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POST-EARTHQUAKE CAPACITY EVALUATION OF RC FRAME STRUCTURES WITH MULTI-STORY FLEXURAL WALLS

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Abstract

After an earthquake, it is important to evaluate the residual seismic capacity of damaged buildings in order to determine the necessity of repair and retrofit and to make an efficient recovery plan. In this regard, a Japanese guideline developed by the Japan Building Disaster Prevention Association (JBPDA) "Standard for Post-earthquake Damage Level Classification of Buildings " [1] is currently in use in Japan to estimate the residual seismic capacity based on an index named R. The R-index represents the ratio of seismic capacity before and after the earthquake. The R-index calculation is intended to be a simple seismic evaluation method that does not require complicated analysis. However, this simplified method is based on the assumption that the ultimate deformation capacity of all members is the same; thus, it is not practical for assessing dual systems, such as reinforced concrete (RC) buildings containing both moment resisting frames and walls. The main objective of this study is to propose a new evaluation method to determine the R-index for buildings of mixed structure types. The proposed method uses two factors: (i) explicit consideration of different deformation capacities of structural members, θ_u, and (ii) a seismic capacity reduction factor, η_W [2], which considers the hysteretic energy absorption reduction of each structural member based on the observed level of damage. The available JBDPA method and the proposed methods are assessed using results obtained from a shake-table test of an RC building.

Firstly, the proposed simplified calculation method is explained in detail. Secondly, the results from a 1/4 scale of a 4 storied RC frame-wall shaking table test are summarized (conducted jointly by Tohoku University and Obayashi Corporation). The 4-storey specimen was designed with different frame and wall strength contributions in the longitudinal and transverse directions to investigate the impact this has on the prediction of residual seismic capacity. The RC walls were designed to fail first, followed by the formation of a 'strong column-weak beam' frame sway mechanism. Finally, each simplified calculation method (the proposed method and the existing Japanese guideline method) are applied to a building model of the shake-table specimen and the accuracy is verified by comparing the result of each evaluation method with the result of the experiment.

In general, results showed that both methods identified the correct tendency of residual seismic capacity observed from the experimental values. In the X-direction, the evaluation accuracy of the *R*-index was higher than for the proposed method than for the standard method. In the Y-direction, the evaluation accuracy of both methods was not conservative relative to the experimental results data. It is considered that over prediction of the *R*-index in the Y-direction is due to the shear failure mode of the wall, compared to the implicit flexural assumption used in determining the θ_u and η_W factors. Therefore, in order to improve the accuracy of the estimation of the seismic residual capacity ratio of structures, it is necessary to propose a seismic performance reduction factor that takes into consideration the expected damage progression and failure mechanism of the members.

Keywords: Post-Earthquake Capacity Evaluation; Shaking Table Test; Multi-story Shear Wall;

1. Introduction

After an earthquake, it is important to evaluate the residual seismic capacity of damaged reinforced concrete (RC) buildings in order to understand the necessity of repair and retrofit, and to make an efficient recovery plan. In this regard, a Japanese guideline developed by the Japan Building Disaster Prevention Association "Standard for Post-earthquake Damage Level Classification of Buildings " [1] (hereafter referred to as the 'JBDPA Guideline') is currently in use in Japan to estimate the residual seismic capacity based on an index named R. The R-index represents the ratio of seismic capacity before and after the earthquake. The R-index calculation is intended to be a simple seismic evaluation method that does not require complicated analysis. Calculation of R in the JBDPA Guidelines, R_{JBDPA}, is done through Eq.(1), which is based on the concept of the internal work and is a weighted sum of the ultimate resistance (ultimate shear force, Q_u, or bending moment, M_u) of members (Fig. 1), where the weighting factors are the seismic performance reduction coefficient, η, listed in the JBDPA Guideline [1] [3] (Table 1). The reduction factor η was empirically determined as the ratio of remaining energy dissipation capacity to the total energy dissipation capacity of components based on their force-displacement backbone curves. This simplified calculation method is based on the assumption that the ultimate deformation capacity is the same for all members (regardless of failure mode); thus, it is thought to not be accurate for assessing dual systems, such as reinforced concrete buildings containing both moment resisting frames and walls.

$$R_{JBDPA} = \frac{\sum(M_{uC}\eta_C) + \sum(M_{uG}\eta_G) + \sum(M_{uW}\eta_W)}{\sum(M_{uC}) + \sum(M_{uG}) + \sum(M_{uW})} \quad (1)$$

Where the subscripts C, G, and W refer to columns, beams and walls.

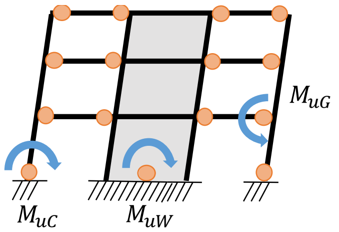


Fig. 1 – Target structure in the guideline

Table 1 – Seismic performance reduction coefficient η

Level	Column	Wall	Beam
I	0.95	0.95	0.95
II	0.75	0.7	0.75
III	0.5	0.4	0.5
IV	0.2	0.1	0.2
V	0	0	0

The main objective of this study is to investigate a proposed alternative evaluation method to determine the R-index for buildings of mixed structure types. The proposed method uses two factors: (i) a contribution factor, θ_u, to evaluate the difference of strength and deformation of each member according to the expected failure mode and (ii) a seismic capacity reduction factor, η_W, which considers the hysteretic energy absorption based on the failure mode of the structural member. The seismic residual capacity calculated from the experimental results from a shake table test carried out in 2019 will be compared with the residual seismic capacity R calculated by the proposed method as well as the standard method.

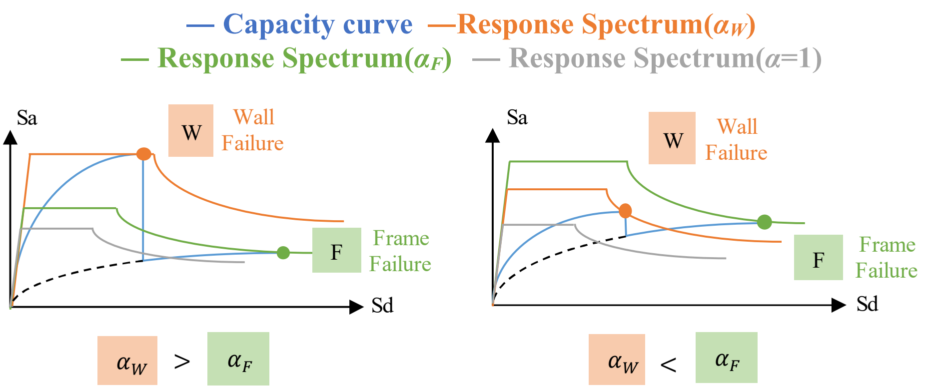
2. Proposed method for estimating residual seismic capacity

An alternative method is proposed to estimate the residual seismic capacity, R, of RC moment resisting frames (hereafter referred to as 'frames') with structural walls. First, the dominant failure mode of the structure is determined (either dominated by wall failure or frame failure), then based on the failure classification the appropriate method is used to determine the proposed residual capacity index R_p.

2.1. Determination of dominant failure mode

RC buildings often utilize several structural component types with different deformation capacities for seismic resistance. For example, for a dual system consisting of an RC frame and RC walls, the response of the structure will have two distinct regions corresponding to frame failure and wall failure, as shown in Fig. 2 (in spectral coordinates). In such cases, the safety limit drift could be determined as the point when the dominant lateral resisting system reaches its ultimate capacity, where the dominant system is defined as one that requires the highest earthquake intensity to initiate failure. In the capacity spectrum shown in Fig.2-a), the wall system requires a higher earthquake demand than the frame system; thus, the structure is considered to be a wall dominant structure. Comparatively, in Fig.2-b) the frame requires a larger earthquake intensity to induce failure compared to the intensity that induces wall failure; thus, even though failure of walls occurs, the frames can still carry the lateral load, and the safety point is taken at failure of frames (frame dominant).

A safety limit evaluation method was proposed by Matsukawa et al [4], where the safety limit is defined as the seismic capacity index, α, (defined in the AIJ Guidelines for Performance Evaluation of Earthquake Resistant Reinforced Concrete Buildings [5]) that results in the structural response reaching its ultimate deformation capacity. The seismic capacity index, α, is defined as a ratio of the intensity of a 5% damped response acceleration spectrum passing through a certain point on the capacity spectrum curve to the intensity of the 5%-damped design demand spectrum (provided in the Japanese Ministry of Construction Notification No.1457 [6] 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. In this study, in order to obtain the capacity spectrum curve of the structure following wall failure (i.e., frame only response), the reduction of strength due to wall failure was considered by modelling the wall plastic hinge with a pin. Using this model, multiple pushover analyses were carried about based on the method using load increment analysis method proposed by Hao et al [2].



(a) Wall dominant failure mode (b) Frame dominant failure mode

Fig. 2 – Determining the dominant failure mode of dual structures.

The points of the capacity spectrum curve at which the structural wall members or frame members (columns or beams) reach their ultimate deformation capacity are defined as the wall failure point (W) and the frame failure point (F), respectively, as shown in Fig. 2. When the seismic performance index α_W at point W is larger than the seismic performance index α_F at point F, the safety limit becomes point W and the dominant failure mode is judged to be RC wall-governed. Likewise, when α_F is larger than α_W, the safety limit becomes point F, and the dominant failure mode is judged as frame-governed.

2.2.Modified reduction factor η_W

In the proposed method, a new seismic performance reduction factor η_W is proposed as an improvement to the seismic performance reduction factor η used in the JBDPA Guidelines. The proposed reduction factor is thought to be more accurate since it explicitly accounts for reduction of strength (η_b), deformation capacity(η_d) and damping (η_h) as proposed by Hao [2] and Ito [7] based on experimental data. The proposed reduction factors η_b, η_d, η_h are summarized in Table 2. Using these three factors, Eq. (2) can be used to calculate the modified residual reduction factors η_W of the damaged member.

$$\eta_w = \eta_b \times \eta_d \times \eta_h \tag{2}$$

Table 2 — Proposed seismic performance reduction factors used for columns, beams and walls.

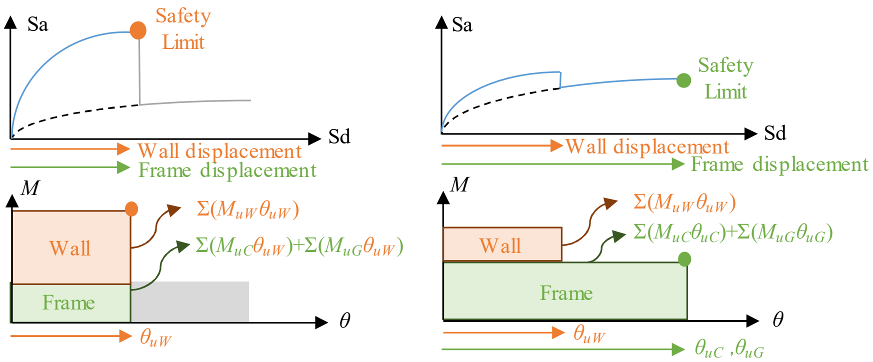
Level	Strength	Deformation capacity	Damping	Energy
	η_b	η_d	η_h	η_w
I	1	1	0.95	0.95
II	1	0.95	0.80	0.76
III	1	0.85	0.75	0.64
IV	0.6	0.75	0.70	0.32
V	0	0	0	0

2.3. Calculation of residual seismic capacity *R* index by the proposed method

In this study, the *R* index is defined as the residual ratio of internal work, which is the same concept used in the JBDPA Guidelines [1]; however, it is modified to account for the different deformation capacities of each structural member. The original energy dissipation capacity of a member is estimated as the product of the members ultimate moment capacity (M_u) and ultimate rotation capacity (θ_u). Subsequently, the member residual energy dissipation is the product of the member original energy dissipation capacity and the proposed seismic performance modification factor ($\eta_w \theta_u M_u$). The proposed calculation method for R_p for a structure with frames and multiple structural walls is shown in Eq. (3), where the fracture mode member contribution coefficient, θ_u, considers the difference in the deformation capacity of the wall and the frame, and is described below. As before, subscripts *W*, *C*, and *G* correspond to walls, columns, and beams, respectively.

$$R_p = \frac{\Sigma(\theta_{uC} M_{uC} \eta_{WC}) + \Sigma(\theta_{uG} M_{uG} \eta_{WG}) + \Sigma(\theta_{uW} M_{uW} \eta_{WW})}{\Sigma(\theta_{uC} M_{uC}) + \Sigma(\theta_{uG} M_{uG}) + \Sigma(\theta_{uW} M_{uW})} \quad (3)$$

In this method, the value of θ_u depends on the dominant failure mode. Fig. 3(a) shows the concept of θ_u when wall failure is dominant. As the point of wall failure is taken as the safety limit of the whole structure, the ultimate deformation capacity of the wall, θ_{uW}, is the same as that of the frame. So, the energy capacity of the wall is $\Sigma(M_{uW}\theta_{uW})$ and the energy capacity of the frame is $\Sigma(M_{uC}\theta_{uW})+\Sigma(M_{uG}\theta_{uW})$ as shown in Fig. 3(a). Therefore, when the wall failure mechanism dominates the response of the structure, $\theta_{uC} = \theta_{uG} = \theta_{uW}$ and Eq. (3) can be simplified to a ratio of moment capacities only. Fig. 3(b) shows the concept of θ_u in the case where column-beam failure is the dominant failure mechanism. In a frame-dominant structure, the frame does not fail after the wall member reaches the ultimate deformation capacity. Therefore, the respective deformation capacity angles, of each member θ_{uW}, θ_{uC}, θ_{uG} are used in Eq. (3). The energy capacity of the wall is $\Sigma(M_{uW}\theta_{uW})$ and the energy capacity of the frame is $\Sigma(M_{uC}\theta_{uC})+\Sigma(M_{uG}\theta_{uG})$ as shown in Fig. 3(b).



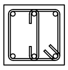
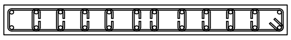
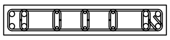
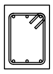
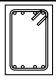
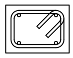
(a) Wall dominant mode (b) Frame dominant mode

Fig. 3 — Energy balance of the whole structure.

3. Design of RC shake table structure

To evaluate the suitability of the proposed method for evaluating the residual seismic capacity against the existing JBDPA Guidelines method, a shake-table test was carried out in 2019 in a collaborative project between Tohoku University and Obayashi Corporation. The test structure was a combined wall and frame 4-storey RC building constructed at a ¼ scale. As a key design parameter, the relative base shear demand of the shear walls and the frame was changed in the X- and Y-directions of the structure such that wall failure was the dominant failure type in the X-direction and frame failure was the dominant failure type in the Y-direction. In other words, the lateral strength of the X-direction was mainly contributed by the walls and in the Y-direction the lateral strength was mainly taken by the frame. Fig. 4 shows the appearance of the as-built test structure, and Fig. 5 shows the dimensions of the plan and elevations of the structure. Cross-section details of each member are summarized in Table 3.

Table 3 — Structural member cross-section details.

Name	C1	CW1	CW2	G1	G2	G3
Detail						
Size	130×130	80×700	70×400	100×140	100×150	120×90
Main bar	6-D10	24-D10	8-D13+6-D6	6-D6	8-D6	4-D6
Hoop/ Stirrup	D4@60	D4@60	D4@100 (subhoop D4@50)	D4@60	D4@60	D6@30

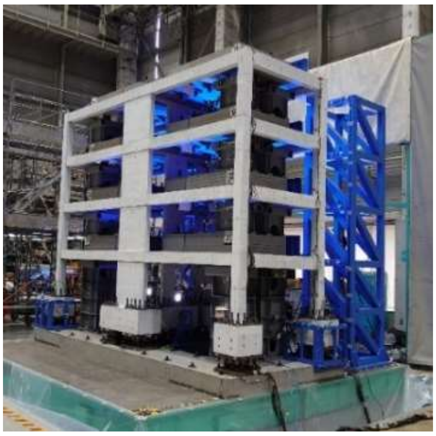


Fig. 4 — Photo of specimen.

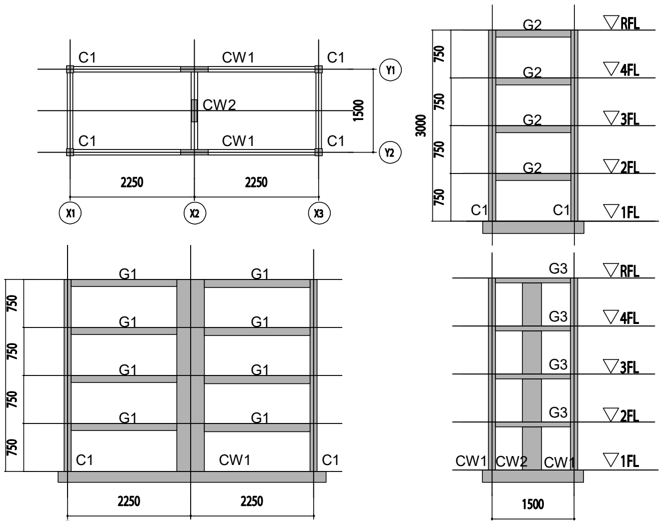


Fig. 5 — Drawing of specimen.

The structure weighted a total of 30 tonnes. Realistic seismic weight was equally distributed to each floor. The structure was designed with story shear demands determined according to the distribution (A_i) prescribed in the AIJ standard [8], which represents the approximate load distribution demands from dynamic analysis. The column to beam flexural moment capacity ratio was designed to be over 1.5 without consideration of effective slab width. As the slab would inevitably increase the beam strength capacity, the likely column to beam moment ratio was lower than this. As a barbell wall (common wall geometry used in Japan) was impractical to construct for a ¼ scale specimen, a rectangular wall with an equivalent flexural stiffness and yield strength was constructed.

4. Ground motion record

The ground motion record used in both the X- and Y-directions of the structure was based on the JMA 1995 Kobe record but modified to match the design spectrum used in the Japanese design criteria [6]. The acceleration record time histories is shown in Fig. 6, and the 5%-damped acceleration response spectra are shown in Fig. 7. The NS component of the record was applied in the X-direction of the structure (longitudinal direction) and EW component was applied in the Y-direction (transverse direction) in all excitation cases. As can be seen in Table 4, a total of nine cases of the ground motion were applied in the X-direction. After the seventh excitation, the Y-direction experienced excessive residual drift, so no further accelerations were applied in this direction.

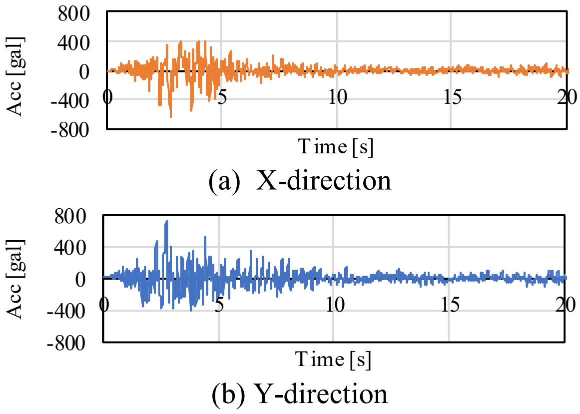


Fig. 6 — The acceleration record time history.

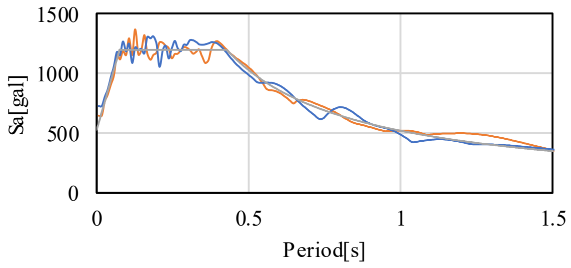


Fig. 7 — The 5%-damped acceleration response spectra of the AIJ Standard and the input ground motion.

Table 4 — List of excitations and their amplification relative to the AIJ Standard design spectrum.

Case	Wave	Amplification (%)	
		X-direction	Y-direction
Run.1	JMA 1995 Kobe (the acceleration response spectrum is Japanese standard)	20	20
Run.2		80	60
Run.3		160	100
Run.4		240	150
Run.5		260	170
Run.6		130	100
Run.7		220	120
Run.8		220	-
Run.9		260	-

5. Preliminary Analysis

Pushover analysis of the specimen was carried out using the nonlinear analysis software SNAP (ver7) [9]. The capacity spectrum of the structure was obtained by converting the system to a single degree of freedom model from the story force-deformation response obtained by the pushover analysis of the full structure.

Each member is modelled by the spring model shown in Fig. 8. A rotational moment spring shown in Fig. 9 was used to model the plastic hinges of each member. A trilinear shear spring was also used to model the wall shear deformation. All other shear springs and axial springs were set as elastic because the test specimen was designed to fail in a flexurally-governed collapse mechanism. The flexural stiffness, flexural cracking strength, flexural yield strength, yield deformation and ultimate deformation were calculated using the AIJ "Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept" [10], and are summarized in Table 5. The rigid zone length of each column and wall was assumed to extend from the member centre line to 1/4 times the member depth from the face of the connecting member. The slab effective width for all beams was assumed to be 0.5 times the length of span perpendicular to the beam longitudinal direction. This resulted in an assumption of 100% total slab span participation in the X-direction and 67% of total slab span participation in the Y-direction. Finally, the safety limit state of the entire building was defined as the step at which either the walls or the frame members reached their ultimate deformation capacity.

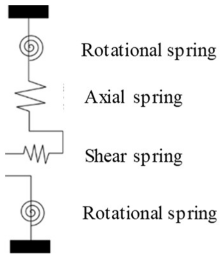


Fig. 8 – Member spring model.

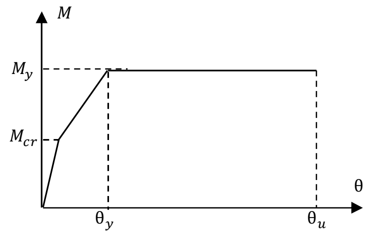


Fig. 9 – Trilinear rotational moment spring model.

Table 5 – Calculated moment and deformation angle used in the SNAP analysis.

		Crack moment	Yield moment	Yield deformation angle	Ultimate deformation angle
		M_{cr} [kNm]	M_y [kNm]	θ_y [rad]	θ_u [rad]
X-dir.	C1	2.04	7.08	0.0048	0.0430
	CW1	31.3	179.0	0.0015	0.0218
	CW2	1.54	12.8	0.0185	0.0440
	G1	3.40	7.22	0.0048	0.0468
Y-dir.	C1	2.04	9.49	0.0056	0.0420
	CW1	3.46	20.8	0.0109	0.0439
	CW2	9.43	78.0	0.0023	0.0227
	G2	3.82	10.4	0.0040	0.0419
	G3	2.07	3.86	0.0021	0.0600

After the walls reached their ultimate deformation capacity, the bottom of the walls were subsequently modelled as pin connections (i.e., assuming their lateral capacity had completely diminished), then pushover analysis was carried out again. Fig. 10 shows the capacity spectrum curves of the entire structure, including the demand spectrums that would cause the structure to reach the safety limit. The safety limit state is the point at which either the walls or frame reach their ultimate deformation capacity (as explained previously in Fig. 3). Using the methodology described in section 2.1, it was confirmed that the X-direction failure mode was governed by wall failure and the Y-direction governed by frame failure, as intended by the design.

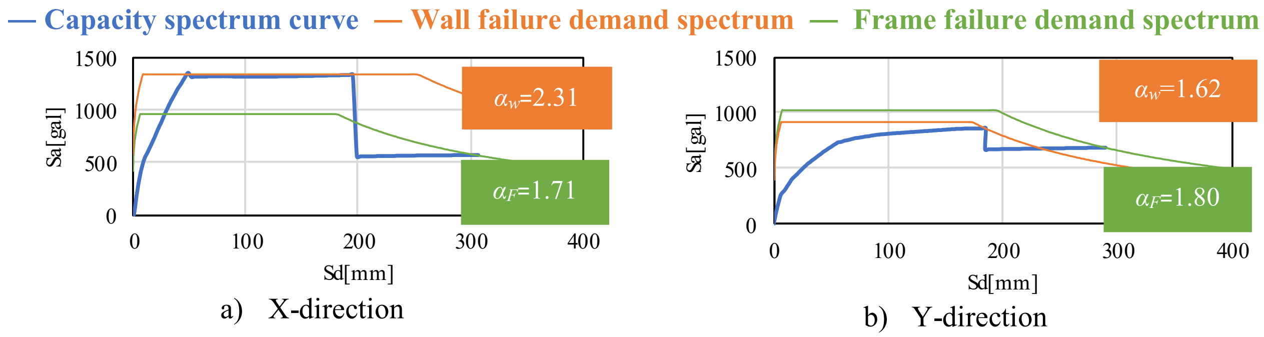
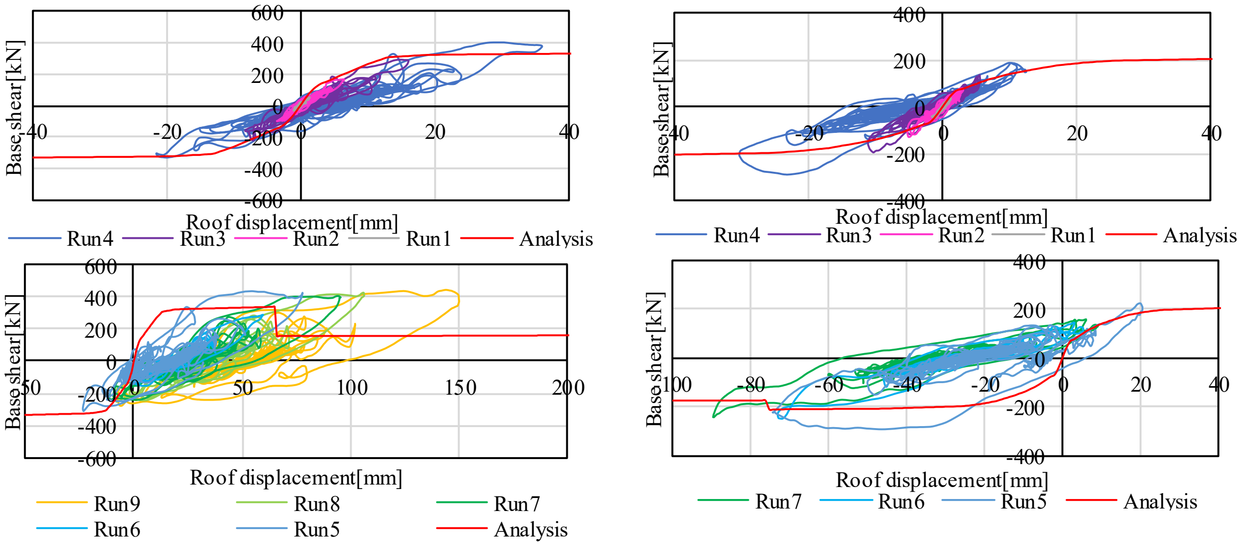


Fig. 10 – Capacity spectrum and demand spectrum curves for the structure from dynamic analysis.

6. Results of Experiment

The relationship between the horizontal roof displacement and the base shear for each run is shown in Fig. 11 together with the results of the preliminary nonlinear pushover analysis. The experimental results showed that the structure's yield strength was 126% times higher in the X-direction and 144% times higher in the Y-direction than the preliminary analysis. In the preliminary analysis, it was considered that the deformation capacity of the wall in the X-direction would be exceeded before the ultimate strength of the frame was attained; however, because of high wall deformation capacity, the flexural capacity of the frame was also reached before the wall had experienced failure.

The damage observed of each member after Run.9 is shown in Fig. 12. The behaviour of the test specimen in both the X- and Y-directions was almost within the elastic range for Run 1 (X:20%; Y:20%) and Run 2 (X:80%; Y:60%). For Run 3 (X:160%; Y:100%), shear and flexural cracks became apparent and after Run 4 reinforcement yielding occurred at each beam, first-floor column and wall; thus, the intended yielding mechanism was formed. After Run 5 (X:260%; Y:170%) the deformation of the test structure increased greatly, and spalling of concrete was observed in first floor columns and walls. Damage did not increase significantly after Run 6, but became more severe in each member following Run 7. After Run 9, rebar buckling was observed in the first-floor walls, and spalling of concrete became severe in the first floor columns and walls. Large cracks (~3.5mm) were found in the beams. In the X-direction, the columns (C1; Fig. 12(a)), beams (G1; Fig. 12(d)) and walls (CW1; Fig. 12(b)) had fully formed plastic hinges. In the Y-direction columns (C1; Fig. 12(a)), and beams (G2, G3) also formed flexural plastic hinges, while the wall (CW2; Fig. 12(c)) had undergone a diagonal shear failure. However, since the main reinforcement of CW2 had yielded and deformation was distributed in each story, it was considered that CW2 failed in shear immediately after flexural yielding. Based on these observations, the collapse mechanism of the test specimen was concluded to be a total collapse mechanism (frame sway mechanism) in both the X- and Y-directions.



a) X-direction

b) Y-direction

Fig. 11 – The relationship between horizontal roof displacement and base shear of the experiment.

(a) C1 (X1-Y1 section)

(b) CW1 (X2-Y1 section)

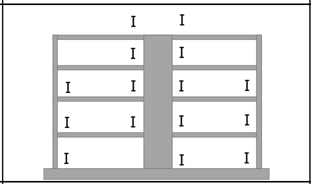
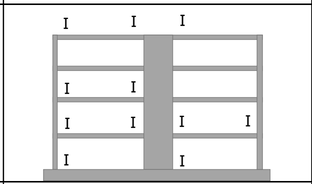
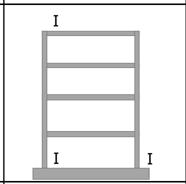
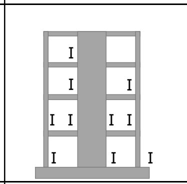
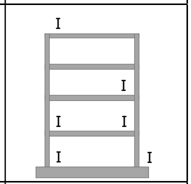
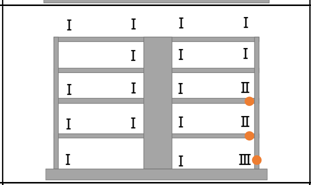
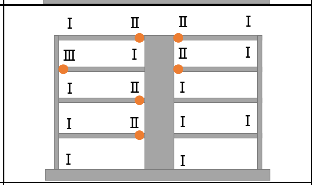
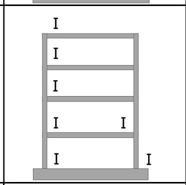
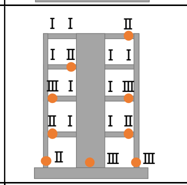
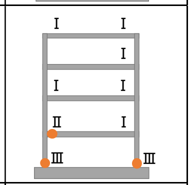
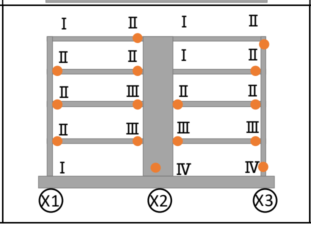
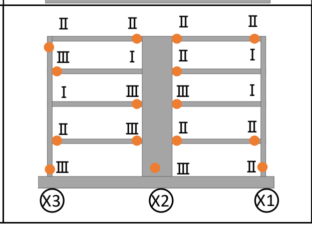
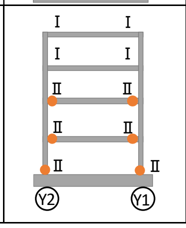
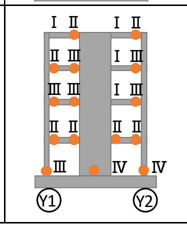
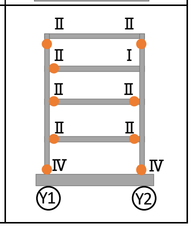
(c) CW2 (X2-Y1a section)

(d) G1 (2nd floor)

Fig. 12 – The damage observed in each member after Run 9.

For each excitation, the degree of damage to each member was determined based on five levels according to criteria provided in the JBDPA Guidelines [1], where level I represent slight damage and level IV represent very severe damage. Table 6 shows the degree of damage to each component after Run 3 (just cracking), Run 4 (structure yielding), and Run 5 (significant damage).

Table 6 — Damage level of structure following Run 3, Run 4 and Run 5.

	Y-1	Y-2	X-1	X-2	X-3
Run3					
Run4					
Run5					

7. Evaluation of Residual Seismic Capacity

7.1. Evaluation method of residual seismic capacity from the experimental result

In order to have a point of reference, the residual seismic capacity derived from the experimental results, R_e, was determined based on the definition from Maeda et al. [11]. Maeda et al. [11] defined the residual seismic capacity ratio as the ratio of the earthquake intensity causing failure of a damaged building to the earthquake intensity required to achieve the same failure for an undamaged building. As several excitations were used in the experiment prior to failure of the structure, the R-index following each run could not be determined by strictly following the R-index definition provided by Maeda et al. Thus, instead of using the failure of the structure as the reference point, the R-index was determined by comparing the damaged building performance and original building performance using the maximum response of the *following excitation* as the reference point. The method for calculating the residual seismic capacity following Run N is described below with reference to Fig. 13.

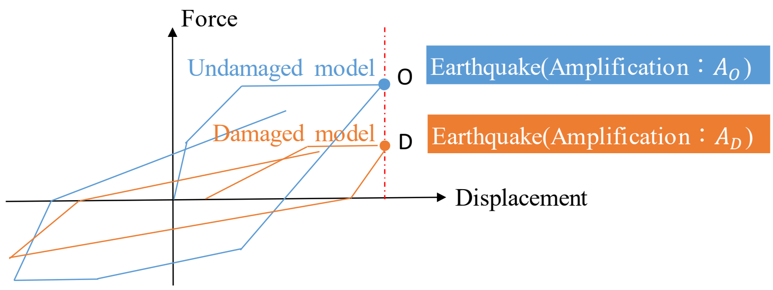


Fig. 13 — The residual seismic capacity ratio based on the magnitude of the demand spectrum.

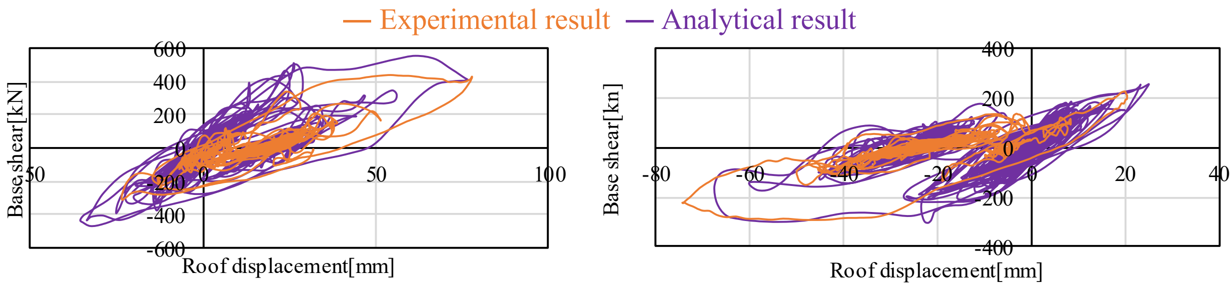
1) The point D is the maximum response deformation point in the experimental results of the excitation case Run N + 1. The excitation amplification is denoted as A_D.
2) An analysis model in SNAP [9] is calibrated such that it reproduces the experimental results based on continuous analysis (i.e., the experimental excitation sequence is applied to model progressive damage degradation).

3) The undamaged version of the calibrated model is then directly analysed with the Run N+1 excitation but with increasing amplification until it produces the same maximum deformation demand as point D. The excitation amplification factor that produces this deformation from a single excitation is termed A_O.
4) The experimental residual seismic capacity ratio is calculated by Eq. (4).

$$R_e = A_D / A_O \quad (4)$$

7.2. Application of the Evaluation Method of Residual Seismic Capacity

The analytical model developed by Tabata et al. [12] was used for dynamic analysis to reproduce the experimental results. Since a single calibrated model was unable to perfectly reproduce the experimental results for every excitation, the model was modified following each run. Thus, different models are available for each excitation, and the accuracy of each model to predict the deformation demand for each run is verified. The analytical model closest to the experimental values for each excitation was used to determine R_e for that excitation case. As an example, Fig. 14 shows the comparison between experimental and analytical results of Run 5 (continuous analysis from Run 1-5), using the model from Run 1-4 model in the X-direction and Run 1-5 model in the Y- direction.



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

Fig. 14—Comparison between the analytical results and experimental results.

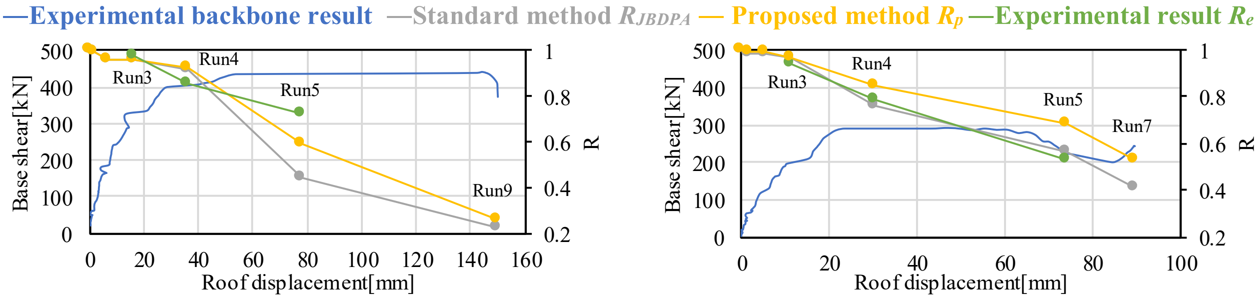
After an appropriate analysis model was determined, the residual seismic capacity, R_e, was calculated at each run. As an example, a comparison between the original (undamaged) model response (using a demand spectrum scale factor of A_O = 3.0 in the X-direction and A_O = 2.6 in the Y-direction) and the damaged model response (using a demand spectrum scale factor of A_O = 2.4 in the X-direction and A_O = 1.7 in the Y-direction) obtained for calculating the R_e of Run 4 is calculated. As a result, the experimental R_e factor in the X- and Y-directions is calculated as 0.87 and 0.71, respectively.

7.3. Result of evaluation

Both the JBDPA Guidelines residual seismic capacity, R_{JBDPA}, evaluation method [1] described in section 1 and the proposed method for R_p described in section 2, were applied following each excitation case. For the proposed method, a dominant failure mode is determined using the pushover analysis results previously shown in Fig. 10, where the failure in the X-direction was wall dominant, and the failure in the Y-direction was frame dominant. Fig. 15 shows a summary of the calculation results of the residual ratio of seismic performance based on each evaluation method. Both R_{JBDPA} and R_p are determined following each run and R_e is determined for Run 3-5.

Overall, it can be seen that JBDPA Guideline residual seismic capacity, R_{JBDPA}, captures trends in experimental residual capacity ratio values, R_e, in both the X- and Y-directions. In the X-direction, R_{JBDPA} is within 2.3% and 7.5% of R_e for Run 3 and Run 4, respectively, as shown in Fig. 15(a). However, R_{JBDPA} of Run.5 was 38.9% lower than R_e; a conservative estimate as expected from a standard. However, in the Y-direction this level of conservatism was not observed as R_{JBDPA} was within 3.3%, 4.0%, and 6.1% of R_e for Run 3, 4 and 5, respectively.

In general, it was confirmed that the proposed method also matched the tendency of experimental R_e values in both the X- and Y-directions. As with the JBDPA Guideline method, Run. 3 and Run. 4 are estimated well, being within 2.3% and 8.3% of the X-direction R_e, respectively. The accuracy of the proposed method R_p in the X-direction of Run 5 is within 17.9% of the experimental R_e, which is better than that of the JBDPA Guidelines while still providing a reasonable conservative buffer. The results in the X-direction are generally consistent with expectations because the X-direction walls (CW1) responded in the expected ductile flexural yielding mechanism. Therefore, it is considered that the estimation accuracy of the proposed calculation method gives a more realistic estimation of R_e compared to the JBDPA Guideline method. In the Y-direction, R_p was not conservative, being higher than the experimental results, R_e, by 3.5%, 7.3%, and 27.9% for Runs 3, 4 and 5, respectively. Even though, the JBDPA Guideline method resulted in a closer estimation to experimental results, it was also not conservative with respect to R_e. This lack of conservatism is because the standard reduction factors used, η, are based on the analysis assumption that all members respond purely in flexure, whereas the primary damage mechanism and failure mode of the wall CW2 in the Y-direction was shear. Shear response of members results in a more rapid strength reduction compared to flexural response; thus, the proposed method and JBDPA method θ_u and η factors would predict a slower rate of strength reduction, and consequently a higher residual capacity factor than observed in reality.



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

Fig. 15 – Residual seismic capacity calculated using the JBDPA Guidelines and the proposed evaluation method compared to experimental residual capacity ratio.

8. Summary

RC buildings with dual systems (RC frame and wall) have members with different deformation capacities. The safety limit state would change based on which structural system (frame or walls) is dominant in the seismic response of the structure. The residual seismic capacity of dual systems is not considered in the existing standard [1], which considers only the case of structural members having similar deformation capacity. In order to accurately evaluate the residual seismic performance of an RC frame with flexural walls, a calculation method was proposed which is capable of estimating the residual capacity depending on the dominant failure mode of the structure and the energy absorption history of each member. The accuracy of the evaluation of the seismic performance residual capacity ratio of the existing standard method and the proposed method was compared to the seismic residual capacity ratio calculated from the experimental results of a shaking table test of ¼ scaled 4 storied RC structure with a dual system. As a key design parameter of the test specimen, in the X-direction the seismic capacity of the structure was mainly contributed by shear wall, but in the Y direction the frame was the dominant lateral load resisting system. The following are the main findings:

1- Both methods (the JBPDA Guideline and the proposed method) were capable of capturing the general tendency of the residual seismic capacity ratio as observed from the experimental values.

2- In the X-direction, the residual seismic capacity evaluation accuracy of the proposed method was closer to the experimental results than of the JBDPA Guideline method; however, both methods gave conservative estimates at severe damage states. In the Y-direction, the same level of conservativeness was not observed for either method. This lack of conservatism in the Y-direction is thought to be due to the difference in deformation capacity and failure mode of the RC walls in the experiment (which failed in shear) compared to the implicit flexural assumption used in the values of θ_u and η_W factors.

3- The seismic response and failure mode of the flexural wall greatly influenced the results of both the proposed method as well as the JBDPA Guideline method for the evaluation of residual seismic capacity. Therefore, in order to improve the accuracy of the estimation of the residual seismic capacity it is necessary to propose seismic performance reduction factors (for strength, deformation capacity and damping) that take into consideration the expected damage progression and failure mechanism of structural members.

9. Acknowledgement

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