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SHAKING TABLE TEST OF 6-STORY WALL FRAME BUILDING TO INVESTIGATE COLLAPSE PROCESS OF RC BUILDINGS

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Abstract

This paper describes the outline of experimental studies on collapse margin of reinforced concrete buildings. To reduce damages caused by large earthquakes, Special Project for Reducing Vulnerability for Urban Mega Earthquake Disasters has been conducted since 2012 by Ministry of Education, Culture, Sports, Science and Technology of Japan. In this special project, to quantify collapse margin of the buildings that are commonly seen in urban areas in Japan, shaking table test of a reinforced concrete building was conducted in *E-Defense*. The test specimen was a 1/3 scale model of a 6-story RC building designed according to the current Building Standard Law in Japan. The test specimen consisted of two moment-resisting frames in the longitudinal direction, and four frames with multi-story shear walls in the transverse direction, and the direction of main concern in this study was the latter. The two interior frames had shear walls and the others had nonstructural walls with openings from the second to the sixth stories. Recorded waves during the 1995 Hyogo-ken Nanbu Earthquake were used as the basic input motions. At the final shaking in the test, shear failure of walls at the first and second stories occurred, and the building specimen collapsed finally. As the test results wall failure mechanism and effect of the eccentricity were discussed. The maximum response capacity of the specimen in the transverse direction was precisely evaluated as the sum of shear strength of the walls and flexural strength of the columns.

Keywords: Shaking Table Test; E-Defense; Reinforced Concrete Building; Earthquake Resistant Wall

1. Introduction

In recent years, it has been feared that large earthquakes will occur in Japan. One of them is a subduction-zone earthquake along the Nankai Trough, and the others are Near-Field earthquakes predicted in Tokyo, Osaka, and Kumamoto. To mitigate damages to be caused by these earthquakes, the special project for Reducing Vulnerability for Urban Mega Earthquake Disasters has been conducted since 2012 by Ministry of Education, Culture, Sports, Science and Technology of Japan (MEXT)[1]. In the special project, sub-project No.2: Maintenance and Recovery of Functionality in Urban Infrastructures has two themes. One is quantification of collapse margin of building, and the other is development of systems for monitoring and prompt condition assessment. For the first theme, static loading tests of reinforced concrete members, a shaking table test of reinforced concrete building and analytical studies were conducted[2]. This paper describes the shaking table test carried out in *E-Defense*, Hyogo Earthquake Engineering Research Center, National Research Institute for Earth Science and Disaster Resilience.

2. Outline of the Test

2.1 Test Specimen

This research focuses on reinforced concrete buildings which are commonly seen in urban areas in Japan. Residential buildings made with reinforced concrete are targeted. Fig. 1 shows an appearance of the test specimen on the shaking table. Plan views and elevations are shown in Fig. 2 and 3, and typical sections are listed in Table 1.

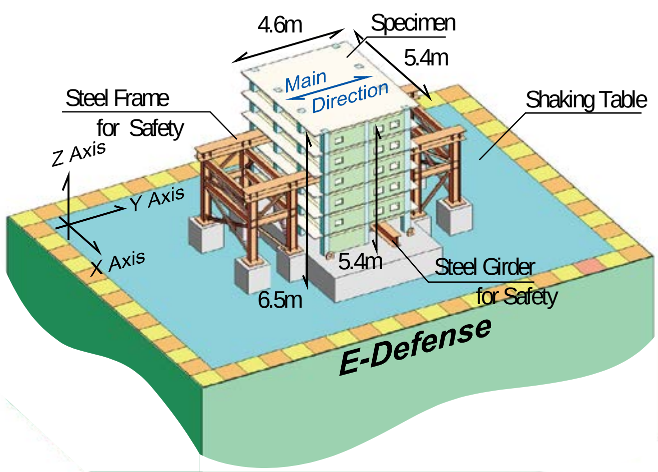


Fig. 1 – Perspective View of Specimen on Shaking Table

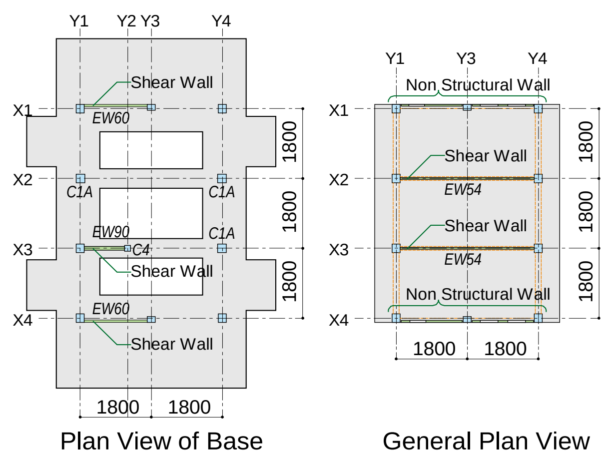


Fig. 2 – Plan Views of Specimen

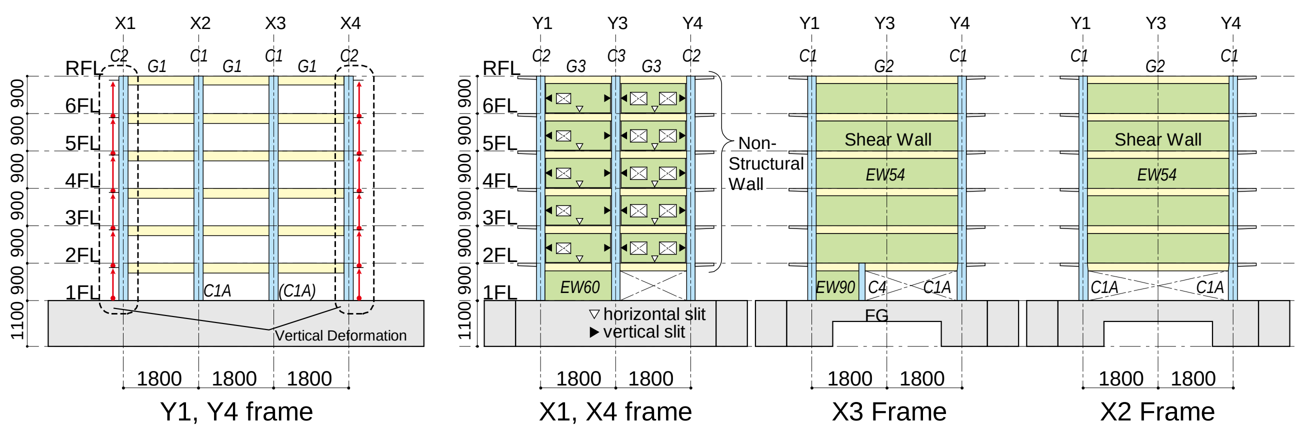


Fig. 3 – Elevations of Specimen

The test specimen was a 1/3-scale model of a 6-story reinforced concrete building designed according to the current Building Standard Law of Japan. The test specimen consisted of two moment-resisting frames in the longitudinal direction, and four frames with shear walls in the transverse direction. The direction of main concern in this study was the latter. In the first story, three shear walls were placed connected with Y1 frame. Although the first story has slight structural eccentricity in the transverse direction, the eccentricity index was smaller than the threshold value specified in the Building Standard Law of Japan, so an extra shear capacity was not considered. The two interior frames, X2 and X3 had shear walls from the second to the sixth stories continuously while X3 frame had a shorter wall in the first story. Because three columns *C1A* in the first story in X2 and X3 frames required high ductility, the cross sectional area was larger than that of *C1*, and the amount of hoops of *C1A* was larger than that of *C1* more than 1%. The two exterior frames, X1 and X4 had nonstructural walls with openings from the second to the sixth stories continuously while they had a shear wall in the first story. Nonstructural walls of the specimen had about 12mm gap along the column side vertically and along the bottom horizontally, and it was a hanging wall from the girder of the upper floor. The gap is called as structural slit. At the gap between frame and nonstructural wall, joint bars were arranged with the pitch of 275mm as shown in Fig. 4 and Photo 1, for the purpose of preventing an excessive displacement out-of the plane direction of the wall. Y1 and Y4 frames were moment-resisting frames with three spans. Y2 frame had only one column *C4* as side column of shear wall *EW90* in the first story. Cantilever slabs were placed along Y1 and Y4 frames as balconies and exterior corridors.

Table 1 – Typical List of Columns, Girders and Walls

	C1 (1F)	*C1A* (1F)	*C2* (1F)	*C3*	*C4*
Column	226 / 196	226 / 226	226 / 196	150 / 196	150 / 150
Main Bar	8-D10	14-D10	10-D10	12-D6	8-D10
Hoop	▥-D6@85	⊞-D6@55	▥-D6@85	□-D4@35	□-D6@85
	G1 (2F, 3F)	*G2* (2F)	*G3* (2F)	*G2* (3F)	*G3* (3F)
Girder	240 / 150	180 / 106	180 / 106	180 / 106	180 / 106
Main Bar	6/4-D10(top/btm)	8/8-D6(top/btm)	7/8-D6(top/btm)	2/2-D6(top/btm)	4/4-D6(top/btm)
Stirrup	2-D6@85	2-D4@75	2-D4@75	2-D4@75	2-D4@75

Wall	*EW90*	*EW60*
	75 / 75 / 90	170 / 170 / 60
	D4@75 ps=0.4%	D6@170 ps=0.6%
	EW54	*Slit*
	135 / 135 / 54	D4@ 27 / gap= 12mm
	D4@135 ps=0.4%	D4@275 ps=0.1%

D4, D6 and D10 are deformed bars and the diameter of them are 4mm, 6mm, and 10mm respectively

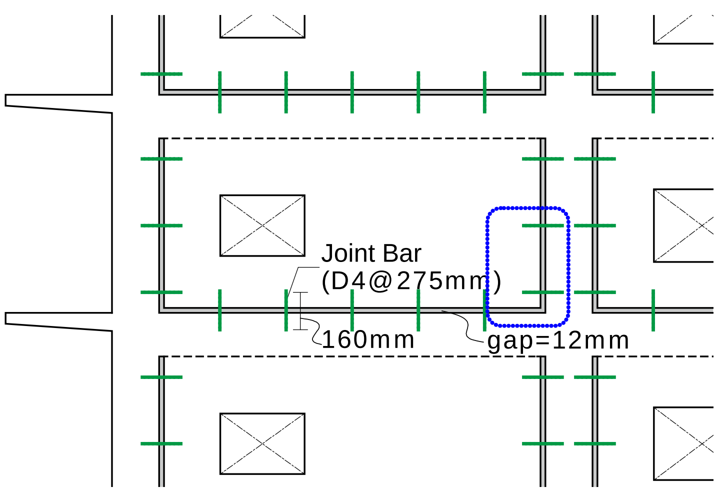


Fig. 4 – Gap and Joint Bar arrangement

Photo 1 – Gap and Joint Bar

To make axial stress of columns caused by dead and live loads corresponding to those of the original scale building, steel plates were placed under every floor. The dead and live loads were 11.6kN/m^2 for each floor. The weight of the specimen was 1837kN without the foundation and the steel frame for safety. The total weight of the specimen including the steel frame was 3190kN. The steel frame for safety was designed in case that specimen supposed to be collapsed. Two steel girders passed through the 4th story.

Material properties were listed in Table 2. The design compressive strength of concrete was 30N/mm^2. As shown in Table 2, normal strength steel reinforcements were used.

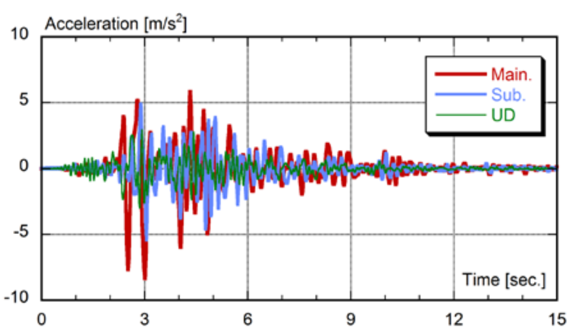
Table 2 – Material Properties

(a) Concrete

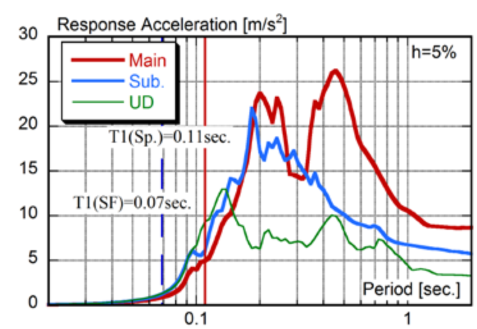
Member	Compressive Strength [N/mm^2]	Young's Modulus [N/mm^2]	Tensile Strength [N/mm^2]
1st – 6th story(Average)	42.4	3.07×10^4	3.33
Basement	98.9	4.34×10^4	4.22

(b) Reinforcement (deformed bars)

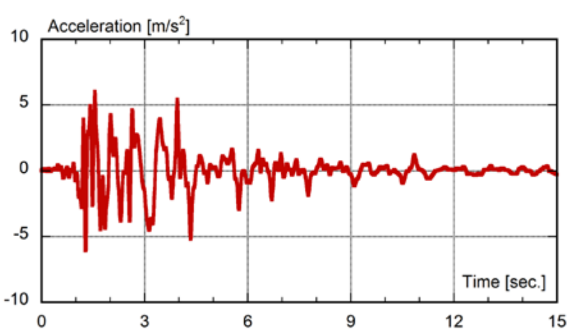
Diameter [mm]	Cross Sectional Area [mm^2]	Yield Strength [N/mm^2]	Tensile Strength [N/mm^2]	Young's Modulus [N/mm^2]	Members
4	14	364	524	1.76×10^5	Wall: EW54, W54, EW90 Shear Reinforcement
6	32	379	518	1.97×10^5	Wall: EW60, Shear and Slab Reinforcement, Longitudinal Bars of G2, G3 & C3
10	71	379	553	1.90×10^5	Longitudinal Bars of G1, C1, C2 & C4



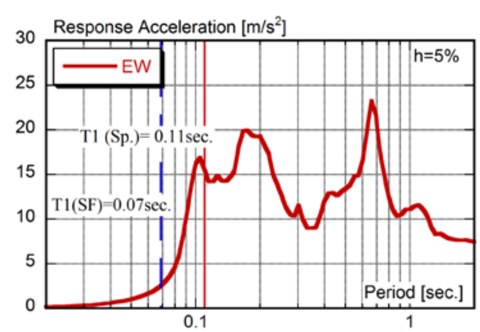
(a) Time history of JMA Kobe



(b) Response spectra of JMA Kobe



(c) Time history of JR Takatori



(d) Response spectrum of JR Takatori

Fig. 5 – Original Input Waves

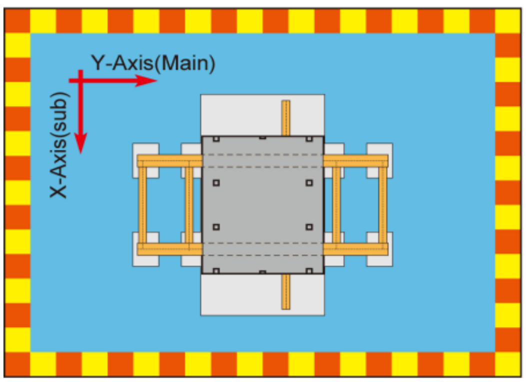
2.2 Input Waves and Instrumentation

Two waves were used as base input motions. They were recorded during the Hyogo-ken Nanbu Earthquake in 1995 at Japan Meteorological Agency Kobe, called JMA Kobe, and at Takatori station of West Japan Railway Company, called JR Takatori. The signals were contracted in time by a factor $\sqrt{0.30}$ based on the law of similitude as shown in Eq.(1). The time histories and response spectra of the input waves were shown in Fig. 5. The trace of horizontal acceleration orbit of the JMA Kobe wave was shown in Fig. 6. Horizontal components of JMA Kobe wave were rotated 135 degree to the east direction. The northwestward component of original wave was applied to Y axis of the specimen and the northeastward component was to X axis. The frequency components higher than 11Hz were attenuated by low pass filter to prevent the steel frame from being shaken excessively. The adopted acceleration amplitude scaling factors for JMA Kobe ranged from 10% to 140%. Following the JMA Kobe motions, the EW component of the JR Takatori motion scaled 120% was applied to force the building to collapse. In case of JMA Kobe 140%, only the NS component was applied.

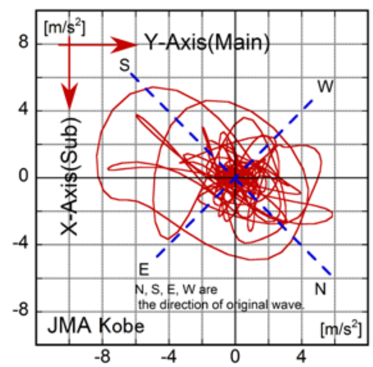
$$T_M = \sqrt{0.30} \cdot T_R \tag{1}$$

where, T_M, T_R: Time axis for Model and Real structure, respectively

Accelerometers were placed at four corners of each floor. Interstory drifts were recorded by laser displacement sensors, as shown in Fig. 7 and Photo 2. Axial deformations of four columns at the corners of each story were recorded by displacement transducers. In Fig. 3, red arrows represent their locations.



(a) Bird's-eye view of specimen on the table



(b) Trace of acceleration orbit

Fig. 6 – Trace of Acceleration Orbit of JMA Kobe Wave

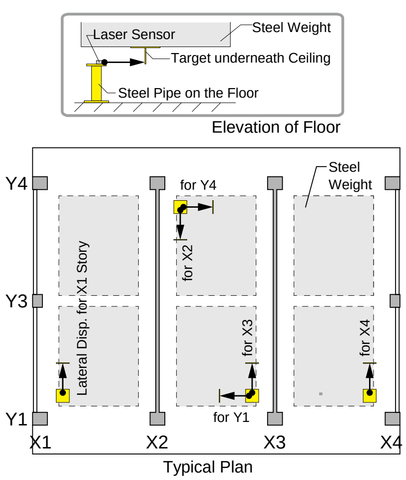


Fig. 7 – Lateral Displacement Measurement

Photo 2 – Laser Displacement Sensor

3. Test Results

3.1 Outline of the Test Results

Shaking table tests were carried out for three days. On the first day, the capacity response as large as the horizontal load-carrying capacity required by the design code had been expected to develop. On the second day, the same intensity of waves as the first day were applied. For the last day, there were two objectives; one was to observe maximum strength of the specimen, and the other one was to observe the specimen collapsing. Some lower waves than the target for each day were input to record the damage process such as cracking of concrete or yielding of reinforcement. At the end of the test each day, damage states such as cracking or crushing of concrete were observed and recorded by sketching. Preceding or following the shaking test of earthquake motions, small white noise wave were input to observe specimen's frequency.

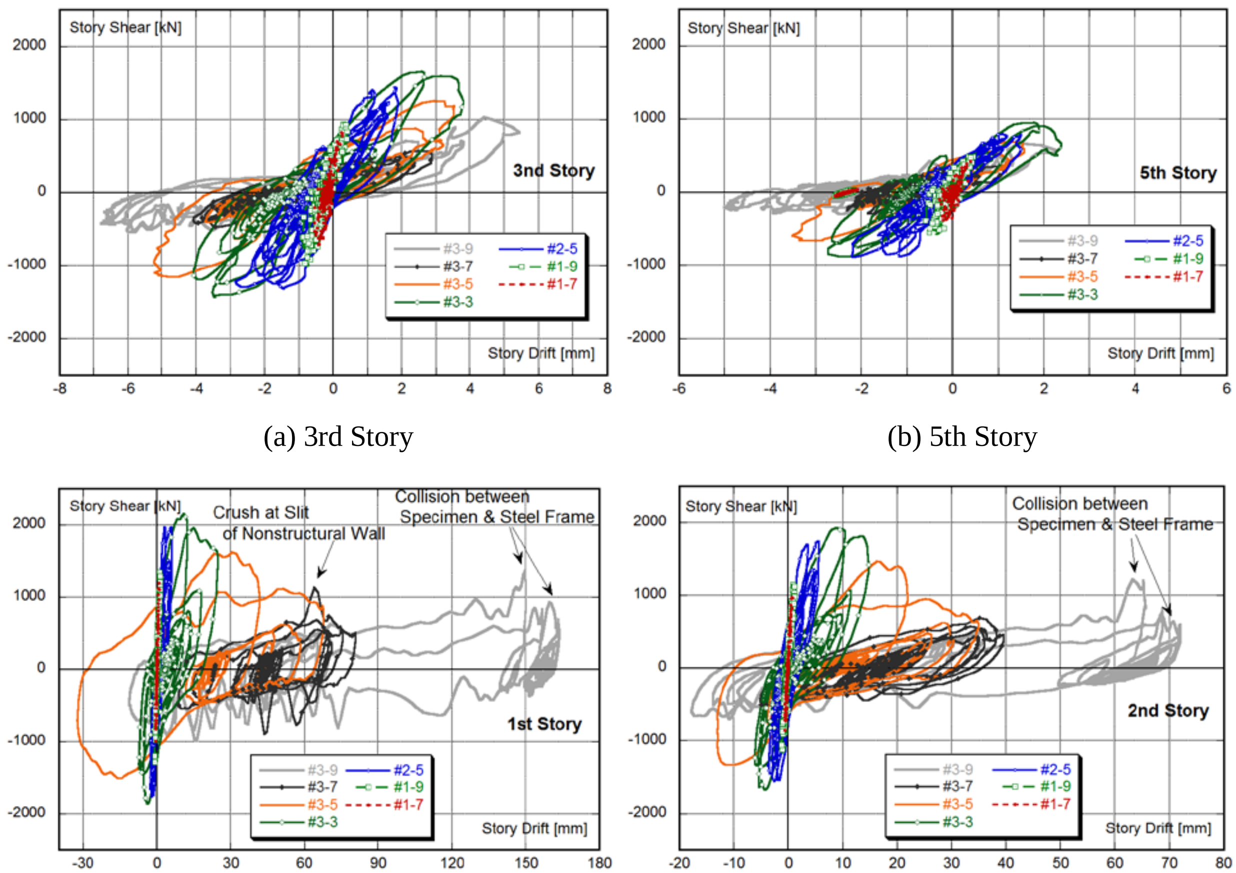
Table 3 summarizes the test results of the representative cases. In #1-7, the maximum base shear coefficient response was 0.66. Yielding of some column main bars was observed. In #1-9, some reinforcements of girders and walls yielded. In #2-5 of the second day, crushing of the concrete in the 1st story wall of X1 frame was observed. After the test, fracturing of the joint bars at the gap of nonstructural wall were observed on the 2nd and 3rd floors of X1 and X4 frames. In #3-3, maximum response base shear coefficient was recorded to be more than 1.0 against the 120% input wave of JMA Kobe. Though the input wave was 140% of JMA Kobe for both cases of #3-5 and #3-7, maximum response base shear was smaller than that of case #3-3. For the case #3-9, 120% of JR Takatori wave was input. In this case, the structural walls of the 1st and 2nd floors failed in shear, collision between the cantilever slab and the steel frame occurred, and the test was completed. Because of the collision, the natural period of #3-9 was shortened from test #3-7. The system was no more 6-Story RC building, but was RC building joined or affected by the steel frame.

Figure 8 shows story shear – interstory drift relationship. The interstory drift was summation of lateral displacement from the laser sensor and the flexural component calculated from vertical deformation as shown in Fig.9. The story shear was the lateral inertia force which was calculated by multiplying the acceleration response and the mass of each floor. In #3-7 or #3-9, spikes were shown due to the collisions. As shown in Fig.10, shear failure of the 1st floor walls caused vertical depression of Y1 frame much more than that of Y4 frame. It was supposed to cause residual story drift of negative direction at upper floors.

Table 3 – Test Results of Main Direction (Y Axis)

Case Number	Magni-fication	Acceleration[m/s^2] Target (Result)	Q_B[kN] (C_B) Main Direction	$R_{1\mathrm{Max}}$[rad.]	T[sec.]	Event
#1-3	10%	0.84 (0.76)	140.4(0.08)	1/12857	0.098	
#1-5	40%	3.34 (4.14)	769.1(0.42)	1/2500	0.104	cracking
#1-7	55%	4.60 (5.84)	1212.0(0.66)	1/882	0.112	rebar yielding (column main bar)
#1-9	70%	5.85 (6.71)	1341.6(0.73)	1/629	0.122	rebar yielding (girder and wall)
#2-3	70%	5.85 (5.99)	1342.0(0.73)	1/536	0.130	
#2-5	100%	8.36 (9.89)	1975.4(1.08)	1/149	0.180	fracture of joint bar around nonstructural wall concrete crushing of EW60
#3-3	120%	10.03 (11.11)	2154.4(1.17)	1/37	0.274	maximum shear capacity
#3-5	140%	11.70 (12.93)	1626.9(0.89)	1/13	0.409	significant damage of nonstructural and structural walls failure at 1st and 2nd stories
#3-7	140%	11.70 (12.80)	1137.9(0.62)	1/11	0.371	
#3-9	JR 120%	8.76 (9.18)	1362.6(0.74)	1/6	0.297	

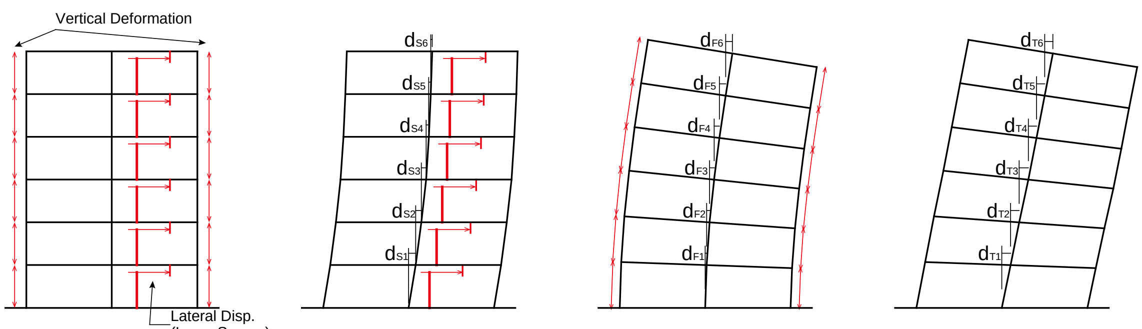
Original wave was JMA Kobe except for #3-9. JR Takatori was used for #3-9. Q_B: Base shear force, C_B: Base shear coeficient(Q_B devided by total weight), $R_{1\mathrm{Max}}$: Maximum interstory drift angle of 1st story, T: Natural period after each shaking test



(a) 3rd Story (b) 5th Story

(c) 1st Story (d) 2nd Story

Fig.8 – Story Shear - Story Drift Relationship of Y Axis



(a) Diagram of measuring system (b) Shear deformation from laser sensor (c) Flexural deformation from column axial deformation (d) Total inter story drift $d_{Ti} := d_{Si} + d_{Fi}$

Fig.9 – Measuring System and Calculation of Interstory Drift

3.2 Process and Detail of the Damage

Fig. 11(a) shows the cracking pattern of X1 frame after the 2nd day test. Joint bars fracture was observed in the 2nd and 3rd floors. Fig. 11(b) shows a captured photo from the recorded video at the 2nd floor during the test case #3-5. Collision of the nonstructural wall with the columns caused shear crack in the wall and crushing of the columns. Fig. 11(c) shows the collision of the cantilever slab and the steel frame.

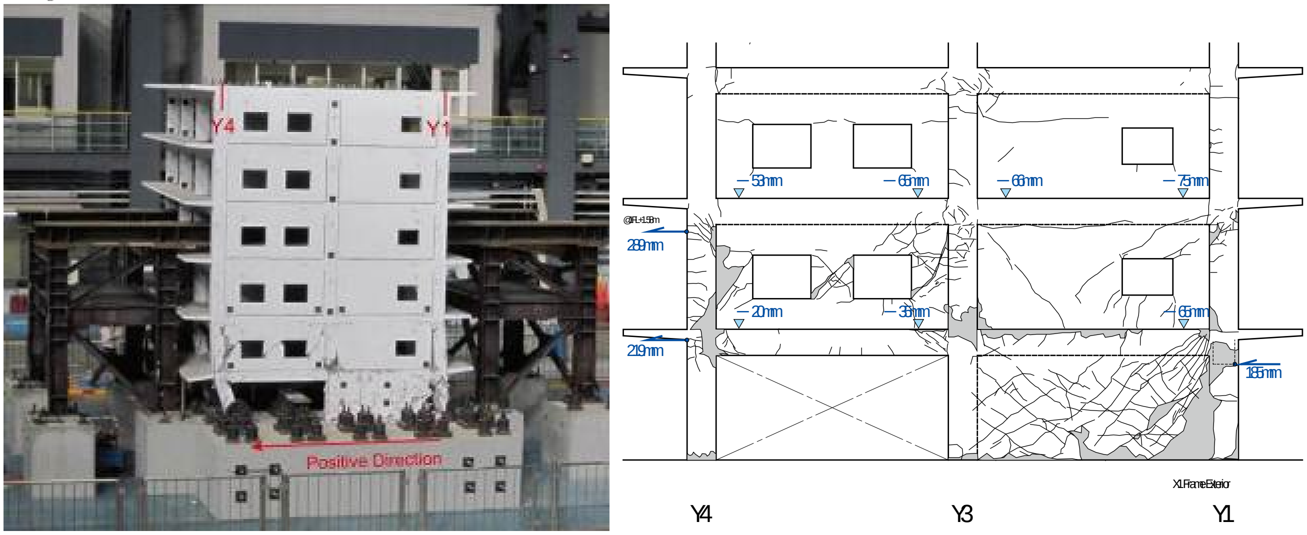
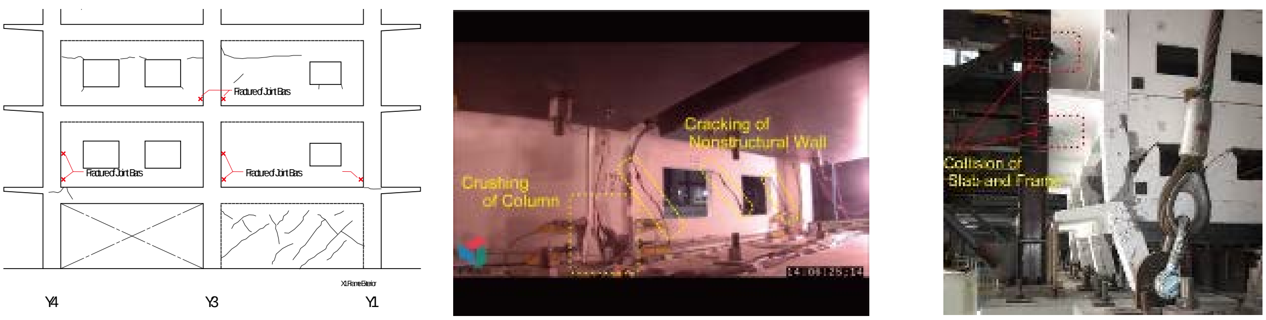


Fig.10 – Damage State of the Specimen, a View from X1 frame



(a) Crack patterns of X1 frame (b) Captured photo in case #3-5 (c) Collision with steel frame

Fig.11 – Damage in Detail

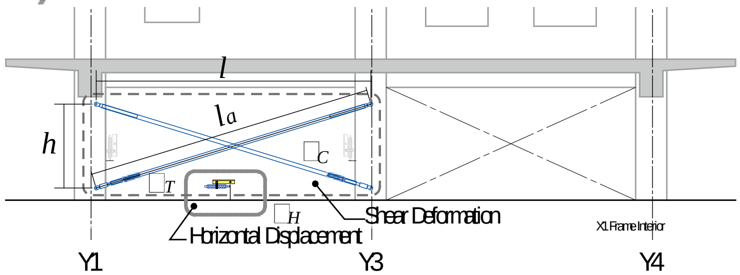
3.3 Behavior of the Walls and Maximum Strength

Diagonal deformation(δ_T and δ_C) and sliding displacement δ_H at the bottom of the wall were recorded. Shear deformation of wall δ_S was calculated from Eq. (2). Fig. 12 shows instrumentation of EW60 in X1 frame.

$$\delta_S = \frac{1}{4l} \cdot (\delta_T - \delta_C) \cdot (2l_\alpha + \delta_T + \delta_C) \tag{2}$$

where, l, l_α: horizontal and diagonal length between the target points, δ_T, δ_C: measured diagonal deformation

Fig. 13 shows the relationship of 1st story shear and lateral displacement of walls. This figure shows only two cases. They are case #3-3 in which maximum response story shear force was observed, and the previous case #2-5. The wall deformation of EW60 in X1 frame was about 2 times larger than that of EW90 or EW60 in X4 frame in both cases. The eccentricity by the difference in stiffness and load capacity between X2 and X3 frames in the 1st story as shown in Fig. 2 was supposed to cause these torsional behavior.



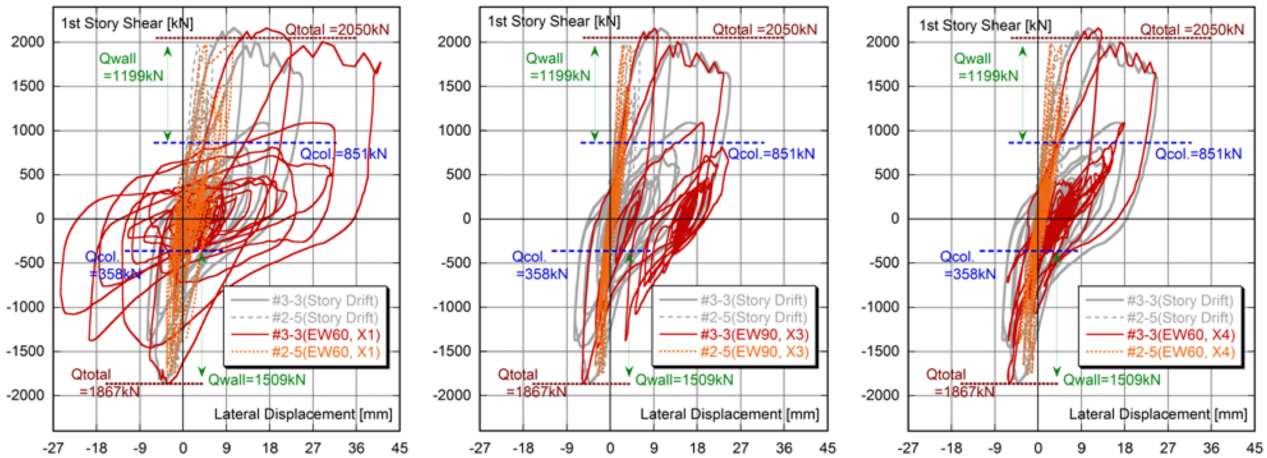
$$\gamma = \frac{1}{4hl} \cdot (\delta_T - \delta_C) \cdot (2l_\alpha + \delta_T + \delta_C)$$

$\delta_S = \gamma \cdot h$: Shear Deformation

δ_H : Horizontal Displacement

$\delta_S + \delta_H$: Lateral Displacement of the wall

Fig.12 – Instrumentation and Lateral Displacement of the Wall



(a) EW60 in X1 frame (b) EW90 in X3 frame (c) EW60 in X4 frame

Qcol.: shear force carried by five columns in the first story

Qwall: shear strength of three walls in the first story, Qtotal = Qcol.+ Qwal

Fig. 13 – Relationship between Story Shear and Lateral Displacement of Wall

(a) Arm length of overturning moment (b) Damaged column (X2Y4) (c) M-N interaction curve(C1A)

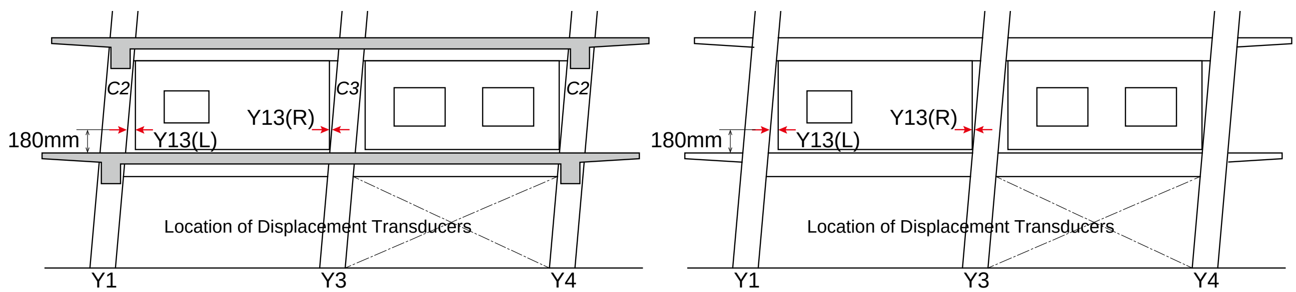
Fig. 14 – Calculation of Column Strength

In Fig. 13, maximum shear force carried by five columns and shear strength of three walls in the 1st story were shown also. The maximum shear force of column was calculated from flexural strength of each column. At this time, axial force applied to each column was estimated from overturning moment at maximum response in both positive and negative loading. The overturning moment was assumed to be distributed equally from X1 to X4 frames. Axial force on each column was calculated as dividing overturning moment by arm length between centers of column and wall as shown in Fig. 14(a). Shear force at flexural strength of the column was calculated as the summation of ultimate flexural moment at the top and bottom sections divided by the column height. As shown in Fig. 14(b), all columns in the 1st floor yielded at both top and bottom section because they had high ductility contributed by large hoop reinforcement ratio. The ultimate flexural moment of column was calculated by section analyses based on the assumption that plane section remained plane after deformation and also by using the estimated axial force. Moment and axial force interaction curve calculated for the column at X2-Y4 was shown in Fig. 14(c). The shear strength of wall was calculated from Eq. (3) based on the standard [3].

$$Q_{wsu} = \left\{ \frac{0.053 p_{te}^{\ 0.23} (F_C + 18)}{M/(QD) + 0.12} + 0.85\sqrt{\sigma_{wh} \cdot p_{wh}} + 0.1\sigma_0 \right\} \cdot t_e \cdot j \tag{3}$$

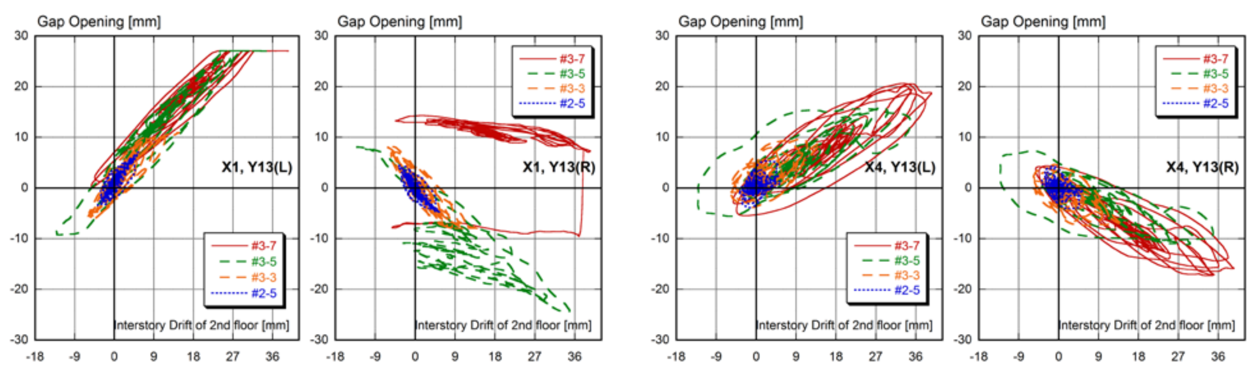
Where, Q_{wsu}: shear strength of wall, t_e: equivalent thickness of wall, j:=7/8d, d:=D-D_c/2, D, D_c: wall length and column depth, respectively, p_{te}: equivalent tensile reinforcement ratio[%], p_{wh}, σ_{wh}: equivalent shear reinforcement ratio and yield strength of shear reinforcement, respectively, F_C: concrete compressive strength, σ_0: average axial stress, $M/(QD)$: shear span to depth ratio (assumed to be 1.0 as a conservative assumption)

The maximum 1st story shear capacity was the total of the shear force at flexural strength of columns and shear strength of walls. The difference of maximum shear between positive and negative loading directions was evaluated by this simple calculation. For negative direction, the capacity of the specimen was not attained because the specimen finally failed in positive direction. Further investigation about the difference of the direction would be required.



X1 Frame X4 Frame

(a) Location of Displacement Transducers



X1 Frame X4 Frame

(b) Interstory Drift – Gap Opening Relations

Fig. 15 – Gap Opening at Slit

3.4 Behavior of the Nonstructural Walls

Fig. 15 shows the relationship of interstory drift and gap opening of nonstructural wall in 2nd story. The displacement transducers were placed at lower part of nonstructural walls as shown in Fig. 15(a). In #2-5, the gap opening was observed more than 5mm. The recorded values more than 5mm were corresponding to the situations that many joint bars were observed to fracture at the end of the 2nd day. In #3-5, the gap opening was observed less than -10mm at Y13(R) in both frames. It means that the gap between wall and column closed, or the nonstructural wall collided to the column, and that corresponded to the failure mode observed at the end of the test. After the collision of wall and column, the columns were heavily damaged as shown in Fig. 10. For Y13(R) in X1 frame of the case #3-7, recorded data were not reliable because of the effect of crushing of the column *C3* in Y3 frame. For all cases, it was observed that displacements of X1 frame were larger than those of X4 frame, which indicated torsional behaviors in the Y direction as mentioned above.

4. Conclusions

A shaking table test was carried out. Specimen was 30% scale 6 story RC structure based on a prototype building designed under the present Building Standard Law of Japan. The findings of this study are as follows:

1. In shaking table test, target responses were achieved in the 1st and 2nd day of the test. Maximum shear capacity of the specimen was observed in the 3rd day. On the final day, 1st and 2nd story walls failed in shear manner. The columns at 1st story had high ductility contributed by large reinforcement ratio of hoop.

2. Story shear calculated by summation of flexural strength of column and shear strength of wall was corresponding to the maximum story shear force of 1st story.

3. Torsional behaviors were observed in wall deformation. Nonstructural wall were damaged and caused column damage after collision.

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References

[1] Masayoshi Nakashima, Keiichiro Suita, Motomi Takahashi, Minehiro Nishiyama, Hideo Katsumata, Koichi Kajiwara and Norihide Koshika (2013): Overview of ‘Quantification of Collapse Margin of High-Rise Buildings’ Quantification of Collapse Margin of Steel High-Rise Buildings (Part 1). *Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan*, pp.967-968, 2013/08 (in Japanese)

[2] Hideo Katsumata, Yasuhiko Masuda, Kuniyoshi Sugimoto, Kenji Yonezawa, Katsuhisa Nishimura, Kota Miura, Minehiro Nishiyama, Yasushi Sanada, Taizo Matsumori(2015): Shaking Table Test of 6 Story Wall Frame Building; Quantification of Collapse Margin of RC Buildings Part 8, 9, 10 and 11. *Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan*, pp.339-346, 2015/09 (in Japanese)

[3] Ministry of Land, Infrastructure, Transport and Tourism(2015): Technological Standard Related to Structures of Buildings, Building Center of Japan (in Japanese)