RESIDUAL CAPACITY AND DAMAGE EVALUATION OF RC STRUCTURES BASED ON CAPACITY SPECTRUM METHOD

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Abstract

After an earthquake, a large number of building structures will be damaged to different levels. The subsequent decision-making process for restoration and reconstruction of damaged buildings requires an appropriate and practical residual seismic capacity evaluation method. In Japan, a residual seismic capacity evaluation procedure has already been recommended by the Guideline for Post-Earthquake Damage Evaluation and Rehabilitation, and has been widely applied after past damaging earthquakes. In this procedure, the seismic capacity deterioration of structural elements in different damage levels are considered by the seismic capacity reduction factor, η. Then the residual seismic capacity ratio of the structure, R, is calculated by the weighted average of η of all the structural elements. Damage to structural elements typically affects strength, deformation capacity and damping characteristics; however, the standard method described above does not consider the latter two effects for simplicity. Additionally, the current method implicitly assumes either a storey collapse or overall collapse mechanism will occur, with no consideration towards partial collapse scenarios.

The main objective of this research is to propose a new post-earthquake damage evaluation method based on the capacity spectrum method that is applicable to buildings with all types of collapse mechanisms. Thus, a procedure is proposed that allows engineers to draw a clear distinction between buildings that are governed by failure of shear components and those governed by failure of flexural components. Additionally, deterioration of strength, deformation capacity and energy dissipation of structural elements are all accounted for in the capacity curve of a damaged building. Taking all this into account a residual capacity ratio, R, can be evaluated as a ratio of seismic capacity after damage to the original capacity. The proposed new method is applicable to many types of buildings, provided an initial capacity curve can be obtained by push-over analysis.

Keywords: RC buildings; Earthquake structural damage; Post-earthquake capacity evaluation; Residual capacity.

1. Introduction

To restore an earthquake-damaged community as quickly as possible, a well-prepared reconstruction strategy is essential. When an earthquake strikes a community and destructive damage to buildings occurs, immediate damage inspections are needed to identify the safe and unsafe buildings given the likelihood of aftershocks following the main event. However, since such quick inspections are performed within a restricted short period of time, the results may be inevitably coarse. Furthermore, it is not generally easy to identify the residual seismic capacity quantitatively from quick inspections. Following the quick inspections, a damage assessment should be more precisely and quantitatively performed, and then technically and economically sound solutions should be applied to damaged buildings, if rehabilitation is needed. To this end, a technical guideline that may help engineers find appropriate actions required for a damaged building is needed.

In Japan, the Guideline for Post-Earthquake Damage Evaluation and Rehabilitation [1] (subsequently referred to as the Damage Evaluation Guideline) was originally developed in 1991 and was revised in 2001 and 2015 considering damaging earthquake experiences in Japan [2]. The main objective of the Damage Evaluation Guideline is to serve as a technical basis and to provide rational criteria when an engineer needs to identify and rate building damage quantitatively; determine necessary actions required for the building and provide technically sound solutions to restore the damaged building. In the Damage Evaluation Guideline, residual capacity ratio R is defined as a ratio of seismic capacity of a building after damage to that before damage, where the seismic capacity is evaluated based on the seismic capacity index I_s provided in the Japanese Seismic Evaluation Standard [3]. Moreover, a simplified evaluation method of R index is introduced based on structural energy dissipation capacity for story collapse mechanism and overall collapse mechanism as shown in Fig. 1(a) and (b) [2,4]. In case of story collapse mechanism (Fig. 1(a)) of an undamaged frame, the internal work can be evaluated as the work done by the deformation of shear columns ($Q_{ui}\ \theta_d\ L_i$, where Q_{ui}: shear strength in i-th column, θ_d: story drift angle and L_i: story height) in the collapsed story. In case of overall collapse mechanism (Fig. 1(b)), internal work is the work done by the rotation of all the plastic hinges ($M_u\ \theta_d$, where M_{ui}: ultimate flexural moment in i-th plastic hinge). Internal work after damage is evaluated by reducing Q_{ui} and M_{ui} by capacity reduction factor η. The residual capacity ratio R is then given by Eq. (1) and (2).

$$R = \frac{\theta_d \cdot \sum \eta_i Q_{ui} L_i}{\theta_d \cdot \sum Q_{ui}\, L_i} = \frac{\sum \eta_i Q_{ui}}{\sum Q_{ui}} \text{ (story collapse)} \tag{1}$$

$$R = \frac{\theta_d \sum \eta_i M_{ui}}{\theta_d \sum M_{ui}} = \frac{\sum \eta_i M_{ui}}{\sum M_{ui}} \text{ (overall collapse)} \tag{2}$$

On the other hand, partial collapse mechanism with combination of ductile flexural and brittle shear structural members (Fig. 1(c)) is not considered in the Damage Evaluation Guideline, although it may be often found in observation of damaged buildings. In this study, an evaluation method of residual seismic capacity ratio based on the response spectrum method (CSM) [5,6] is discussed first for a building with all types of collapse mechanisms. Secondly, an approximation method of residual capacity ratio R is proposed based on the virtual work principle discussed in the authors' previous research [4, 5].

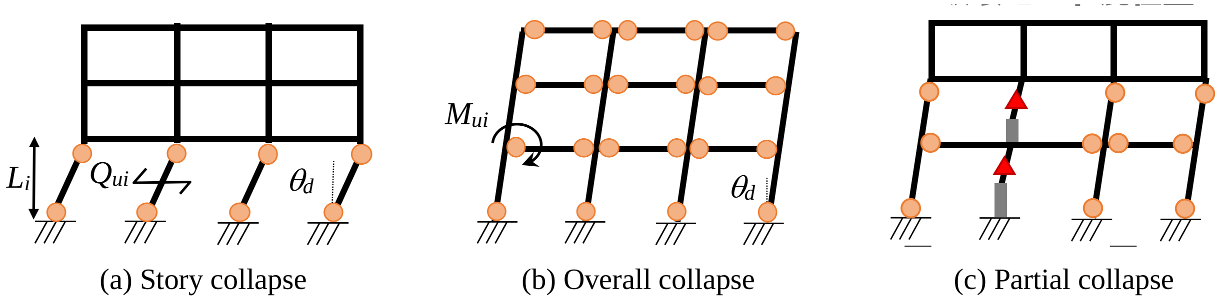


Fig. 1 – Possible collapse mechanisms considered in this study.

2. Residual Seismic Capacity Evaluation Based on Capacity Spectrum Method

The residual ratio of seismic capacity R is defined by Eq. (3) as the ratio of the seismic capacity at the safety limit state of a damaged structure to that of the undamaged structure. There are various ways to define 'seismic capacity', for example, the Damage Evaluation Guideline [1] uses the seismic capacity index *Is* [3]. In addition, approximated evaluation method is introduced based on the authors' research on Internal Work (IW) concept in the Virtual Work Theory [4, 5].

$$R = \frac{Seismic\ capacity\ of\ damaged\ structure}{Seismic\ capacity\ of\ undamaged\ structure} \tag{3}$$

For a more accurate evaluation, Hao et al. [5] and Miura et al. [6] developed an alternative evaluation method based on the Capacity Spectrum Method (CSM), where seismic capacity is defined as the scale factor of the standard seismic response spectrum required to make the structure to reach its safety limit state. Fig. 2 shows the general flow of evaluation by CSM. First, push-over analysis of a target building frame is performed. Safety limit state and damage condition (level/class) is judged based on the performance curve obtained from the push-over analysis. The seismic capacity before damage (undamaged structure), α, is evaluated by amplifying the seismic response spectrum so that response agrees with requirement in accordance with Japanese Seismic Performance Evaluation Guideline [7]. For the damaged condition, capacity (strength, ductility, energy dissipation) of each structural member is reduced based on damage class (judged using the Damage Evaluation Guideline) and pushover analysis is conducted to obtain a damaged capacity spectrum curve. The seismic capacity for the damaged condition ${}_D\alpha$ is then evaluated by CSM using this damaged curve.

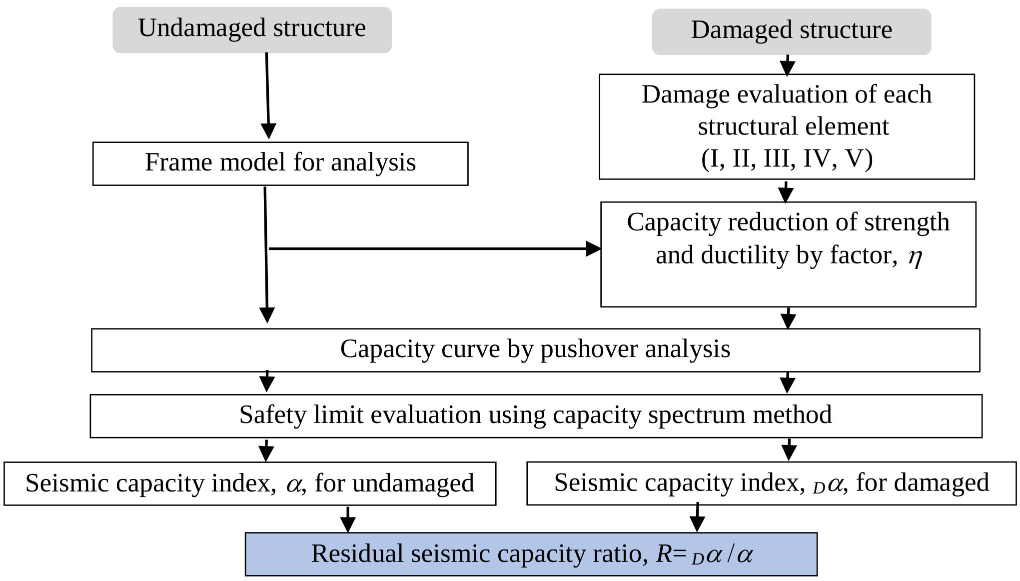
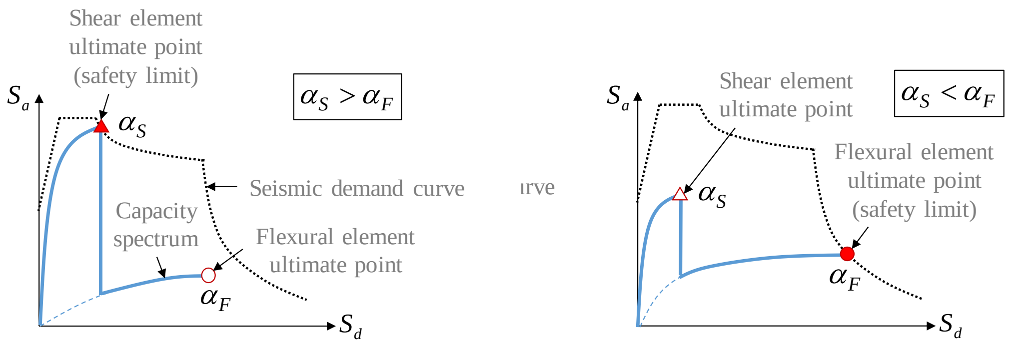


Fig. 2 – Flow of residual seismic capacity ratio evaluation by CSM.

2.1 Judgement of dominant failure mode and safety limit state

Matsukawa and Maeda [8] proposed a judgement method of dominant structural elements which govern the seismic capacity of buildings that contain both brittle and ductile structural elements. Fig. 3 shows the general concept of judgement. For simplification, a building is assumed to consist of structural elements with two levels of deformation capacity corresponding to failure of shear and flexural elements, respectively, as shown in Fig. 3. The Standard seismic response spectrum is amplified to meet the capacity curve and the amplification factor, α, is defined as seismic capacity in accordance with the Japanese Seismic Performance Evaluation Guideline [7]. After α_S (amplification factor corresponding to failure of shear elements) and α_F (amplification factor corresponding to failure of flexural elements) are determined, their ratio $M_d = \alpha_S / \alpha_F$ is used as a judgement factor of dominant failure mode. If M_d is greater than 1 ($\alpha_S > \alpha_F$), shear failure is considered as the dominant collapse mechanism. On the other hand, if M_d is less than 1 ($\alpha_S < \alpha_F$), flexural failure is assumed dominant. The larger of α_S and α_F defines the undamaged structure's seismic capacity α.



(a) Shear dominant structure (b) Flexure dominant structure

Fig. 3 – Judgement of dominant collapse mechanism and safety limit state.

2.2 Evaluation of residual seismic capacity ratio

In order to evaluate the residual seismic capacity after damage, $_D\alpha$, the capacity curve of the damaged structure is first estimated by reduction factors η_b and η_d as shown in Fig. 4. Reduction factors η_b, η_d and η_h are modification factors to reduce strength, deformation capacity, and damping depending on the damage class of each structural element. Damage level of structural elements are classified into five class (slight damage (I) to total failure (V)) in the Damage Evaluation Standard [1]. The magnitude of these reductions has been studied by Itoh et al [9] and are summarized in Table 1. Ultimate strength (flexural strength M_u and shear strength Q_u) and deformation capacity of members are reduced by η_b and η_d, respectively, as shown in Fig. 4. Then, the capacity curve of the damaged structure is evaluated by push-over analysis to determine the safety limit state as shown in Fig. 5. Note that the dominant collapse mechanism of the damage structure is assumed to be the same as for the undamaged structure.

Table 1 – Capacity reduction factors for damaged structural elements [9].

Damage class	Flexural element				Shear element			
	η_{bF}	η_{dF}	η_{hF}	η_{WF}	η_{bS}	η_{dS}	η_{hS}	η_{WS}
I	1	1	0.95	0.95	1	1	0.9	0.9
II	1	0.95	0.8	0.76	1	0.85	0.7	0.6
III	1	0.85	0.75	0.64	1	0.7	0.6	0.42
IV	0.6	0.75	0.7	0.32	0.4	0.5	0.5	0.1
V	0	0	0	0	0	0	0	0

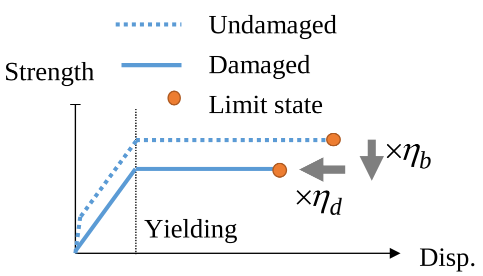


Fig. 4 – Reduction of structural element backbone.

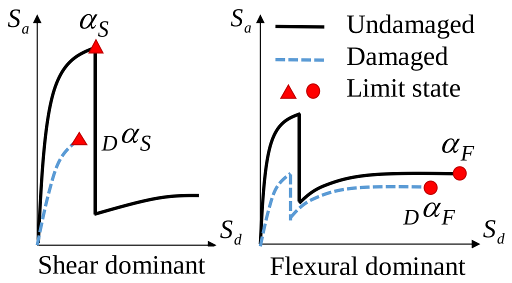


Fig. 5 – Safety limit state of damaged and undamaged structures.

To obtain the seismic demand response spectrum for the CSM evaluation, the standard spectrum is reduced using the factor F_h defined below in Eq. (4). The reduction factor F_h depends only on the equivalent damping factor, h, of the overall structure (defined in Eq. (5)), and is a weighted sum of the damping factors for each member, h_i (defined in Eq. (6)). For a damaged frame, the equivalent damping factor ${}_Dh_i$ of damaged members is reduced by the energy dissipation reduction factor η_h given in Table 1, as shown in Eq. (7). Then, the seismic capacity indices α and ${}_D\alpha$ are obtained from the safety limit point, as shown in Fig. 6 and the residual capacity ratio R is calculated by Eq. (1).

$$F_h = \frac{1.5}{1+10h} \tag{4}$$

$$h = \sum W_{ei}h_i / \sum W_{ei} \tag{5}$$

$$h_i = 0.05 + \beta_h(1 - 1/\sqrt{\mu_i}) \tag{6}$$

$${}_Dh_i = 0.05 + \beta_h\eta_{hi}(1 - 1/\sqrt{\mu_i}) \tag{7}$$

Where, h: equivalent damping factor of the overall structure, W_{ei}: strain energy of a structural member i, h_i and ${}_Dh_i$: equivalent damping factors of the undamaged and damaged structural member i, respectively; μ_i is the ductility factor of member i, and η_{hi}: reduction factor for the equivalent damping factor (Table 1). Coefficient β_h is 0.25 for flexural elements and 0.05 for shear elements [7].

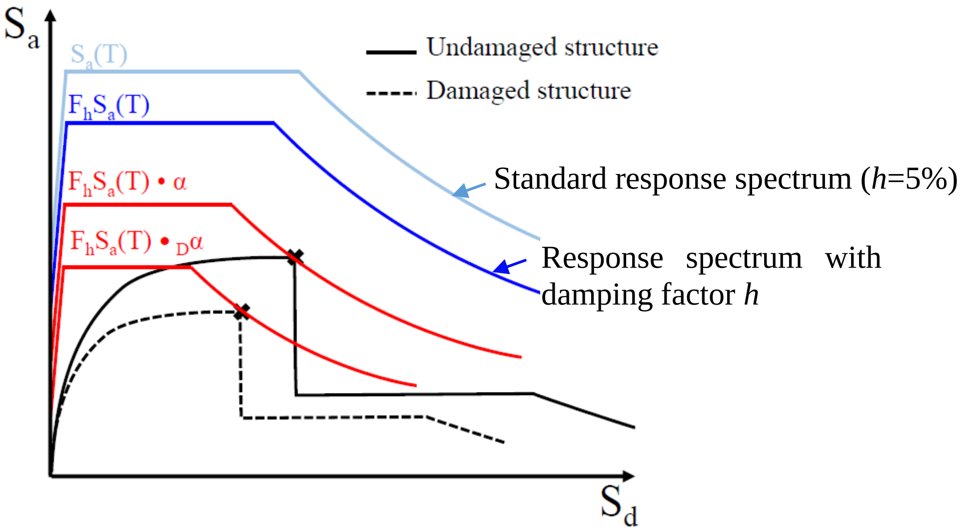


Fig. 6 – Definitions of spectrum damping scale factor, F_h; the undamaged structure safety limit point scale factor α and the damaged structure safety limit scale factor ${}_D\alpha$.

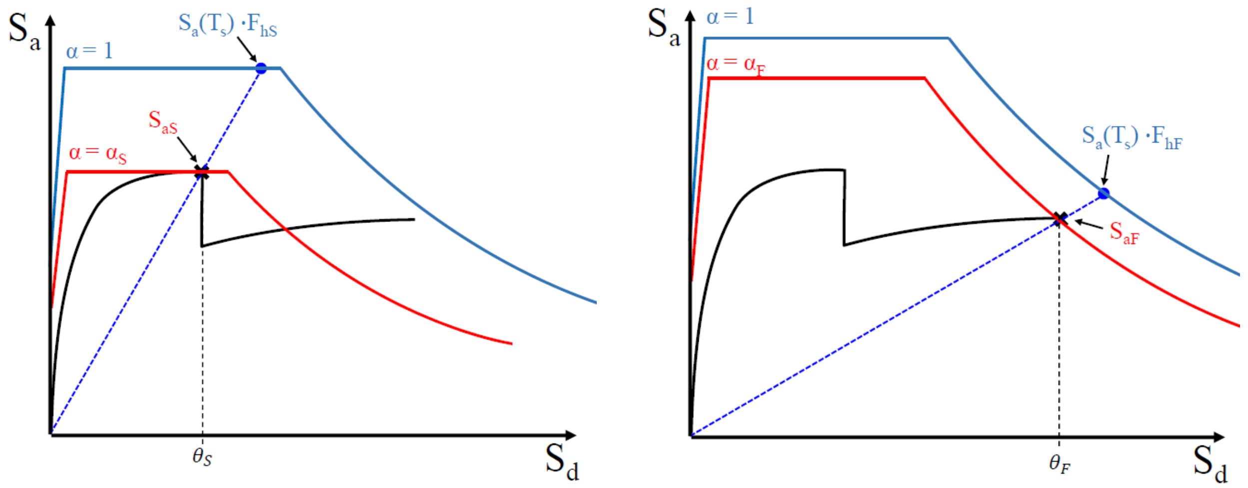
3. Simplified Evaluation of Residual Seismic Capacity

The evaluation method proposed in section 2 requires pushover analysis by computer to calculate the structural response. Although it will provide accuracy, it is rather complicated and thus not suitable for application in field surveys of damaged building just after earthquake disaster. From this background, an approximation of the evaluation method is needed from a practical application point of view. In this section, an approximate method for the calculation of the dominant failure mode determination factor, M_d, is presented. Then, based on the failure mode determined by the M_d factor, an approximate evaluation method of R is proposed.

3.1 Approximated judgement of dominant collapse mechanism

(1) Dominant failure mode judging index M_d

The seismic response spectrum is given in the Notification No. 1457 of Japanese Ministry of Land, Infrastructure and Transport [10]. As before, the safety limit factors for shear and flexural failure of the structure are defined as the scale factor of the response acceleration spectrum resulting in the structure reaching its safety limit state, from the standard response acceleration spectrum scaled for damping only, as shown in Fig. 7a and Fig. 7b for shear and flexural failure, respectively. S_{aS} and S_{aF} are the capacities of a structure, and $S_a(T_S)$ F_{hS} and $S_a(T_F)$ F_{hF} are demands. Based on this definition, Eq. (8) and (9) can be defined for α_S and α_F, respectively. Therefore, dominant failure mode judging index M_d defined as α_S / α_F can be expressed using Eq. (10).



(a) (b)

Fig. 7 – Evaluation of seismic capacity, α for (a) shear dominant and (b) flexural dominant structures.

$$\alpha_s = \frac{S_{aS}}{S_a(T_s)F_{hS}} \tag{8}$$

$$\alpha_F = \frac{S_{aF}}{S_a(T_F)F_{hF}} \tag{9}$$

$$M_d = \frac{\alpha_S}{\alpha_F} = \frac{S_{aS}}{S_{aF}} \cdot \frac{S_a(T_F)}{S_a(T_S)} \cdot \frac{F_{hF}}{F_{hS}} \tag{10}$$

Where, S_{aS} and S_{aF} are the spectral acceleration response at shear failure and flexural failure, respectively; F_{hS} and F_{hF} are the damping response spectrum reduction coefficients; $S_a(T_S)$ and $S_a(T_F)$ are the spectral accelerations at shear and flexural failure of the standard response spectrum (scaled for damping only), respectively.

As can be seen on the right side of Eq. (10), the M_d factor consists of three components; ratio of capacities (S_{aS} / S_{aF}), standard spectral accelerations ($S_a(T_S)$ / $S_a(T_F)$) and damping reduction factors (F_{hS} / F_{hF}). The seismic response spectrum given in the Notification No. 1457 is acceleration constant in the range of 0.16 sec $<T<T_c$, and velocity constant in the range of $T_c<T$ as expressed by Eq. (11). When both of the equivalent periods, T_S and T_F (corresponding to shear and flexural failure, respectively) are shorter than the spectrum corner period T_c, $S_a(T_S)$ and $S_a(T_F)$ both lie in the constant region zone of the response spectrum. Eq. (10) thus simplifies into Eq. (12). On the other hand, when both of T_S and T_F are longer than T_c, the equivalent periods T_S and T_F both fall within the constant velocity region of the response spectrum; thus, Eq. (10) simplifies into Eq. (13). Finally, in the case of $T_S < T_c$ and $T_F > T_c$, as shown in Fig. 8, Eq. (10) can be expressed by Eq. (14). Therefore, dominant failure judging index M_d defined as α_S / α_F can be expressed using Eq. (12) – (14).

$$S_a(T)=\begin{cases} S_{a0} & (0.16\sec < T < T_C) \\ \dfrac{2\pi S_{v0}}{T} & (T_C < T) \end{cases} \tag{11}$$

$$M_d = \frac{S_{aS}}{S_{aF}} \cdot \frac{F_{hF}}{F_{hS}} = \gamma_a \cdot \gamma_h \tag{12}$$

$$M_d = \frac{F_{hF}}{F_{hS}} \sqrt{\frac{S_{aS}S_{dS}}{S_{aF}S_{dF}}} = \gamma_h \sqrt{\gamma_a \gamma_d} \tag{13}$$

$$M_d = \frac{T_c F_{hF}}{T_S F_{hS}} \sqrt{\frac{S_{aS}S_{dS}}{S_{aF}S_{dF}}} = \frac{T_c}{T_S} \cdot \gamma_h \sqrt{\gamma_a \gamma_d} \tag{14}$$

Where, γ_a is S_{aS} / S_{aF}, γ_d is S_{dS} / S_{dF}, and γ_h is F_{hF} / F_{hS}. S_{a0} and Sv_0 are acceleration and velocity response values in the constant region of the standard design response spectrum, respectively. As before, the collapse mechanism is considered to be shear dominated in case of M_d >1 and flexure dominated in case of M_d <1. From here, estimations for the γ_a, γ_d and γ_S are discussed.

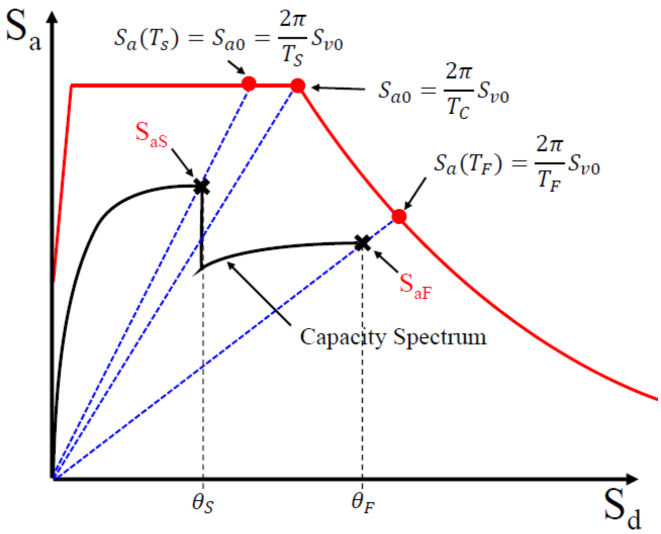


Fig. 8 – Spectral acceleration relationships.

(2) Approximation of strength ratio γ_a

Assumptions of seismic force and deformation distribution along the height of a structure with shear and flexural elements is shown in Fig. 9. According to the principle of virtual work, the work done by the external forces acting on each floor of the frame, P_i, is balanced with internal energy dissipated by all members through force (ultimate flexural moment M_u and shear strength Q_u) and deformation (rotation and displacement). As shown in Fig. 9, the story drift angle of uncollapsed stories is assumed negligible, thus the horizontal displacement of uncollapsed stories is assumed equal to the collapsed story below. For example the structure in Fig. 9 experiencing partial collapse in the first and second stories, the horizontal displacement in the third story is assumed the same as in the second story. The story drift angle of each collapsed story and the rotation angle of flexural beam members is assumed to be the same drift angle, θ_d. The deformation of shear-failing columns is assumed to be $h_i\theta_d$ (h_i is height of i th story). Based on these assumptions, the balance of external and internal work in summarized using Eq. (15), with reference to Fig. 9. By rearranging Eq. (15), base shear of the structure, Q_B, can be expressed as in Eq. (16).

$$\theta_d \sum P_i H_i = \theta_d \sum M_{ui} + \theta_d \sum Q_{ui} h_i \tag{15}$$

$$Q_B = \frac{\sum M_{ui} + \sum Q_{ui} h_i}{\sum H_i} \tag{16}$$

Where, H_i is height of the i th floor from ground, h_i is story height and θ_d is rotation angle of the story.

The response acceleration S_a can be obtained by dividing base shear Q_B by total mass of the frame, m. To determine the response acceleration corresponding to the shear collapse point, S_{aS}, ultimate flexural moment, M_u, was reduced by a reduction factor of β_F (for example, specified as β_F=0.5 for extremely brittle short columns and β_F=0.7 for general shear columns in the Japanese Seismic Evaluation Standard [3]) based on the assumption that flexural members have not reached the ultimate moment capacity when the shear members reach failure. Thus, S_{aS} can be estimated by Eq. (17). The response acceleration corresponding to the flexural collapse point, S_{aF}, is estimated by Eq. (18) where the contribution of shear members is ignored because they have failed earlier. Therefore, γ_a ratio can be approximately calculated as the ratio of S_{aS} and S_{aF} as shown in Eq. (19).

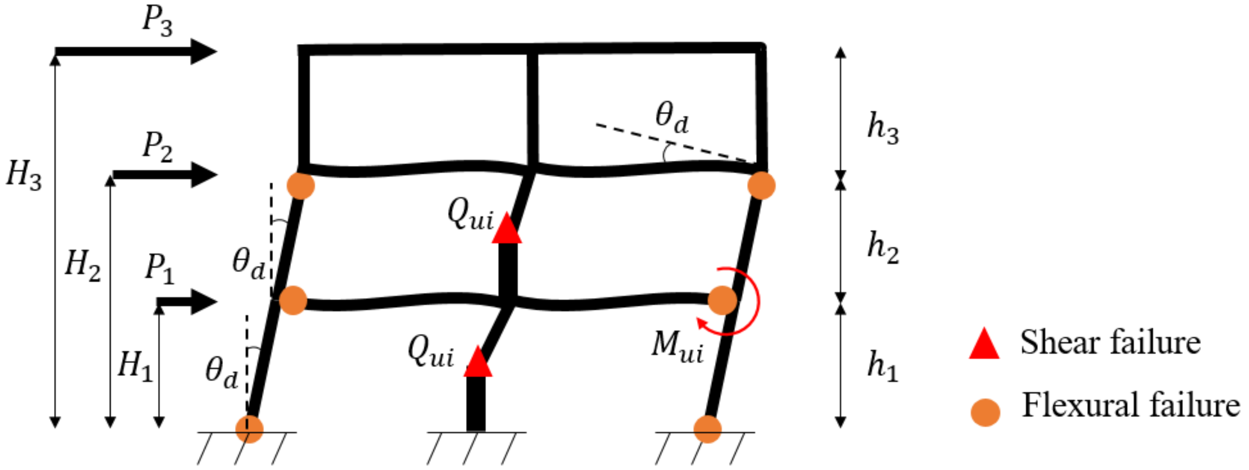


Fig. 9 – Assumptions for approximated evaluation of residual capacity ratio R.

$$S_{as} = \frac{\beta_s \sum M_{ui} + \sum Q_{ui} h_i}{m \sum H_i} \tag{17}$$

$$S_{aF} = \frac{\sum M_{ui}}{m \sum H_i} \tag{18}$$

$$\gamma_a = S_{as}/S_{aF} = \beta_s + \frac{\sum Q_{ui} h_i}{\sum M_{ui}} \tag{19}$$

(3) Approximation of deformation capacity ratio, γ_d

In order to approximate deformation capacity ratio γ_d, displacement response S_d is estimated from Eq. (20) as the product of maximum member deformation capacity, θ_d (equal to θ_S for shear failure or θ_F for flexural failure), at failure and the representative height H_e of the structure (i.e., height of an equivalent single-degree-of-freedom system). The representative height H_e can be calculated from Eq. (21), where $m_i$and δ_i are the mass and deformation of the i-th storey.

$$S_{dS} = \theta_{Si} H_{eS};\ \ S_{dF} = \theta_{Fi} H_{eF} \tag{20}$$

$$H_e = \frac{\sum m_i \delta_i^2}{\sum m_i \delta_i} \tag{21}$$

Assuming that the rotation demands for shear members, θ_{Si}, and flexural members, θ_{Fi}, are a product of the member yielding angle, θ_y, and member ductility factor, μ, the displacement response capacity can be estimated using Eq. (22). The yield rotation, θ_y, of the shear dominant and flexure dominant members was assumed to be 1/250 rad and 1/150 rad, respectively, based on Japanese Seismic Evaluation Standard [3]. Thus, γ_d can be approximated by Eq. (23). Maximum ductility factors of shear members and flexural members can be selected in accordance with their deformation capacities. μ_S=1 to 2 and μ_F = 3 to 5 may be reasonable values from Japanese practice and lessons from previous damaging earthquakes.

$$S_d = \theta_y \cdot \mu \cdot H_e \tag{22}$$

$$\gamma_d = \frac{S_{dS}}{S_{dF}} = \frac{(1/250) \cdot (\mu_S) \cdot (H_{eS})}{(1/150) \cdot (\mu_F) \cdot (H_{eF})} \tag{23}$$

(4) Approximation of damping factor ratio, γ_h

For estimation of the F_{hF}/F_{hS} ratio, it can be shown with Eq. (4)-(6) that F_{hS} is close to 1 when the ductility factor is 2 or less (i.e., shear member failure point). When the ductility factor μ_i is 3 or more, the F_{hF} of the flexural members has little fluctuation, and its upper limit is approximately 0.6. Thus, F_{hS} and F_{hF} are set to 1 and 0.6, respectively, and γ_h can be approximated as γ_h = 0.6.

(5) Approximation of equivalent period ratio T_c / T_S

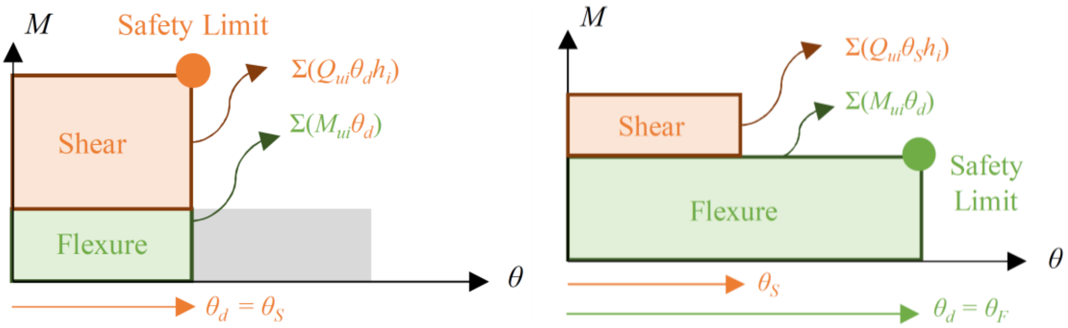
The corner period T_c in the standard response spectrum is 0.864 sec for ground with soil type 2 (standard soil) according to the Notification No. 1457. The equivalent period T_S at the shear failure point is estimated to be approximately equal to the period at structural yield, T_y, as the deformation from the yield point to shear failure is assumed to be small. Meanwhile the structural yield period is roughly calculated as 0.02 times the structure height H, according to Japanese structural design practice given in Notification No. 1793 of Japanese Ministry of Construction [11]. Thus, T_s is defined as shown in Eq. (24), and the equivalent damping ratio, T_c / T_S, can be determined using Eq. (25).

$$T_s \approx 0.02H \tag{24}$$

$$\gamma_s = \frac{T_c}{T_s} = \frac{0.864\ s}{0.02H} = \frac{43.2}{H} \tag{25}$$

3.2 Evaluation of residual seismic capacity ratio R based on internal work method

As described previously in Eq. (15), for a frame with a partial collapse mechanism, such as that shown in Fig. 9, the internal work of an undamaged frame is evaluated as the summation of work done by the rotation of all plastic hinges (M_u θ_d) and deformation of shear columns (Q_u h_i θ_d). This is schematically summarized in Fig. 10, assuming a uniform story drift angle of θ_d for all damaged stories.



(a) Shear failure dominant mode (b) Flexure failure dominant mode

Fig. 10 – Internal work for a building with partial collapse mechanism.

The residual internal work capacity of the damaged frame is evaluated by reducing the original internal work capacity of each member by respective η_{Wi} values. The energy dissipation reduction factor η_{Wi} is proposed in Eq. (26) as a product of the reduction factors for strength η_{bi}, deformation capacity η_{di}, and damping η_{hi}, previously presented in Table 1. The basic concept of historical energy dissipation of undamaged members W_i and damaged members $_DW_i$ is shown in Fig. 11.

$$\eta_{Wi} = {}_DW_i / W_i = \eta_{bi} \cdot \eta_{di} \cdot \eta_{hi} \tag{26}$$

(a) Undamaged member (b) Damaged member

Fig. 11 – Reduction of energy dissipation due to damage in structural members.

As a result, the internal work of a structure with shear and flexural elements is a summation of (Q_u h_i θ_d) and (M_u θ_d), and thus the residual capacity ratio, R, can be evaluated by a combination of Eq. (1) and (2). Weighting factors β_S and β_F are also introduced to account for the difference in deformation capacity of shear and flexural elements resulting in the general expression for R as given by Eq. (27).

$$R = \frac{\beta_S \theta_d \sum Q_{ui} H_i \eta_{WSi} + \beta_F \theta_d \sum M_{ui} \eta_{WFi}}{\beta_S \theta_d \sum Q_{ui} H_i + \beta_F \theta_d \sum M_{ui}} \tag{27}$$

The basic concept of β_S and β_F are shown in Fig. 12. For structures judged to be shear failure dominant (i.e., M_d > 1), at the shear failure point, the flexural elements do not always reach their maximum flexural capacity and as such their moment contribution to the internal work should be reduced from the ultimate flexural moment M_{ui}. The drift angle at the shear failure point θ_S, is assumed as θ_d. As previously explained in the description of strength ratio γ_a in Eq. (13), only a reduction of ultimate flexural moment M_{ui} needs to be considered; therefore, β_S =1.0. As a result, the residual seismic capacity ratio in Eq. (27) can be simplified to Eq. (28) for shear failure dominated structures.

$$R_s = \frac{\sum Q_{ui} H_i \eta_{WSi} + \beta_F \sum M_{ui} \eta_{wFi}}{\sum Q_{ui} H_i + \beta_F \sum M_{ui}} \quad (28)$$

On the other hand, for flexural failure dominated structures (i.e., $M_d < 1$), energy dissipation of shear elements should be reduced by β_S, because deformation capacity of shear elements is smaller than flexural elements. Drift angle at the flexural failure point, θ_F, is used as θ_d. Therefore, β_S is taken as the ratio of the deformation capacity of shear and flexural element (θ_S/θ_F) and β_F is taken as 1.0 (no reduction) such that Eq. (29) is obtained. Although verification of the approximate method for the *R*-index (section 3) with the CSM method (section 2) is not shown in this paper due to space limitations, good applicability and agreement between the two methods has been found from studies reported in [12] and [13].

$$R_F = \frac{\beta_S \sum Q_{ui} H_i \eta_{WSi} + \sum M_{ui} \eta_{wFi}}{\beta_S \sum Q_{ui} H_i + \sum M_{ui}} \quad (29)$$

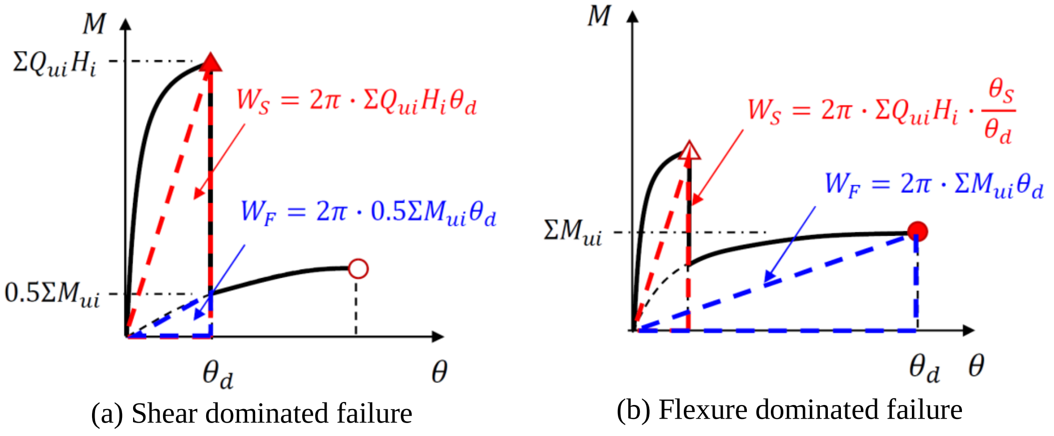


Fig. 12 – Energy dissipation, W, of shear and flexural members.

4. Concluding Remarks

In order to simplify the evaluation of residual seismic capacity of RC frames with mixed failure modes, a residual capacity evaluation method based on the seismic capacity index and a simplified method based on the internal work of the members inside the structure are considered. Within the proposed method a procedure has been developed to judge the dominant failure mode of the structure as either a shear or flexural dominated failure. The reduction of internal energy dissipation explicitly takes into account several structural performance degradation characteristics such as strength, deformation and damping.

Based on the above considerations, the flow of the procedure for determining the approximate residual capacity factor based on internal work is shown in Fig. 13. First, the failure mode and damage class of each structural member are determined and η_{WF} and η_{WS} values are subsequently assigned to each member using Table 1. Next, the dominant collapse mechanism of the structure is determined based on the observed damage level in structural elements. In practical field damage surveys, the dominant collapse mechanism cannot be clearly identified when the damage level is relatively small. In such cases, the dominant collapse mechanism is assumed from analytical results or any other possible information. Finally, the *R* index is evaluated by either Eq. (28) or (29) according to the dominant collapse mechanism.

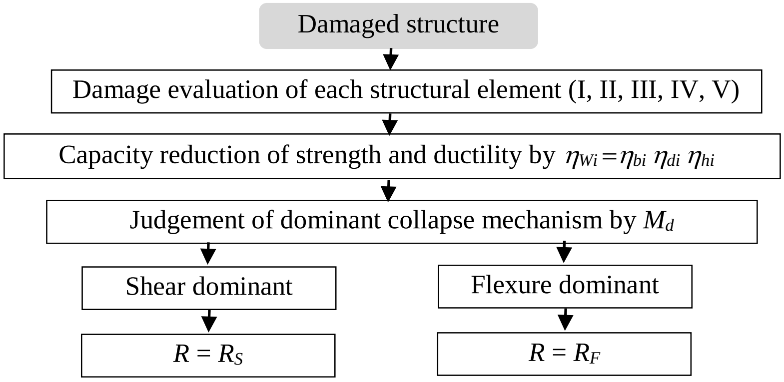


Fig. 13 – General process flow for approximation of residual seismic capacity based on internal work.

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