SHAKE-TABLE TEST OF A 4-STORY FRAME-WALL RC STRUCTURE TO INVESTIGATE THE COLLAPSE PROCESS AND SAFETY LIMIT STATE

K. Miura[(1)], K. Yonezawa[(2)], Y. Tabata[(3)], K. Fujita[(4)], A. Shegay[(5)], H. Alwashali[(6)] and M. Maeda[(7)]

[(1)] *Research Engineer, Technical Research Institute, Obayashi Corporation, miura.kota@obayashi.co.jp*
[(2)] *Senior Research Engineer, Technical Research Institute, Obayashi Corporation, yonezawa.kenji@obayashi.co.jp*
[(3)] *Graduate Student, Graduate School of Engineering, Tohoku University, tabata@rcl.archi.tohoku.ac.jp*
[(4)] *Graduate Student, Graduate School of Engineering, Tohoku University, fujita@rcl.archi.tohoku.ac.jp*
[(5)] *Researcher, Graduate School of Engineering, Tohoku University, ashegay@rcl.archi.tohoku.ac.jp*
[(6)] *Assistant Professor, Graduate School of Engineering, Tohoku University, hamood@rcl.archi.tohoku.ac.jp*
[(7)] *Professor, Graduate School of Engineering, Tohoku University, maeda@archi.tohoku.ac.jp*

Abstract

After an earthquake, it is important to judge the safety of buildings and make an efficient recovery plan. For that, it is necessary to know quantitatively the damage level and safety limit of buildings. An evaluation method of residual seismic capacity is described in the Japanese Standard for Post-earthquake Damage Level Classification of Buildings; however, the method does not consider the difference of deformation capacity of members such as shear walls and frames (columns and beams). Even though evaluation methods were proposed in previous research for the damage level and safety limit of reinforced concrete (RC) buildings, the focus of the method has been mainly for moment-resting frames. Moreover, not enough experimental investigation has been done to verify the application of these methods. In this research, a new evaluation method for the damage process to collapse (collapse mechanism) and safety limit state of dual structures, which have members with different deformation capacity, was proposed. A shake-table test has been carried out to investigate the applicability of the proposed method to RC buildings with dual structural systems consisting of moment resisting frames and shear walls.

The test specimen was a 1/4 scale model of a 4-story RC building with multi-story shear walls in both longitudinal and transverse directions (X- and Y-directions). The structure was designed to exhibit a total collapse mechanism (frame-sway mechanism) and so, plastic hinges were designed at the bottom of columns and walls in the first story and beam ends of each story. To investigate the difference of collapse mechanism (process) in the X- and Y-directions, contribution ratio of shear wall strength to the whole seismic capacity of the frame was varied in each direction. In the X-direction, two shear walls were placed with the intention of making the failure of shear walls dominate the collapse mechanism of the whole structure (i.e., the failure point of walls corresponded to the global structural safety limit state). In the Y-direction, only one shear wall was used such that failure of columns and beams would dominate the global collapse mechanism, which meant that failure of the wall should not induce collapse of the structure. The design concept was quantitatively confirmed based on seismic capacity indices using results of nonlinear pushover analyses of the specimen in structural modeling software.

In the shake-table test, scaled artificial ground motions compatible with the Japanese standard design demand spectrum were used as input. The damage of walls preceded in both directions and at the end of the test, the walls were severely damaged and the whole structure was close to collapse. The strength and the deformation capacity of the structure were higher than predicted by the analysis. Finally, the collapse mechanism and the safety limit state of the specimen was evaluated. As a result, the collapse mechanism of the X-direction (longitudinal direction) frame was wall-dominant and the wall failure point corresponded to the safety limit state similar to the result based on analyses before the test. However, the test results were not so clear compared to the analytical results because of the high residual strength of walls after failure, which is ignored in the analytical evaluation. In the Y-direction (transverse direction), the collapse mechanism was also wall-dominant, even though a frame-dominant response was anticipated. Therefore, in both directions the wall failure point corresponded to the safety limit state. It was concluded that the accumulated damage of columns and beams by former shakings degraded their seismic capacity after the wall collapse, which is not was not considered in the analytical evaluation.

Keywords: Shake-table Test; RC Building; Safety Limit State; Collapse Process; Multi-story Shear Wall;

1. Introduction

After an earthquake, it is important to judge the safety of buildings and make an efficient recovery plan. For that, it is necessary to know quantitatively the damage level and safety limit of buildings. In the Japanese Standard for Post-earthquake Damage Level Classification of Buildings (hereafter referred to as the JBDPA Guidelines [1]), an evaluation method of residual seismic capacity of damaged buildings is described. A key limitation of this method is that it implicitly assumes that all structural members (shear walls and frames (column and beams)) have an identical deformation capacity. Although previous research [2-3] has discussed evaluation methods for the damage level and safety limit of reinforced concrete (RC) buildings, the focus has been predominantly on moment-resisting frames. Moreover, limited system-level experimental investigation has been done to demonstrate and verify the implementation of these previous methods.

In this research, a new method is proposed to evaluate the damage process to collapse (collapse mechanism) and the safety limit state of dual-system structures, which have members with different deformation capacities. The method is essential for the evaluation of the residual seismic capacity of structures. A 1/4-scale shake-table test of a 4-story RC building with multi-story shear walls and frames with different deformation capacity has been carried out. The main objective is to investigate the collapse mechanisms and safety limit states of dual structures through application of the proposed evaluation method.

2. Evaluation Method of Collapse Mechanism and Safety Limit

The seismic capacity index (SCI), α, is defined in the AIJ Guidelines for Performance Evaluation of Earthquake Resistant Reinforced Concrete Buildings [4] as a ratio of intensity of a 5% damped response acceleration spectrum passing through a certain point on the capacity spectrum curve to the 5% design demand spectrum (provided in the Japanese Notification No.1457 [5] and generally used in structural design of buildings). The capacity spectrum is obtained by reducing the structure's story shear-inter-story drift relationships to an equivalent base shear-deformation response of a single-degree-of-freedom system.

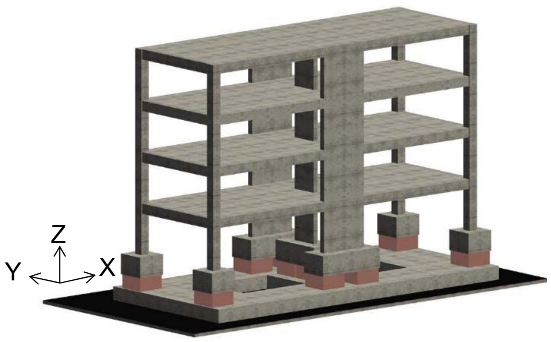
Matsukawa et al. [3] has previously defined the safety limit of moment frames as the point of the maximum SCI, α, required to reach the structure's deformation capacity. While this method was shown to correlate well to other collapse evaluation procedures, such as incremental dynamic analysis [3], it is limited to application to structures with a single performance limit state. Structures containing multiple limit states, such as dual systems (for example, a frame-wall dual system), a single SCI value is not sufficient to understand the expected dominant response of the structure. The proposed procedure outlined next determines the SCI for each major limit state in the structure and compares them to determine the dominant collapse mechanism.

For a structure containing both walls and a moment-frame, two SCIs are determined. The wall SCI, α_W, is defined as the design demand spectrum magnification resulting in the wall failure point while the frame SCI, α_F, is defined as the design demand spectrum magnification resulting in failure of the frame elements following the wall failure. This interpretation is used in the later discussion of the test results. Next, the two SCIs, α_W and α_F, are compared. When α_W is larger than α_F, the wall failure point is taken as the safety limit of the entire structure and the collapse of the structure is considered to be dominated by walls. When α_F is larger than α_W, the column/beam failure point is taken as the safety limit of the structure and the collapse of the structure is considered to be dominated by columns and beams. To demonstrate and evaluate the application of this method, an RC structure with different collapse mechanisms was designed and dynamically tested.

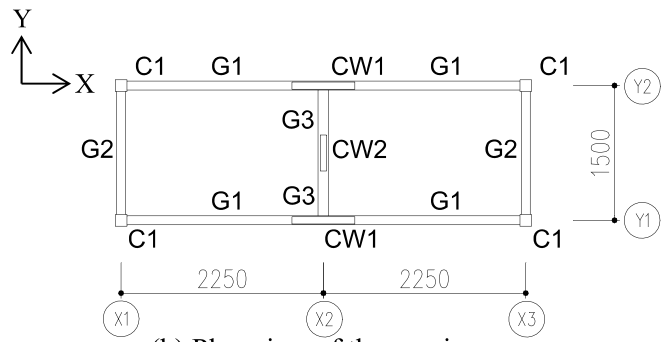
3. Shake-Table Experiment Design

3.1 Outline of Specimen

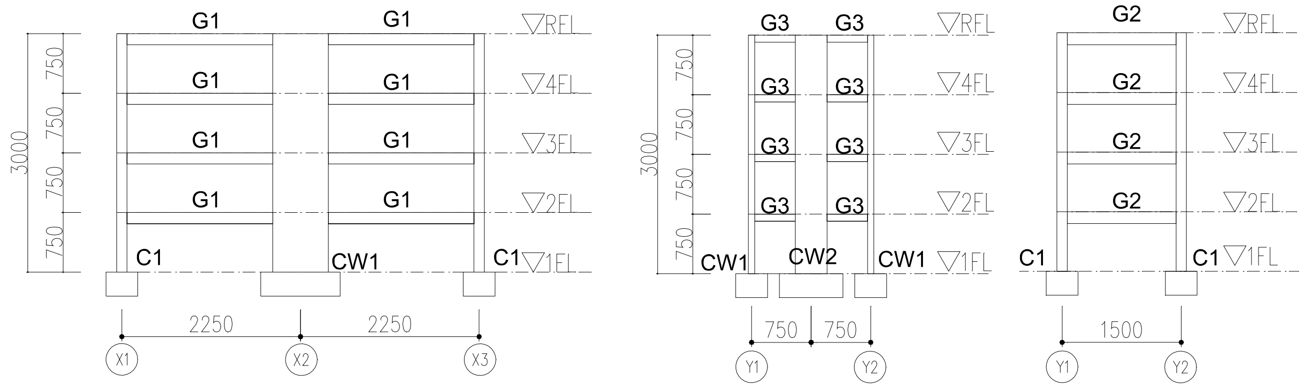
The general plan and elevation views of the test structure are shown in Fig. 1. Member section drawings and section reinforcement details are listed in Table 1. The test specimen is a 1/4 scale model of a 4-story RC building with multi-story shear walls in both longitudinal (X-direction) and transverse directions (Y-direction) of the structure.



(a) Appearance of the specimen



(b) Plan view of the specimen

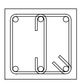
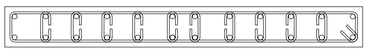
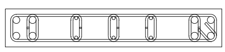
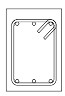
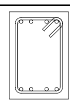
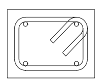


Y1 and Y2 frame X2 frame X1 and X3 frame

(c) Elevations of the specimen

Fig. 1– (a) General view; (b) plan view; (c) X- and Y-direction elevations of the test structure.

Table 1– List of members.

Member	Column	Wall		Beam			Slab
	C1	CW1	CW2	G1	G2	G3	
							
Size (mm)	130×130	80×700	70×400	100×140	100×150	120×90	70
Main bar	6-D10	24-D10	8-D13 + 6-D6	3/3-D6	4/4-D6	2/2-D6	D4@80 (X-dir.)
Hoop / Stirrup	D4@60	D4@60	D4@100 (cross-ties D4@50)	D4@60	D4@60	D6@30	D4@60 (Y-dir.)

3.2 Design Concept and Details of Specimen

The design concept of the specimen is shown in Fig. 2. Material properties are shown in Table 2 and Table 3 and flexural strength and ultimate deformation of members as calculated by the AIJ Guideline [6] are shown in Table 4. The structure was designed to exhibit a total collapse mechanism (frame-sway mechanism) and so, plastic hinges were designed to form at the bottom of columns and walls in the first story and beam ends of each story. To investigate the difference of collapse mechanism in the X- and Y-directions of the structure, contribution ratio of shear wall strength to the whole seismic capacity (base shear) of the structure was varied in each direction. In the X-direction, two shear walls were placed with the intention of making failure of shear walls dominate the collapse mechanism of the whole structure, i.e., the failure point of the wall corresponded to the global structural safety limit state. In the Y-direction, one shear wall was used such that failure of columns and beams would dominate the global collapse mechanism, which meant that failure of walls did not induce collapse of the structure. In the calculation of flexural strength of beams, effective width of slabs was calculated as half of the total floor span in the direction of the beam, and varying axial force was considered in the calculation of columns and walls. Contribution ratio of shear wall strength to the whole seismic capacity of the structure was estimated to be 55% in X-direction and 20% in Y-direction.

Steel masses were fixed on the slabs to make axial stress of columns and walls by dead and live loads equal to those of the original scale building. Total weight of the specimen and the masses is approximately 75 kN for each floor (including the roof). Several accelerometers were placed at the corners of each floor slab and load-cells were installed at the base of all columns and walls. Inter-story drifts were recorded by laser displacement sensors. Each floor of the structure was instrumented with reinforcement strain gauges and displacement transducers in the member plastic hinge regions.

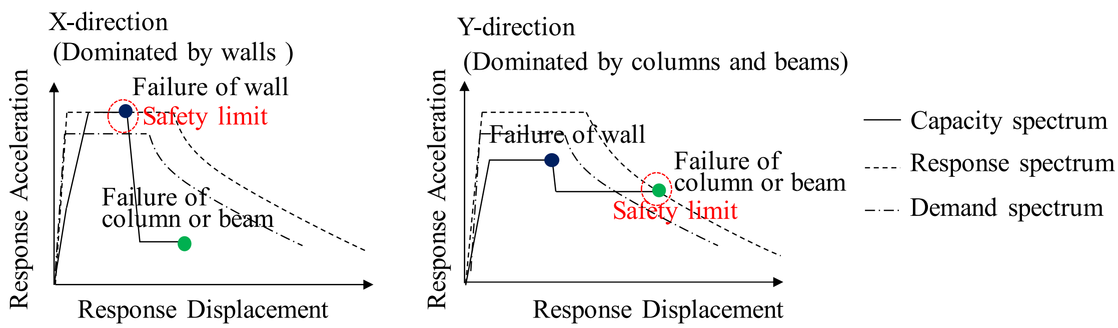


Fig. 2 – Design concept (target collapse mechanism) of specimen.

Table 2 – Tested material properties of concrete (average of 1st-4th stories).

Grade	Tested day	Compressive strength(N/mm^2)
Fc30 (early strength)	7 days after casting	41.2

Table 3 – Tested material properties of reinforcement.

Diameter (mm)	Grade	Yield strength (N/mm^2)	Maximum strength (N/mm^2)	Young's modulus (N/mm^2)
4	SD295A	402	533	1.90×10^5
6	SD345	419	613	1.97×10^5
10	SD345	339	562	1.93×10^5
13	SD390	407	602	1.95×10^5

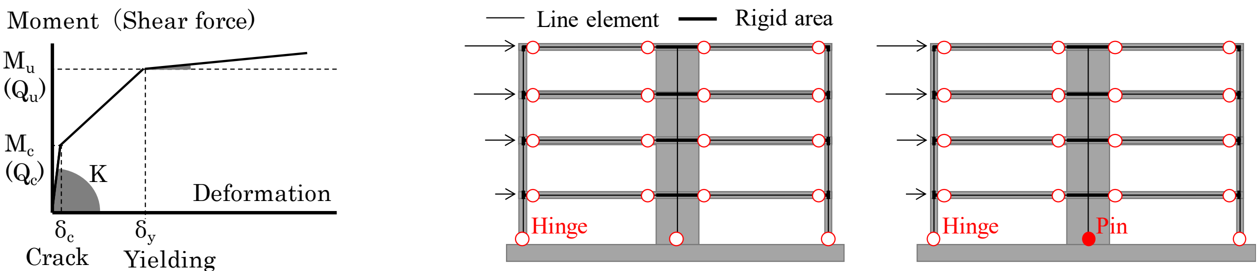
Table 4 – Calculated flexural strength and ultimate deformation capacity of all members.

Member	Column		Wall				Beam		
	C1		CW1		CW2		G1	G2	G3
	X-dir.	Y-dir.	X-dir.	Y-dir.	X-dir.	Y-dir.			
Flexural strength (kNm)	7.1	9.5	179.0	20.8	12.8	78.0	7.2	10.4	3.9
Ultimate deformation (rad.)	0.043	0.042	0.022	0.044	0.044	0.023	0.047	0.042	0.060

4. Pre-Experiment Analysis

4.1 Analysis Model

A 3-D frame model of the test structure was created where columns, walls and beams were modelled as line elements with nonlinear rotational springs at the ends of the elements and axial springs and shear springs at the center. Rotational springs and shear springs of walls were given tri-linear backbone curve characteristics determined by the AIJ Guideline [6], as shown in Fig. 3(a), while all other springs were elastic. Dead and live loads were considered as concentrated loads to nodes and distributed loads on beams. Pushover analyses were performed for two kinds of models shown in Fig. 3(b)(c), where horizontal external force distribution was applied based on the 'Ai' distribution (as prescribed in the Japanese Notification [7] as a load distribution that approximately represents the demands from dynamic analysis). The first model is a pre-wall failure model (normal frame model). The second is a post-wall failure model, in which the bottoms of the walls are pinned to represent the condition of a structure where the wall experiences failure and no longer contributes to the overall structure's stiffness and deformation capacity. In the analyses, the material test results shown in Table 2 and Table 3 were used and Young's modulus of concrete was estimated by its compressive strength based on the AIJ Standard [8].



(a) Tri-linear backbone curve of flexural (and shear) springs.

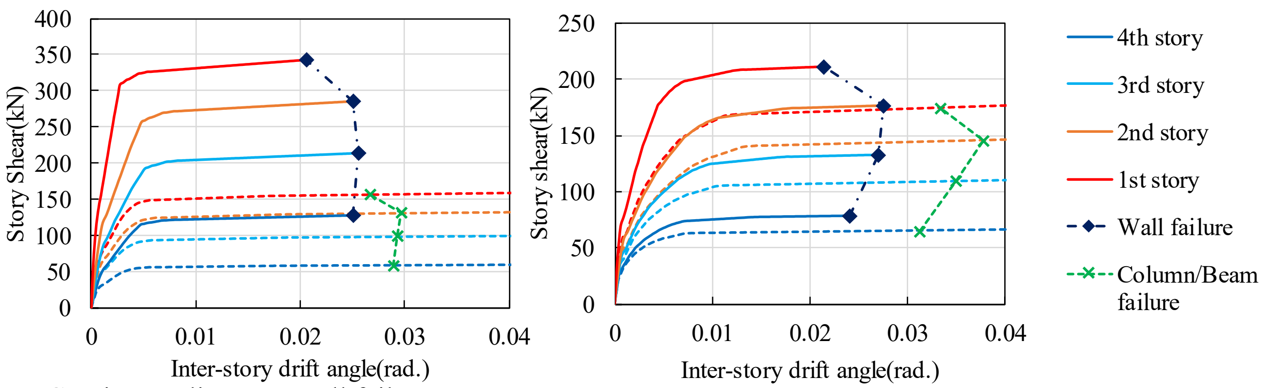
(b) Pre-wall failure model (Y1 and Y2 frame)

(c) Post-wall failure model (Y1 and Y2 frame)

Fig. 3 – Models for pushover analysis.

4.2 Results of Analyses

Story shear-inter-story drift angle relationships by pushover analyses are shown in Fig. 4. The failure point of the wall or column/beam shown in the figure was defined as the point when the drift angle of any of the walls or columns/beams reached the ultimate deformation capacity shown in Table 4. In the X-direction, frame failure was estimated to occur soon after wall failure, whereas in the Y-direction the structure has additional deformation capacity before failure of the frame. Additionally, in the X-direction, the deterioration of structural lateral capacity following wall failure is significant (50%) compared to that of the Y-direction.



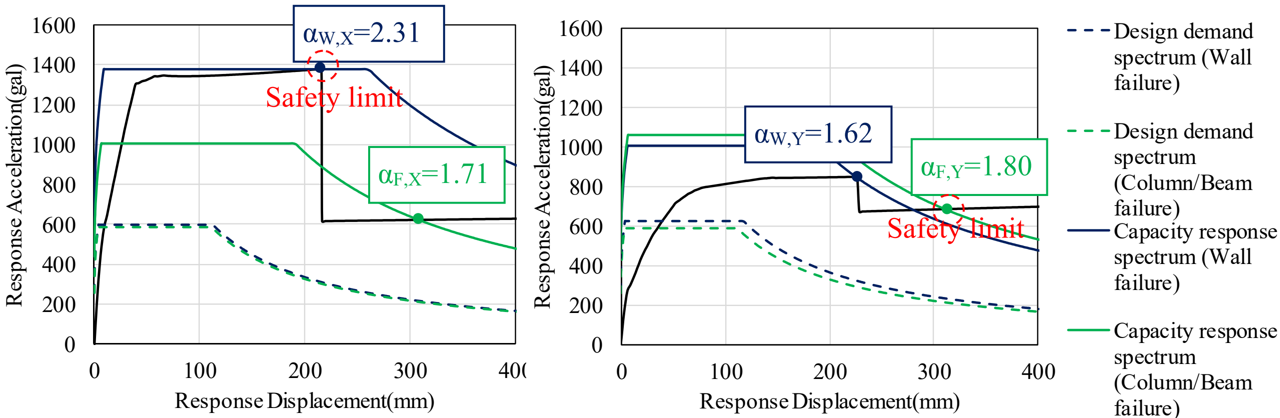
Continuous line: Pre-wall failure model Broken line: Post-wall failure model

(a) X-direction (b) Y-direction

Fig. 4 – Story shear- inter-story drift angle relationships by pushover analyses.

5. Evaluation of Collapse Mechanism and Safety Limit Based on Analytical Results

According to the proposed evaluation method described in section 2, the collapse mechanism and the safety limit point of the specimen was evaluated based on the pre-analysis results. Calculated capacity spectra and seismic capacity indices of the experimental structure are shown in Fig. 5. In this figure, displacement was quadrupled to match real scale because the design demand spectrum used in the calculation is designed for real scale buildings. In calculation of the capacity spectrum, the pre-wall failure model shown in Fig. 3(b) was used prior to the wall failure point; the post-wall failure model shown in Fig. 3(c) was used for displacements after the wall failure point. It assumed that the lateral strength capacity of the walls decreased rapidly to zero after the failure point. As shown in Fig. 5, $\alpha_{W,X}$ was determined to be larger than $\alpha_{F,X}$ in X-direction and $\alpha_{F,Y}$ was larger than $\alpha_{W,Y}$ in Y-direction; thus, the X-direction was evaluated to be dominated by the wall collapse mechanism (i.e., the wall failure point is the safety limit of the entire structure) and the Y-direction dominated by the frame collapse mechanism (i.e., the frame failure point is the safety limit).

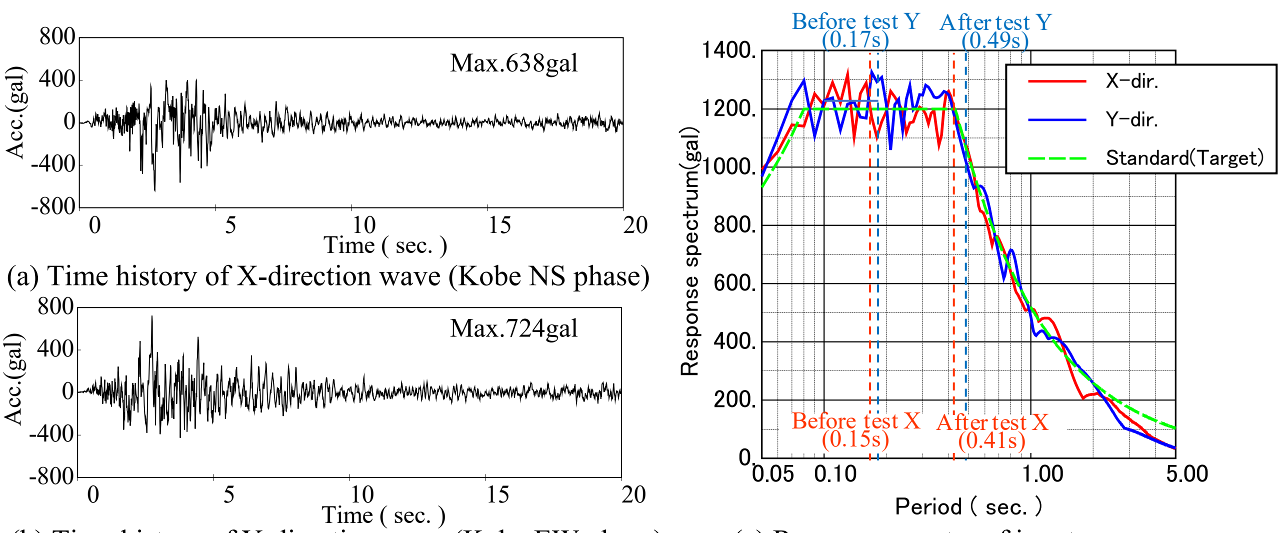


(a) X-direction (b) Y-direction

Fig. 5 – Capacity spectra of the structure and the corresponding seismic capacity indices.

6. Input Wave

Artificial earthquake waves compatible with the Japanese standard design demand spectrum (shown as the 'standard' spectrum in Fig. 6(c) were used as input for the shake-table test. The phases of the waves were



(a) Time history of X-direction wave (Kobe NS phase)

(b) Time history of Y-direction wave (Kobe EW phase) (c) Response spectra of input waves

Fig. 6 – Time histories and response spectra of input waves.

decided based on JMA Kobe record observed in the 1995 Hyogo-ken Nanbu Earthquake. Time histories and response spectra of the waves are shown in Fig. 6, respectively. In Fig. 6(c), the 1st natural periods of the specimen before and after the test are shown. In the waves, time scale is reduced to 1/2 of the original waves based on the law of similitude. The acceleration response spectra of the records correspond to the design spectra used in the evaluation of seismic capacity indices and safety limit points of the specimen in Fig. 5. The records' acceleration response spectra were gradually scaled in the X- and Y-directions using the nine sets of amplification factors shown in Table 5.

7. Shake-Table Test Results

Member structural damage states were observed and damage level determined after each shaking (except Run 7-8). The damage level was evaluated from level '0' to 'V' according to JBDPA Guidelines [1]. In this standard, the expected damage for a damage level 'V' member includes buckling of longitudinal reinforcement, crushing of concrete, and residual vertical or lateral deformation of members as determined by visual observation.

Fig. 7 shows the damage state of the test structure after all shakings. The damage progression of the structure is as follows. In Run 1, flexural cracks were observed in the plastic hinge zones of the walls and some frame members. In Run 3, the longitudinal reinforcement of the first-floor columns and X-direction walls experienced yielding. Following Run 4, longitudinal reinforcement of the columns at the fourth floor and the beams at each floor yielded indicating that the frame-sway structural response mechanism had initiated. The longitudinal and transverse reinforcement of the Y-direction wall at the first floor yielded almost at the same time. After Run 5, cover concrete spalling at the base of the first-floor X-direction walls and first-floor columns occurred. In the Y-direction wall, significant shear cracking and spalling of concrete were observed in the first floor. The specimen almost reached its maximum strength in both X-direction and Y-direction. In Run 7, the first floor wall in the Y-direction experienced shear failure (damage level V) as shown in Fig. 7(d), even though a flexural failure was anticipated in the original design (as shown in section 3.2). The reason for this is thought to be that G3 beams (shown in Fig. 1) framing into the wall had a higher than anticipated flexural strength, due to the effect of the slabs. Finally, following Run 9, severe concrete crushing and buckling of longitudinal reinforcement was observed at the lower corners of the first-floor X-direction walls as shown in Fig. 7(b), suggesting that the wall had undergone a flexural failure.

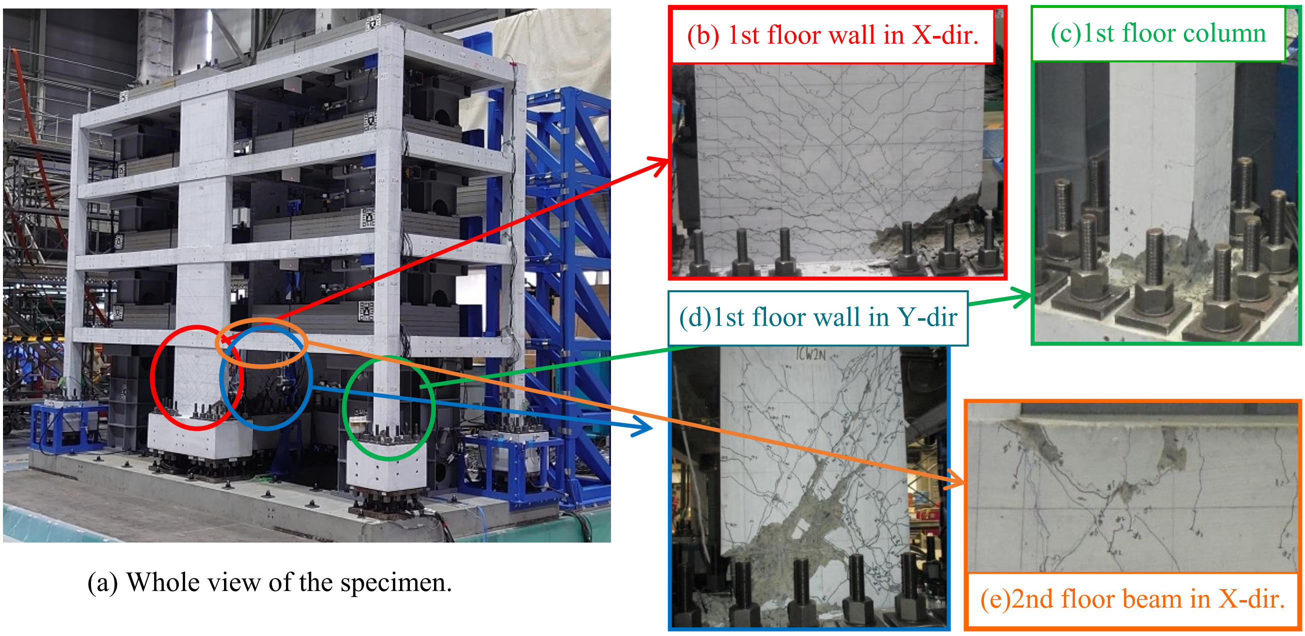


Fig. 7 – Damage state of structural members after the final excitation (Run 9).

Key test results are summarized in Table 5, and X- and Y-direction story shear-inter-story drift angle relationships are shown in Fig. 8, respectively. The story shear-inter-story drift angle backbones from results of the pre-analyses are also shown in Fig. 8. In the analysis, the assumption that lateral load capacity of the walls decreased rapidly to zero after the failure point was applied. In the test, the maximum strength of the structure was higher than that of the analysis result by 30% in the X-direction and 40% in the Y-direction. This strength difference is also attributed to a higher than anticipated slab contribution to the strength of the beams. Moreover, the deformation capacity was higher in the test than the pre-analysis in both directions. Particularly in X-direction, strength deterioration was not observed in the final excitation (Run 9) despite the maximum inter-story drift angle reaching 1/18 rad. Therefore, as expected, the ultimate member deformation capacity values provided in the AIJ Guideline [6] (shown in Table 4) are conservative compared to the real deformation capacity. In addition, the assumption of rapid lateral strength loss of walls after the flexural failure (used in pre-analysis) may be too conservative to evaluate the performance of the structure.

Table 5 – Outline of test results.

Run	X-direction			Y-direction			Event
	Magnification	Q_{1max} (kN)*	R_{max} (rad.) [story]**	Magnification	Q_{1max} (kN)*	R_{max} (rad.) [story]**	
1	20%	51.2	1/1384[2nd]	20%	48.6	1/1475[2nd]	Member cracking
2	80%	167.0	1/320[2nd]	60%	128.0	1/447[1st]	
3	160%	292.4	1/154[2nd]	100%	195.1	1/210[2nd]	1st floor column and wall yielding
4	240%	402.9	1/73[3rd]	150%	290.2	1/77[2nd]	Frame-sway mechanism reached
5	260%	437.8	1/34[2nd]	170%	292.2	1/32[2nd]	Cover concrete spalling of 1st story column and wall (X-dir.) and significant shear cracking and cover concrete spalling of 1st floor wall (Y-dir.)
6	130%	290.1	1/45[3rd]	100%	248.6	1/33[1st]	Checked residual seismic capacity by small input magnification
7	220%	406.7	1/29[2nd]	120%	243.1	1/25[1st]	Shear failure of 1st story wall in Y-dir.
8	220%	427.8	1/25[2nd]	-	-	-	
9	260%	444.2	1/18[2nd]	-	-	-	Flexural failure of 1st story wall in X-dir.

*Q_{1max}: Maximum base shear force, **R_{max}: Maximum inter-story drift angle [story where inter-story drift angle is the largest].

8. Evaluation of Collapse Mechanism and Safety Limit

8.1 Evaluation Method of Collapse Mechanism and Safety Limit Based on the Test Results

As only a single structure was tested with nine input excitations, the seismic capacity index corresponding precisely to wall and frame failure cannot be determined and must therefore be estimated from available data. An evaluation method of the experimental SCI, ${}_e\alpha$, of the test structure derived from the test results (where the subscript 'e' represents the experimental value) is shown in Fig. 9 and is described below:

(1) Check of Failure Point of Member

The shaking when wall failure was observed for the first time (i.e., the wall is evaluated as damage level V by observation) is named as Run W. Following Run W, the maximum response of the wall has already surpassed its ultimate deformation; thus, the actual failure point of the wall is in the response between that of the previous excitation (i.e., Run W-1) and Run W.

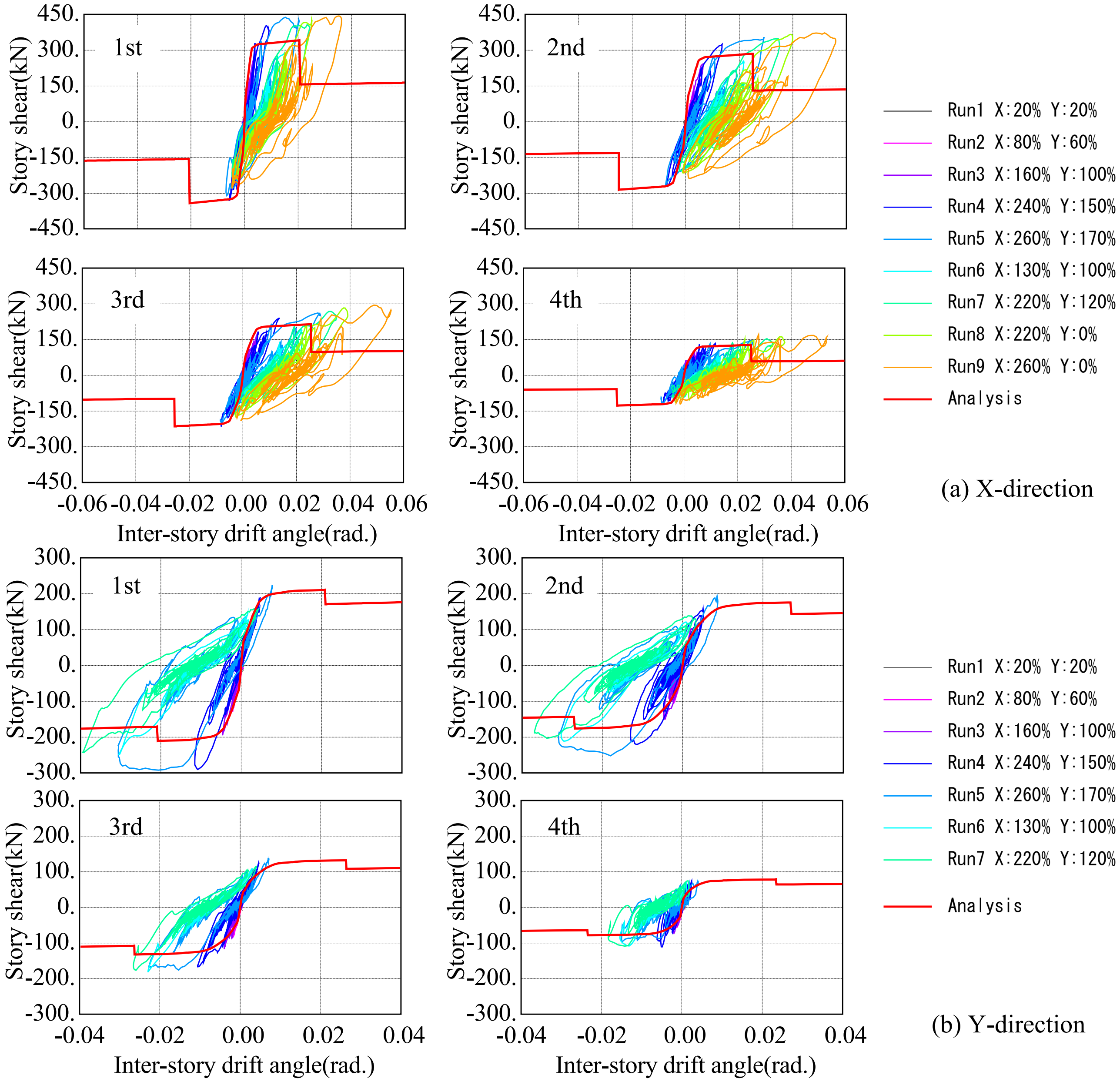


Fig. 8 – Story shear – inter-story drift angle relationships results from the shake-table test.

(2) Definition of the Seismic Capacity Index

According to the definition explained in section 2, SCI based on wall failure, $_e\alpha_W$, is defined as the maximum input magnification in shakings between Run 1 to Run W-1, prior to the wall reaching its ultimate deformation capacity. In the same way, the excitation after which column/beam failure occurred is named as Run F and the SCI based on column/beam failure, $_e\alpha_F$, is defined as the maximum input magnification between Run W to Run F-1 i.e., after the wall failure and before the column/beam failure.

(3) Evaluation of Safety Limit Point and Collapse Mechanism of Structure

Next, SCIs $_e\alpha_W$ and $_e\alpha_F$ are compared. If $_e\alpha_W > {}_e\alpha_F$, the structure's collapse mechanism is considered to be dominated by the walls, and the maximum response in the Run with the maximum input magnification between Run 1 to Run W-1 is the safety limit point of the structure. If $_e\alpha_F > {}_e\alpha_W$, the collapse of the structure is considered to be dominated by the frame and the maximum response in the Run with the maximum input magnification between Run W and Run F-1 is the safety limit point of the structure. In the case when $_e\alpha_W =$

${}_e\alpha_F$, to be conservative the safety limit point of the structure is defined as the response which produces the lowest deformation demand, generally, the response for ${}_e\alpha_W$.

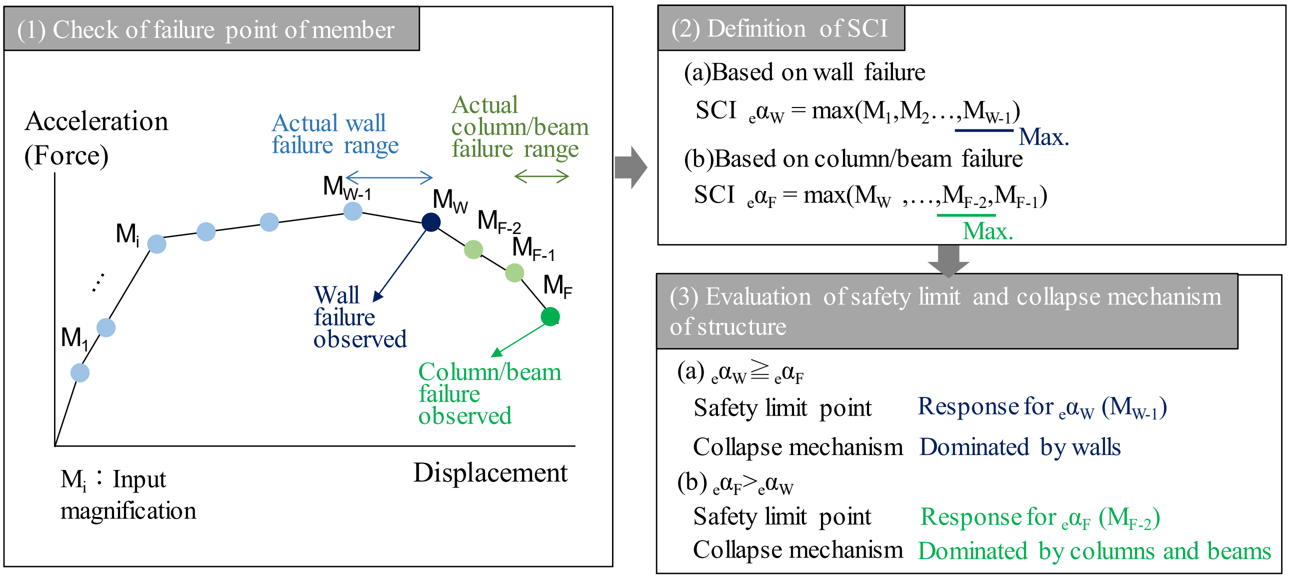


Fig. 9– Evaluation method of safety limit point and collapse mechanism based on the test results.

8.2 Results of Evaluation

Fig. 10 shows the results of the evaluation based on the method explained in Section 8.1. In this figure, the displacement-acceleration envelope of points corresponding to the maximum base shear of each excitation is plotted as the response curve. The predicted seismic capacity evaluation results before the test (from Fig. 5) are also shown in Fig. 10. In Fig. 10(b), results of Run 8 and 9 are omitted because there was no excitation input in the Y-direction for these runs. In the test, wall failure was observed at Run 9 in the X-direction and at Run 7 in the Y-direction; these runs correspond to Run W in Fig. 9. On the other hand, column or beam failure was not observed in the experiment in either direction; thus, the final excitation in each direction (i.e., Run7 for the Y-direction and Run 9 for the X-direction) was regarded as the case just before frame failure was observed (i.e., Run F-1 in Fig. 9). It is acknowledged that this assumption may give conservative ${}_e\alpha_F$ values because it is possible that larger input magnifications were required to induce frame failure.

Based on the above described procedure, SCI in the X-direction, ${}_e\alpha_{W,X}$, was the Run 5 input magnification of 260% (the maximum input magnification between Run 1-8), which is larger than the analytical value $\alpha_{W,X}$ by about 10%. This difference is expected because the strength of the structure in the test was higher than the analytical result. In the Y-direction, ${}_e\alpha_{W,Y}$ was also decided as the Run 5 magnification factor (170%) and ${}_e\alpha_{W,Y}$ was almost equal to $\alpha_{W,Y}$, despite the strength of the structure being higher in the test than predicted in analysis. It is likely that ${}_e\alpha_{W,Y}$ is smaller than if the original structure was tested directly due to accumulated damage from excitations prior to Run 5. Typical damage levels of members (classified using the JBDPA Guidelines [1]) in each frame direction observed after Run 4 are shown in Fig. 11(a)(b). It can be recognized that the damage levels were not insignificant in the Y-direction. On the other hand, in the X-direction, as the damage levels were generally low prior to Run 5, the increase of ${}_e\alpha_{W,X}$ compared to the initial prediction is dominated by the difference between the predicted and experimental strength of the structure rather than the effect of prior damage.

SCI based on column/beam failure, ${}_e\alpha_{F,X}$, was much larger than $\alpha_{F,X}$ in the X-direction. This is likely due to a higher than predicted deformation capacity of walls, which allowed the strength of the structure to be maintained even if visual damage was severe. On the other hand, in the Y-direction, the wall failed in shear and story shear capacity degraded, resulting in a ${}_e\alpha_{F,Y}$ smaller than $\alpha_{F,Y}$. In the calculation of $\alpha_{F,Y}$, the analysis model considered only prior damage for walls (by introducing a base pin) but ignored the effect of prior damage of columns and beams. In reality, prior damage levels of columns and beams (prior to Run 6) were not insignificant as shown in Fig. 11(c). This is one potential reason for why the experimental value ${}_e\alpha_{F,Y}$ was smaller than anticipated. In addition, high residual deformation (about 1/100 rad. after Run 6) could degrade the seismic performance further and was not captured by the analytical result. The result means that the seismic performance deterioration after the wall collapse was larger than expected.

As a result, in both the X- and Y-directions, SCI ${}_e\alpha_W$ was equal to or larger than ${}_e\alpha_F$ and thus collapse mechanism was evaluated as 'wall-dominated'; however, it is acknowledged that this conclusion may be affected by the input order of excitations. The safety limit points for both directions were regarded as the maximum response of the structure to Run 5. In the Y-direction, the experimental result was different from the analytical prediction while in the X-direction, ${}_e\alpha_{W,X}$ is not larger than ${}_e\alpha_{F,X}$ meaning that the dominant collapse mechanism is not as obvious as the analytical results suggested. Therefore, in order to evaluate the dominant collapse mechanism and a safety limit of a structure appropriately, it is important to evaluate deformation capacity and residual strength of walls with flexural failure mode more accurately. This demonstrates that the equal deformation capacity assumption used in the JBDPA Guidelines may not be appropriate for dual structural systems. Additionally, seismic performance deterioration such as stiffness and damping by damage accumulations from prior earthquake needs to be taken into account.

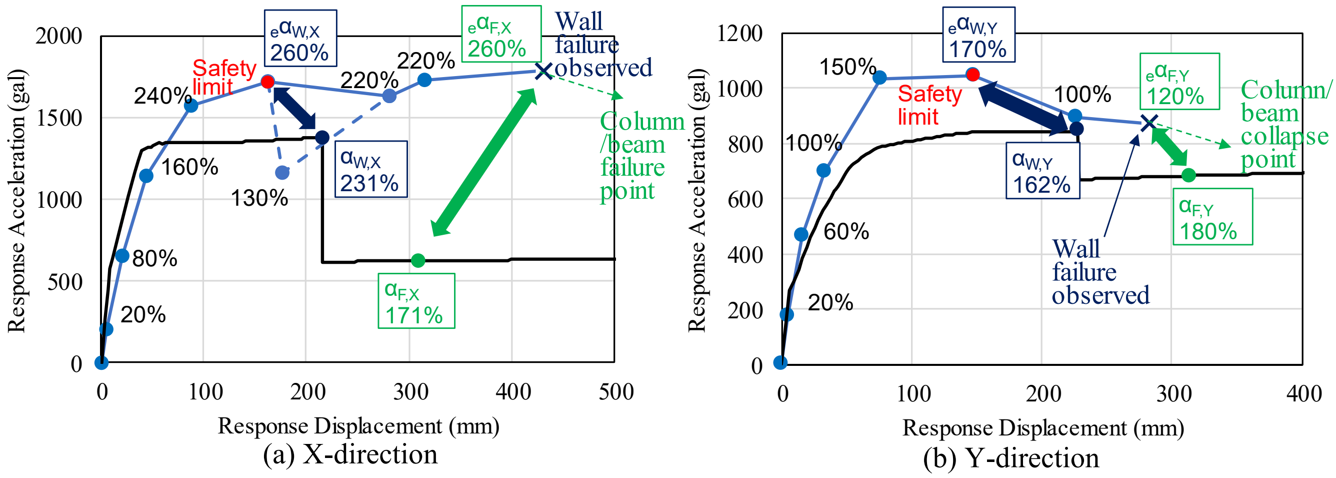


Fig. 10 – Results of evaluation of structure collapse mechanism and safety limit point based on the test.

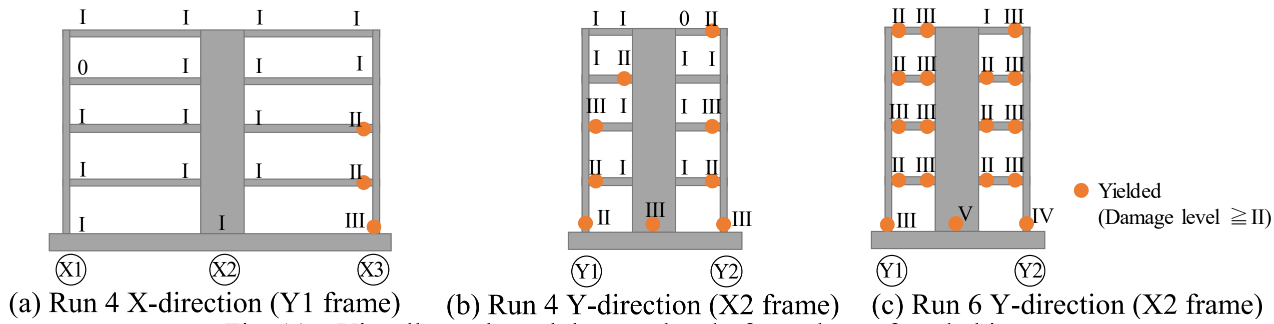


Fig. 11 – Visually evaluated damage level of members after shaking.

9. Conclusions

To investigate collapse mechanisms and an evaluation method of the safety limit of RC buildings, a shake-table test of a 1/4 scale model of a 4-story RC structure with shear walls was carried out. The findings of this study are as follows:

1. The specimen was designed to exhibit a frame-sway mechanism, and so, plastic hinges were designed at the bottom of columns and walls in the first story and beam ends of each story. In the X-direction (longitudinal direction), the specimen was intended to show a wall-dominant collapse mechanism in which the failure point of walls was the safety limit of the whole structure. In the Y-direction (transverse direction), the structure was designed to show a frame-dominant collapse mechanism in which the failure point of columns or beams was a safety limit. This design concept was confirmed quantitatively based on nonlinear pushover analyses before the test using deformation capacity characteristics provided in the AIJ Guideline.

2. In the test, plastic hinges occurred at the bottom of columns and walls in the first story and beam ends of each story in both the X- and Y-directions, as intended in the original structural design. However, in the Y-direction, the first-floor wall finally failed in shear, which did not match the design, while the first-floor walls in the X-direction failed in flexure. The maximum strength and deformation capacity of the structure was higher than that of the analytical result both in the X- and Y-directions.

3. As a result of the evaluation of the collapse mechanism and the safety limit of the test structure based on observed damage levels in members and input magnifications in shakings, the collapse mechanism of the X-direction frame was evaluated as wall-dominant and the wall failure point corresponded to the safety limit state, similar to the result of analyses before the test. However, the test result was not so clear compared to the analytical result because of the high residual strength of walls after failure, which was not considered in the analytical evaluation. In the Y-direction, the collapse mechanism was wall-dominant and the wall failure point corresponded to the safety limit, unlike what was predicted in the analytical results. It is thought that the accumulated prior damage of columns and beams from former shakings degraded the seismic performance of the frame after the wall collapse, which was not considered in the analytical evaluation.

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