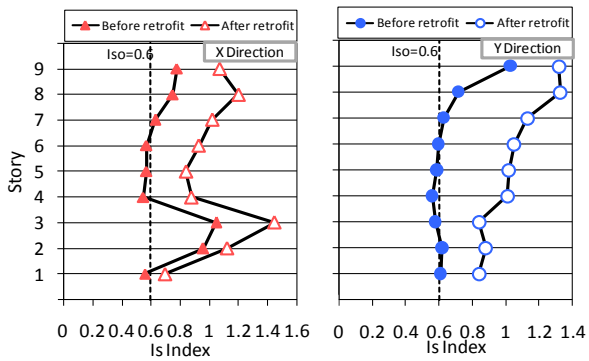
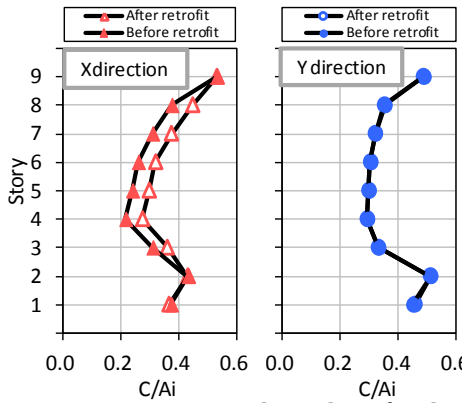


Figure 4 I_s index of each story



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 and Ai of each story

Disaster Prevention Association) standard, Japanese Standard for Seismic Capacity Evaluation of Existing Reinforced Concrete Building, (JBDPA 2001b) to the building in order to evaluate seismic capacity and to decide seismic retrofit scheme.

I_s-Index which represent seismic performance of 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).

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

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

C-Index is strength index that denotes the lateral strength of the buildings in terms of story shear coefficient which namely the 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).

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

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

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 story 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

Figure 6 shows the *C*-*F* curve of 3rd story. Both directions have *F*=3.5, and the building was stronger in X direction than in Y direction. In Y direction, the strength index is as low as 0.29.

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

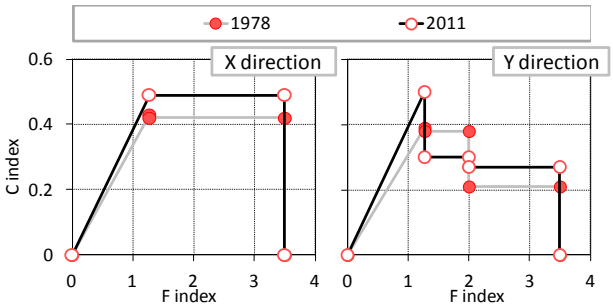


Figure 6 3F C-F curve

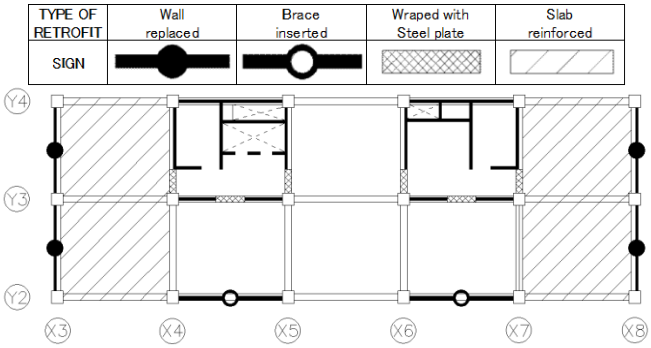


Figure 7 Part of retrofit

Table 2 List of replaced Wall

Floor	After retrofit			Before retrofit	
	Wall thickness (mm)	Arrangement of bar	Arrangement of anchor	Wall thickness (mm)	Arrangement of bar
9	180	D10・13−@200 S	D13−@100 S	150	9Φ @200 S
8					
7					
6					
5	200	D13−@200 D		200	9Φ @200 D
4					
3	250	D13−@150 D	D13−@150 D	250	

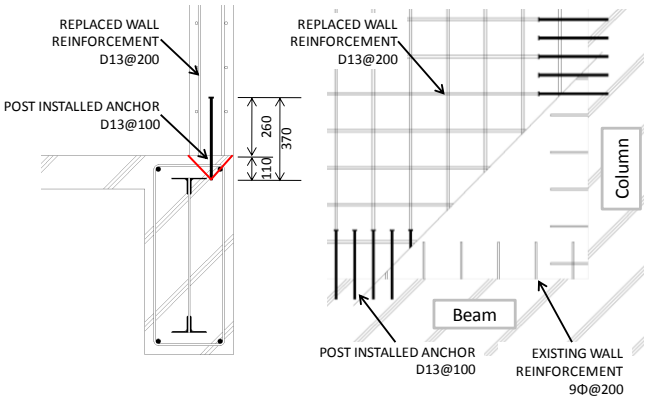


Figure8 Detail of replaced wall

steel braces in the longitudinal direction, replacement of RC shear walls in transverse direction (frame X3 and X8) from 3rd to 9th story, and jacketing of adjacent short beams with steel plates to avoid shear failure, as shown in Figure 7. The main scheme of the seismic retrofit was to increase I_s index (seismic performance 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 (which had cracks from previous earthquakes) with new exterior shear wall. However, the story shear coefficient *C*-index is not increased (0.3 or less) even after retrofitting.

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

The reinforcing details of the panels replaced in walls X3 and X8 are shown in Table 2 and Figure 8. These wall panels were replaced as follows:

1) the original concrete was removed with jack hammers,
2) 310-mm long adhesive anchors (D13 bars) with an embedment length of 110 mm were installed in beams and columns,
3) existing reinforcing bars (round 9-mm plain bars) were cut 200 mm away from beam and column faces,
4) new reinforcing bars (deformed 13-mm bars) were spliced with the adhesive anchors in both the longitudinal and horizontal directions.
5) concrete was cast in the wall panel.

It is important to emphasize that development length of post installed anchors were 110mm according design document 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 8.

3. OBSERVED DAMAGES

3.1 Earthquake history

The building has experienced many earthquakes. Table 3 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.

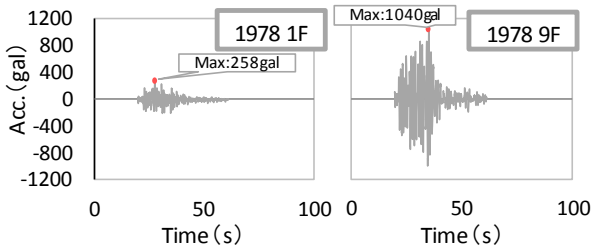
Comparing the two seismic records, one of the main characteristics of 2011 Great East Japan earthquake is long duration of about 180 seconds. Comparison of acceleration time history on 1st and 9th floor of the 1978 and 2011 earthquakes is shown in Figure 9.

Figure 10 shows the Displacement time history, and Figure 11 zooms into the record obtained in 2011 at times between 80 and 88 sec. The Figure has two interesting features:

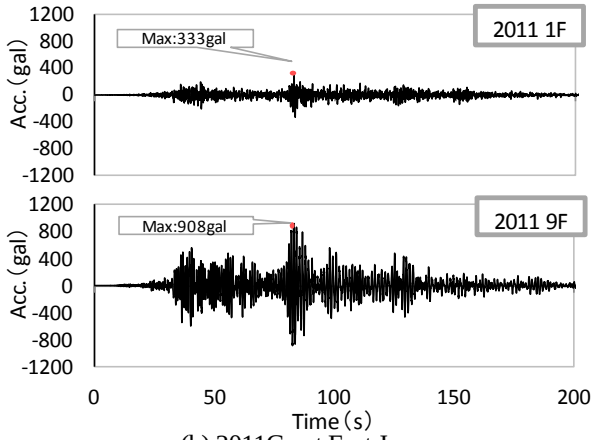
1) the period of the building was close to T=1 sec. before t=82.7 sec., and T=1.2 sec. after t=84 sec.

Table 3 Observed earthquake
(1st floor Acc. 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

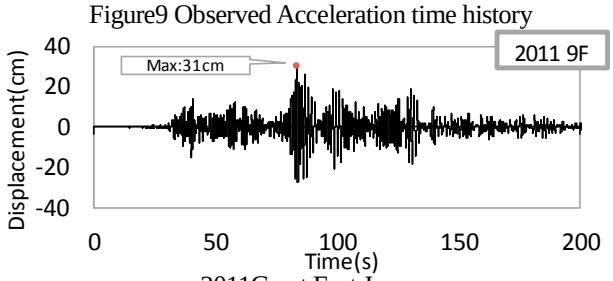


(a) 1978Miyagi Oki



(b) 2011Great East Japan

Figure9 Observed Acceleration time history



2011Great East Japan

Figure10 Observed Displacement time history

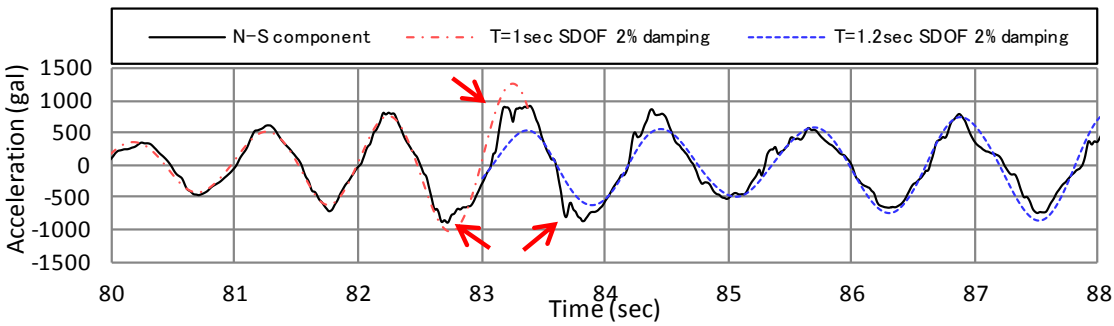


Figure 11 Zoom viewing of Acceleration time history

2) there were sudden drops in acceleration (marked with red arrows) at t=82.7, 83.3, and 83.8 sec.

These features may be evidence suggesting that something of consequence occurred in the structure as time approached and exceeded 83 sec. Integrating the record from 2011 until t=82.7 sec. leads us to believe that, up to that point, the displacement at the ninth floor had not exceeded 20 to 25 cm. Later the building may have reached displacements approaching 30 to 35 cm. Of course, we express our estimates as ranges because the integration is not a simple procedure and it required us to "adjust" the signal (by filtering it and shifting it) before we obtained results that we judged to be reasonable.

Figure 12 shows acceleration response spectra 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 N-S direction. This conclusion is supported by two observations:

1) The maximum displacement reached before t=82.7 sec. in 2011 was inferred not to have exceeded 20 to 25 cm. After t = 82.7 sec. the building exhibited abrupt softening and su dden drops in lateral acceleration indicating the possibility of local failures.
2) The maximum displacement inferred from the 1978 record was approximately 20 cm.

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 N-S direction.

3.2 1978 Miyagi Oki earthquake

Detail 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 13), adjacent beams and a few columns on the third and fourth floor. Typical wall and short beams crack patterns are shown in Figure14 and 15. 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 June earthquake, was considered to be fairly minor. As can be seen in Figure 15, exterior shear wall successfully sustained lateral force and contributed as shear resisting elements, because shear cracks were relatively uniformly distributed in the wall panels.

3.3 2011 Great East Japan Earthquake

The authors carried out detailed damage surveys of the building after the 2011 earthquake. The most severe damage took place in the 3rd story. Four corner boundary columns in exterior walls X3 and X8 (which were intervened in 2001)

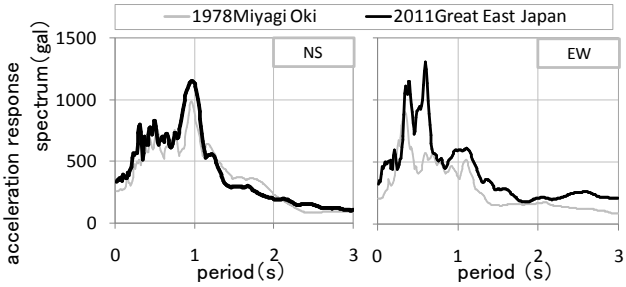
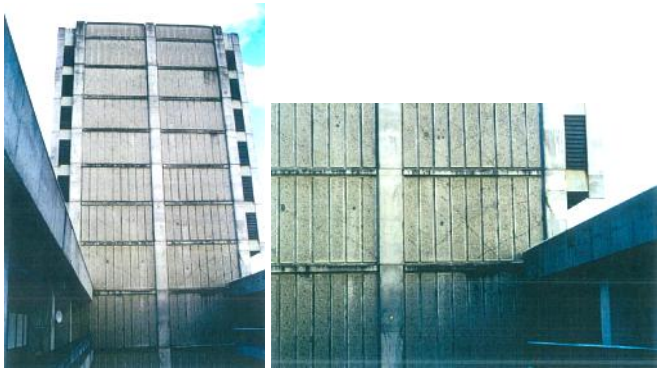
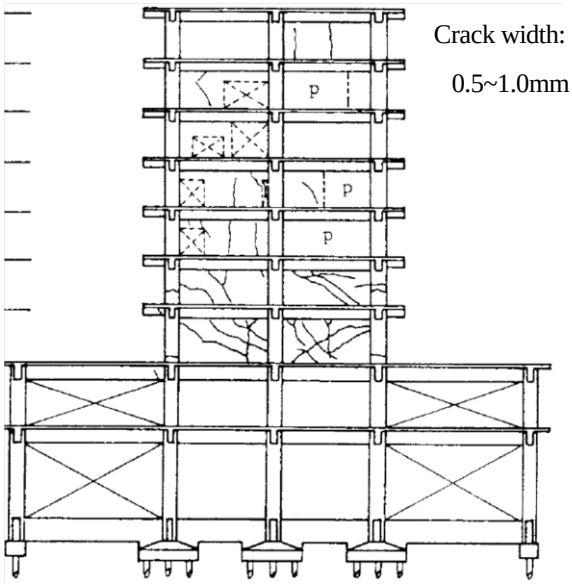


Figure 12 5% damping acceleration response spectrum



(a)All (b) 3rd Floor

Figure 13 Damaged wall in 1978 Miyagi Oki



X8 axis

Figure 14 1978Miyagi Oki earthquake crack map (Shiga et al 1981)

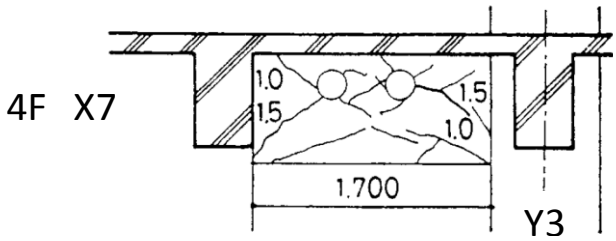


Figure 15 Damaged short beams

fractured as shown in Figures 16 and 18a). A close-up of the disintegrated near their bases, and the steel angles and reinforcing bars embedded in them either buckled or base of column X8-Y2 is shown in Figure 17.

Figure 18 shows cracks observed on the exterior walls in the transverse direction. A horizontal separation formed at the base of the third story, between the post-installed structural wall panels (shown in Figure 18b)) and the beam supporting them. The post-installed anchors that were supposed to prevent this separation appeared to have pulled out of the beam as shown in the close-up in Figure 18c) and Figure 19.

Along axes X4 to X7, shear cracks with widths of up to 1.7mm were observed on "wing walls" or "stems" forming the staircases in all stories. Spalling of concrete and a large horizontal crack were also observed at the 3rd floor level (Figure 18d), e)). The damage and cracks of the wing walls indicate the interior C-shape walls remained integral and were effective in resisting lateral loads.

In the longitudinal direction, the damage concentrated on the walls, which had cracks of 0.2~0.5mm in width as shown in Figure 20. These walls also remained integral and were effective in resisting lateral loads. Additionally, there were horizontal cracks at the 3rd story.

Even 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 18) and 1978 (Figure 14) 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. This inference and the followings facts:

1) the demands in 1978 and 2011were similar,
2) the steel braces installed in axis Y2 must have reduced problems related to torsion,
3) anchor rods installed in 2001 were observed to have pulled out,

led us to the focus on the connection between wall webs and 3rd floor beams.

Figure17 X8-Y2 column after remove the crush concrete

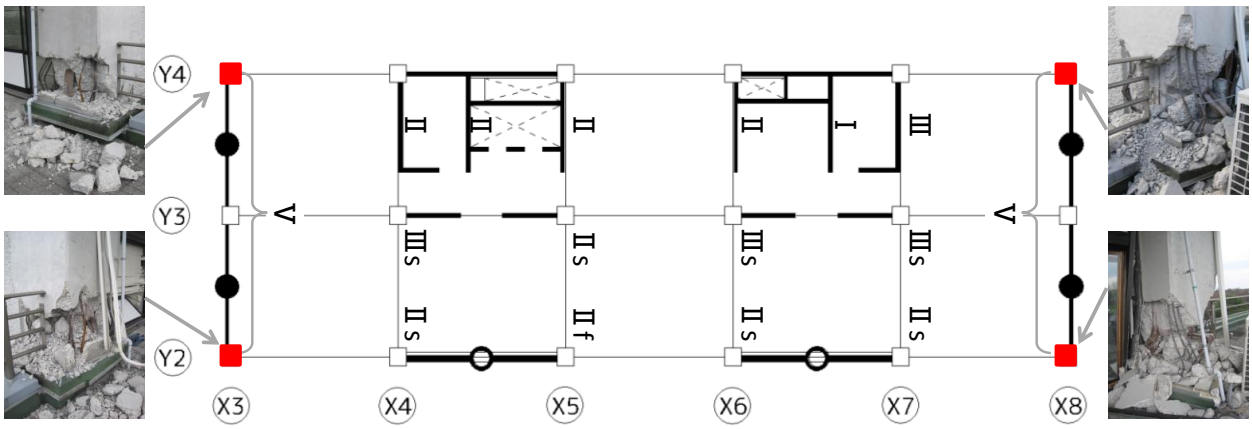


Figure16 Damage situation of 3F-Y direction

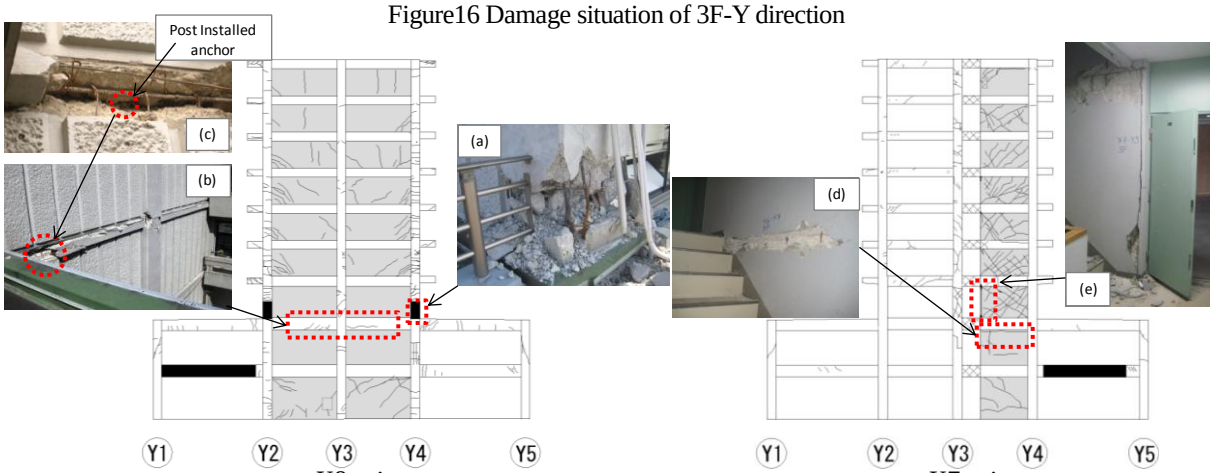


Figure 18 2011Great East Japan earthquake crack map (transverse)

Figure 21 illustrates a plausible explanation for what occurred:

1) The anchor bolts installed in the retrofit had an extremely short embedment length
2) The existing reinforcement was smooth, and was cut too close to the beam face to develop its strength through bond (gained mainly through adhesion to the concrete cast in 2001)
3) If one draws lines from the bottom of the anchor bolt in Fig. 8 to the points where the wall outer and inner faces meet the cold joint at the top of the slab, one obtains a "surface" that is not crossed by any reinforcement working in tension and adequately anchored.
4) The result of 1), 2) and 3) was that, at the tops of the slabs and at every floor, the vertical web reinforcement was essentially discontinuous.
5) The discontinuity in the web reinforcement caused rotations to concentrate at the top of the third-floor slab causing the steel angles and reinforcing bars in the boundary elements to reach large tensile strains.(see Figure 19) These strains led to either fracture or buckling during load reversals.

It is believed that it was the buckling of the steel what triggered the disintegration of the concrete at column bases because even the exterior columns along axis Y3 exhibited bar buckling and spalling of concrete despite the fact they resisted no compression at peak displacements. Sliding could have damaged these columns too but no clear signs of sliding at wall bases were observed. If compression had been the trigger, we would have expected severe spalling and crushing (similar to what was observed at columns in axes X3 and X8) in the stems of the C-shaped walls that formed the staircases. That did not occur. The stems of the staircase walls did have some spalling but it was limited to the concrete cover.

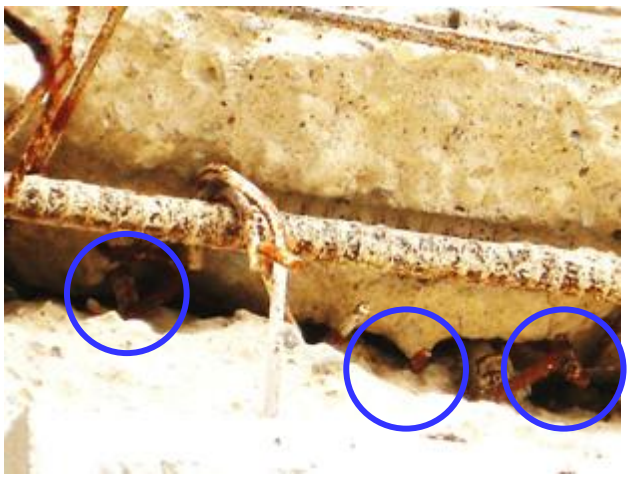
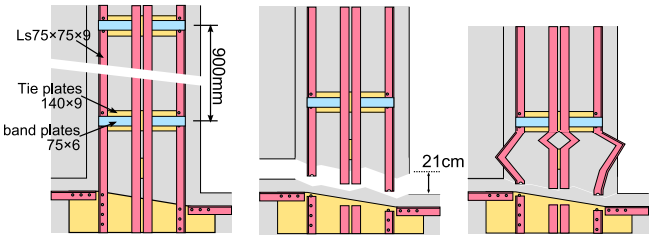
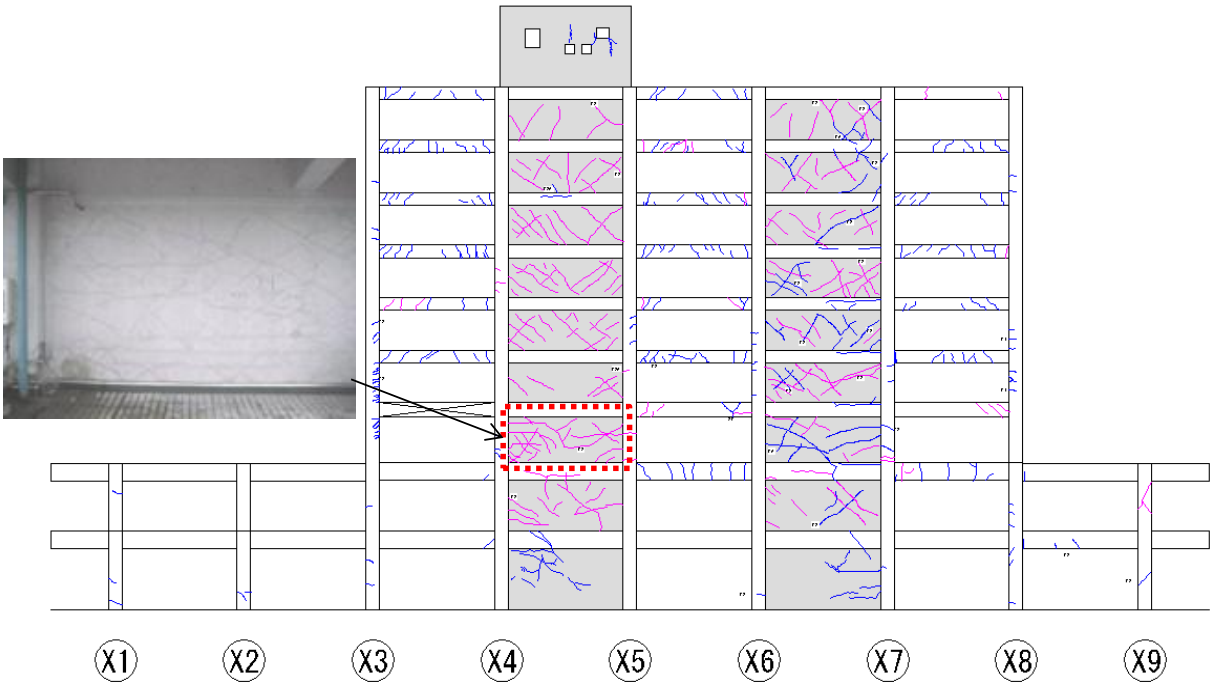


Figure 19 Zoom viewing of wall anchor



(a) (b) (c)

Figure 21 Mechanism of Column crushed



Y4 axis

Figure 20 2011Great East Japan Earthquake crack map(longitudinal)

4. EVALUATION OF SEISMIC CAPACITY AND FAILURE MECHANISM

4.1 Estimate 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 ninth 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.

4.2 Lateral strength using virtual work method

Lateral capacity of the frames in the transverse direction was checked using virtual work method. The following assumptions were considered:

・1st to 2nd stories are assumed to be stiff and strong, and acted as the base of the upper part of the building.

Assumption through X3・X8 axes

- The wall will break and rotate at the base of 3rd floor (see Figure 22a)).
- The strength of the wall reinforcement are assigned in the center of wall.

Assumption through X4 to X7 axes

- The seismic lateral forces were applied in different direction (from North to South and from South to North) because the wing walls strength will be different.

Two different cases are considered: Case 1 considers the exterior installed wall effective and behaved exactly as it was expected in the seismic evaluation . In Case 2, the steel in the boundary column is ignored and the wall reinforcement is 50% ineffective because the boundary column have already yielded and wall reinforcement were easily pulled out. Table1 shows the different assumptions of each case.

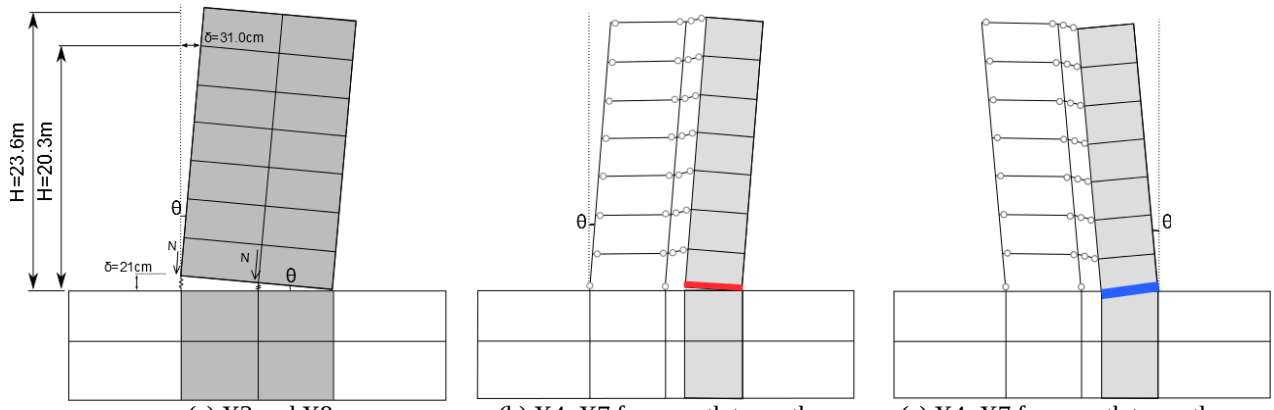
As it shown in the Figure 23. The difference between the two cases is more than 30% of the total strength. This building depends mainly on these exterior installed walls. Any poor detailing of the installed walls shall led to dramatic damage.

4.3 Capacity spectrum method

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 modeled from 3rd~9th story. The 1st and 2nd story are very stiff with many walls and were assumed as the base of the upper stories. 2 cases were considered; 1978 model and retrofitted model of 2011. The shear versus

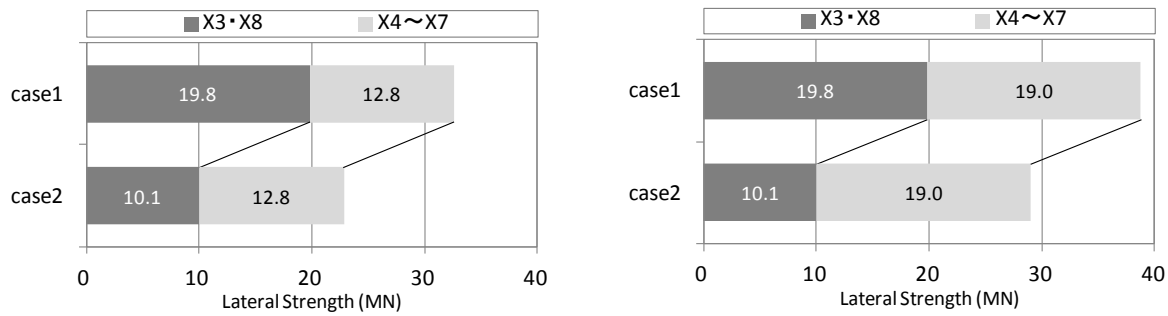
Table 4 Assumption of Virtual work method

Model ID	AxesX3 and X8		
	Location of Wall Hinges	Vert.column Steel	Vert.Web Steel
case1	3rd Story Base	Continuous	Continuous
case2		Ignored	50% ineffective



(a) X3 and X8 (b) X4~X7 from south to north (c) X4~X7 from north to south

Figure 22 Estimated building collapse mechanism



(a) from South to North (b)from North to South

Figure 23 Lateral strength at 3rd floor

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(JABRP 2000). Eq.(4), Eq.(5), Eq.(6) and Figure 24 shows the concept of conversion to SDOF.

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

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

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

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

The capacity spectra (Sa-Sd) of building are plotted with response spectra of the 1978 earthquake and 2011 earthquake with 5%~20% damping. (see Figure 25).

The dotted horizontal line in Figures 25b) shows the lateral strength capacity that was previously calculated using the virtual work method for case 1 stated previously. The ultimate strength of pushover and the lateral strength by virtual work method of case 1 showed good agreement

The maximum horizontal displacement was evaluated by double integration of acceleration records observed on the 9th floor. The recorded maximum displacement at the 9th floor for the two earthquake cases (1978 EQ and 2011 EQ) is converted to the spectral displacement Sd and plotted in in Figure 24. using assumed equivalent height of the SDOF. Specifically, the 9th floor recorded deformation are converted to the equivalent height deformation by assuming the distribution of deformation as inverted triangle from 3rd~9th(see Figure26) and the equivalent height is assumed at 0.8H (=18.8m).

Using capacity spectrum method, in the case of 1978 earthquake, the estimated response drift was about 15 cm and the calculated displacement was about 23 cm, in the case of 2011 earthquake, the estimated response drift was about 18 cm and the calculated displacement was about 29 cm, each model have big difference between estimate and recorded deformation. The difference between the two values might be caused due to the rocking at the base and unique failure mechanism of this building.

However, further investigations involving different cases of structure models are needed to have better simulation of the actual damage, such as cases assuming the vertical web reinforcement of exterior transverse wall ineffective.

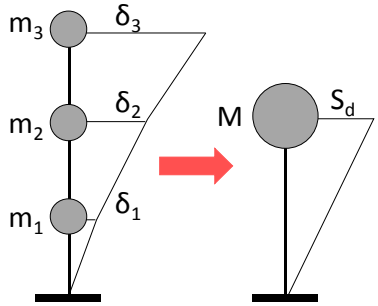


Figure 24 Concept of convert to SDOF

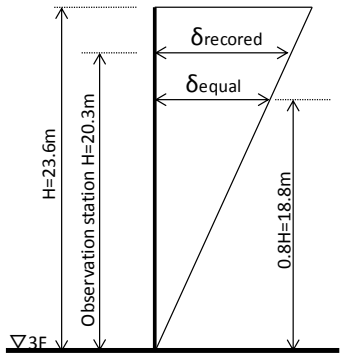
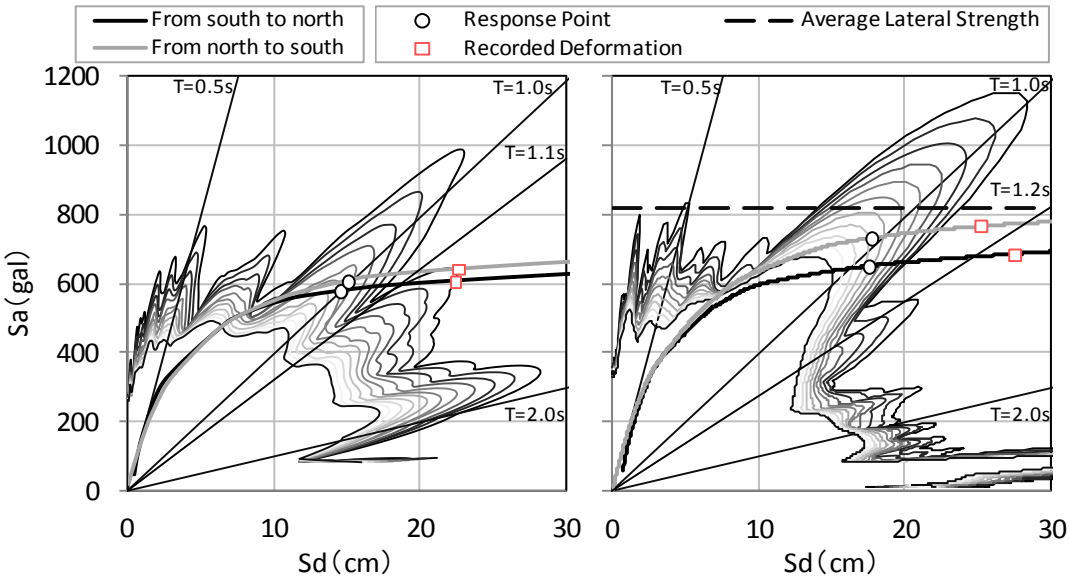


Figure 26 Deformation of equivalent height



(a) 1978 Miyagi Oki (b) 2011 Great East Japan

Figure 25 Result of the pushover analysis

5. CONCLUSION

Considering:

1. The similarities between the response spectra of the N-S components of the 2011 Great East Japan earthquake and the 1978 Miyagi earthquake. However both of damage is completely difficult.
2. The distribution of cracks and spalling throughout the building
3. Observations indicating that the vertical web reinforcement connecting walls to beams pulled out during the 2011 earthquake

We conclude that discontinuities in the vertical web reinforcement caused concentrations of rotation at the 3rd floor level which led to buckling and fracture of the vertical reinforcement in exterior wall boundary elements. This damage resulted in the evacuation and demolition of the structure.

In additional, the capacity spectrum method of both cases(1978 and 2011) couldn't estimate the actual displacement. This point needs further investigations and will be studied thoroughly in the near future.

6. FUTURE WORK

Experiments are being conducted to test the hypothesis that the discontinuity in the reinforcement at the base of the 3rd story led to the failures of the exterior columns.

The experimental program consists of six small-scale structural-wall models with identical cross-sectional properties. The web reinforcement in three specimens is cut off at the base of the wall, while in the other three specimens web reinforcement is well anchored in the foundation. Two specimens will be tested under static loading reversals and four specimens will be tested using an earthquake simulator at Bowen Laboratory, Purdue University.

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