Evaluation of Seismic Reparability Limit State of R/C Frame Structure Based on the Damage of Columns and Beams

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SUMMARY:
In recent years, not only seismic safety but also seismic reparability is considered as important performance required for reinforced concrete buildings. The final goal of this study is to develop an evaluation method of seismic damage in reinforced concrete structural members which is essential for accurate estimation of reparability limit state of reinforced concrete buildings damaged due to earthquakes.
In this paper, firstly, general concept and outline of damage evaluation procedure of ductile reinforced concrete members are presented. Analytical models are proposed that evaluate residual crack length, crack width and the area of spalling concrete in ductile column and beam. These models are intended to be applied to push-over analysis of frame structure in practical seismic design. For example, the analytical model of crack lengths consists of two parts; models inside and outside the plastic hinge regions which correspond to flexural spring and shear spring, respectively, for the frame analysis under seismic loads. These models are used for calculating total crack length from the predicted number and average length of flexural and shear cracks.
Secondly, evaluation results with the proposed method are shown. Reparability limit states, which are governed by repair cost corresponding to the damage of the structure, are evaluated for reinforced concrete frame structures with different collapse mechanism. As a result, repair costs and economic loss of damaged structure were strongly affected by the type of collapse mechanism. It was found that reparability performance of reinforced concrete buildings with total collapse mechanism is inferior to that of buildings with story collapse mechanism.

Keywords: Damage evaluation, Repair cost, Seismic reparability, Economic loss

1. INTRODUCTION

“Seismic safety” is considered as the most important performance required for reinforced concrete (R/C) structures in terms of protecting human life. In recent years, it is reported that many buildings, which did not collapse but got fatal damages by the severe earthquakes, can’t help being re-constructed. It is also reported that many companies suffered huge economic loss by their business stop due to the non-collapse damage of their buildings. It is recognized that not only “seismic safety” but also “seismic reparability” is necessary to every R/C building. “Seismic reparability” means the seismic performance for buildings that their owners can repair damages within an acceptable repair cost and/or economic loss. However, there are few studies that tried to establish quantitative evaluation methods of seismic reparability performance of R/C structure based on the amount of damages by severe earthquakes and on the repair costs for recovery. The purpose of this study is to develop the method of evaluating seismic damages and reparability performance. Seismic damages are evaluated with cracks or spalling concrete, which appear to R/C frame structure, and reparability performance is estimated based on repair cost and economic loss due to the damage of the building.
In this paper, the outline of damage evaluation procedure of ductile R/C members is described and proposed evaluation method is applied to reinforced concrete frame structures with different collapse mechanism.

2. PROCEDURE FOR EVALUATING REPARABILITY PERFORMANCE OF R/C FRAME

The proposed "damage evaluation models" for R/C beams and columns, which are applied to ductile R/C members, enable the prediction of the amount of damages occurred to R/C members. They include "crack length evaluation models", "crack width evaluation model" and "spalling concrete evaluation model". They are intended to apply to push-over analysis which is typically used for structural design of R/C buildings. In the push-over analysis, every member of the frame is regarded as spring model, which has two flexural springs and one shear spring. The proposed damage evaluation models relate the deformation of these springs to the amount of damages, such as residual crack length or crack width and so on. Therefore, the amount of damages which appear to each member can be estimated analytically. Total damages of the frame can be summed up, and the repair cost and/or the recovery period can be also estimated in the design stage, which are useable for assessing the seismic reparability performance, restoration possibility and business continuity of/in buildings after severe earthquake. Figure 2.1. shows the procedure for damage evaluation of R/C frame structure.

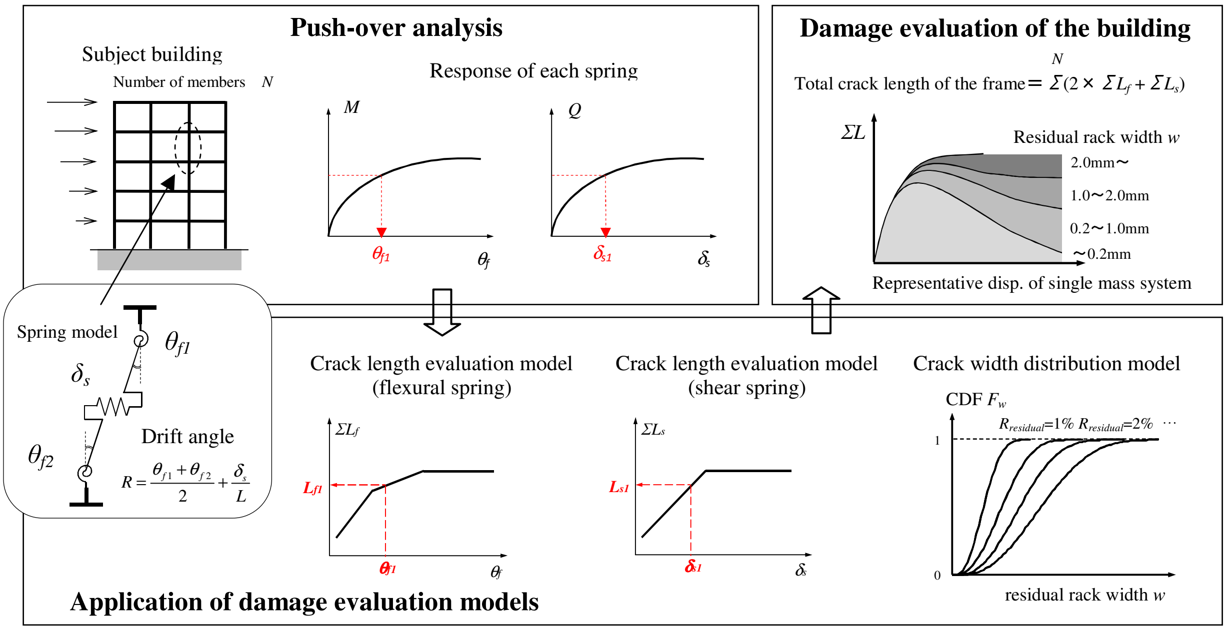


Figure 2.1. Procedure for damage evaluation of R/C frame structure

In order to express the easy repair possibility of a building, "R_r-index" is introduced, which is the ratio of repair cost to the re-construction cost of the buildings. The R_r-index, which indicates the burden of repair cost due to severe damages, can be used for deciding reparability limit state of a building. Here is an expression of R_r-index to Eqn. 2.1..

$$R_r = \frac{C_r + L_r}{C_n + L_n} \tag{2.1}$$

where, R_r: the ratio of repair cost to re-construction cost for the building, C_r : repair cost for the damaged building, C_n: re-construction cost for the damaged building, L_r: economic loss during the period for finishing recovery, L_n: economic loss during the period for re-construction.

If a building tends to suffer serious damages due to severe earthquakes and need large cost and long recovery periods, the value of its R_r-index is large, and it is judged to have insufficient reparability performance.

3. MODEL FOR EVALUATING THE AMOUNT OF DAMAGE IN R/C MEMBERS

The analytical models for estimating the amount of damages which progress in the surface of R/C member are proposed in this section. The modelled damages include crack length, residual crack width and the area of spalling concrete. Crack width distribution model which consist of proposed models are constructed as the beta-distribution based on some loading tests which is conducted by the group of Maeda et al. The area of spalling concrete is modelled based on the strain corresponding to the compressive strength of the concrete. Details of the model for crack length/width and spalling concrete are described in reference by Igarashi and Maeda (2009, 2010).

Figure 3.1. illustrates cracks distribution in R/C beam or column when they are under the cyclic lateral load, where the thick lines show the cracks induced by shear force of clockwise direction. The "hinge region" is the area at two ends of the member that flexural-shear cracks come up in the surface. The "non-hinge region" is the other area, where flexural cracks and shear cracks come up in the surface.

In the damage evaluation models, the progress of crack length in "hinge region" is related to the rotation angle of flexural springs and the progress of crack length in "non-hinge region" is related to the displacement of shear spring.

In brief, the relations are sketched in Figure 3.2. The flexural crack length ΣL_f progress curve matches to the skeleton curve of flexural spring and the shear crack length ΣL_s progress curve matches to the skeleton curve of shear spring. According to some experimental studies conducted by the group of authors, the progress curve of total crack length in one hinge region, ΣL_f , is represented as tri-linear curve, which has a start point $P_s(\theta_{cr}, L_{cr,f})$ and two pass points $P_1(\theta_y, L_{y,f})$ and $P_2(2\theta_y, L_{max,f})$. In the similar way, the progress curve of total crack length in one non-hinge region, ΣL_s , is represented as bilinear curve, which has a start point $P_s(\delta_{scr}, L_{cr,s})$ and pass point P_1 $(\delta_{mu}, L_{ms,s})$. The expressions for calculating $L_{max,f}$ and $L_{max,s}$ are shown in Eqn. 3.1.and Eqn. 3.2.. Each equation is represented with the number of cracks, average length of one crack and effect of cyclic load.

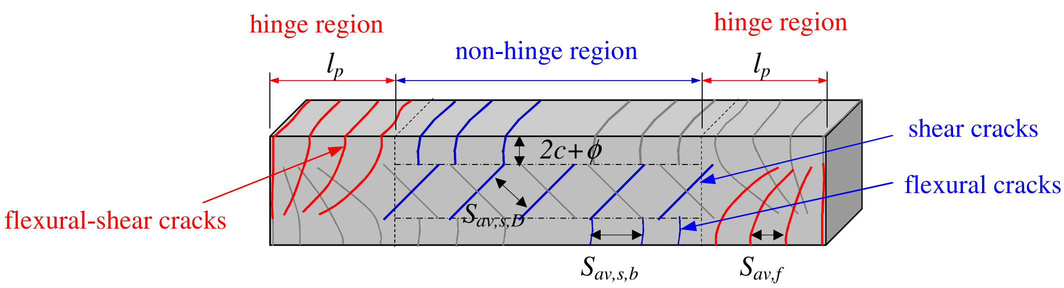
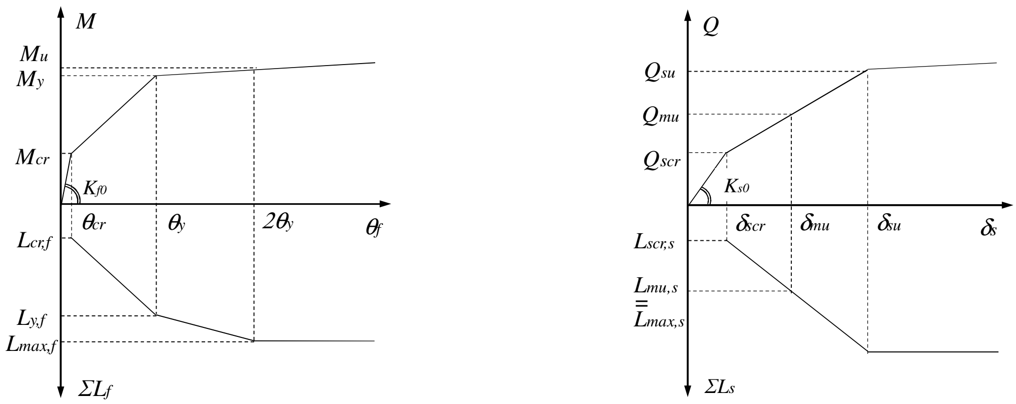


Figure 3.1. Idealization of crack state



θ_{cr}: rotation angle of flexural spring at the time the first flexural crack appears, θ_y: rotation angle of the flexural spring at the time the member yields, $2\theta_y$: rotation angle of the flexural spring at the time total crack length of hinge region reaches to the maximum (in this case, ductility factor of the member is assumed to be 2.0), M_{cr}: moment of flexural spring at the time the first flexural crack appears, M_y: moment of flexural spring at the time the member yields, M_u: moment of flexural spring at the time the member reaches to the ultimate strength. $L_{cr,f}$: total crack length of the hinge region at the first flexural crack appears. $L_{y,f}$: total crack length of hinge region at the time the member yields, $L_{max,f}$: maximum total crack length of hinge region at the time the member reaches to ultimate strength.

δ_{scr}: displacement of the shear spring at the time the first shear crack appears, δ_{mu}: displacement of the shear spring at the time the member reaches to flexural yielding, δ_u: displacement of the shear spring at the time the member reaches to the maximum shear strength, Q_{scr}: shear strength of the shear spring at the time the first shear crack appears, Q_{mu}: shear strength of the shear spring at the time the member reaches to flexural yielding, Q_{su}: shear strength of the shear spring at the time the member reaches to shear ultimate strength. $L_{scr,s}$: total crack length of non-hinge region at the time the first shear crack appears. $L_{mu,s}$: total crack length of non-hinge region at the time the member reaches to flexural yielding, $L_{max,s}$: maximum total crack length of non-hinge region, which is the same as $L_{mu,s}$.

Figure 3.2. Crack progress curves and skeleton curve of springs

$$L_{\max,f}=\frac{l_p}{S_{av,f}}\{2\alpha(D-x_n)+\beta b\}\times 2 \tag{3.1}$$

$$L_{\max,s}=\left[\frac{l_{cr}-l_p}{S_{av,s,b}}\{2(2c+\phi)+b\}\times 2+\frac{\{D-2(2c+\phi)\}\cos\theta+(L-l_p)\sin\theta}{S_{av,s,D}}\cdot\frac{D-2(2c+\phi)}{\sin\theta}\times 2\right]\times 2 \tag{3.2}$$

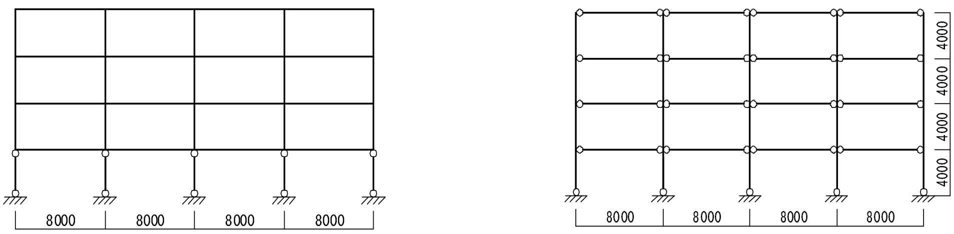
where, D: depth of the member, b: width of the member, $S_{av,f}$: averaged interval of flexural cracks in hinge region, α and β: progress coefficient of crack length, α is 1.4 and β is 1.2, l_p: length of hinge region of the member, x_n: neutral position, l_{cr}: length of the member that flexural cracks come off, $S_{av,s,b}$: averaged interval of flexural cracks in non-hinge region. $S_{av,a,D}$: averaged interval of shear cracks in non-hinge region. c: the distance from the surface of concrete to the surface of main bar, ϕ: diameter of main bar, θ: angle between a shear crack and an axis of member, L: length of the member.

4. PUSH-OVER ANALYSIS AND DAMAGE EVALUATION OF R/C STRUCTURES

This chapter describes damages and seismic performance evaluation of R/C structures, where subject structures have different type of collapse mechanisms; story collapse mechanism or total collapse mechanism.

4.1. Overview of subject structures and push-over analysis

Figure 4.1. shows the R/C frame model which has 4 spans and 4 stories. The mass of each floor is 307.2 ton, which is intended 1.2 ton/m^2 of unit mass. The analysis is conducted for 3 frames (S03, S06 and T03), which are different from the type of collapse mechanism (story collapse mechanism for S03, S06 or total collapse mechanism for T03) and base shear coefficients C_B at the collapse mechanism (C_B=0.3 for S03, T03 and 0.6 for S06). The circle points in Figure 4.1. show the locations of yielding of the members. Figure 4.2. shows the cross section of columns and beams of T03 frame and Table 4.1. shows details of the members; bar arrangement and material strength. As for S03 and S06, the strength of the members and cross sections of members are set so as that the seismic performance of S03 is almost the same with that of T03 at serviceability limit state, and that the seismic performance of S06 is almost the same with that of T03 at safety limit state. In this analysis, the serviceability limit state of frames is decided as the state when one member reaches yielding, and the safety limit state of frames is decided as the state when one story displacement reaches to 1/50rad.



a) Story collapsed frame (S03, S06) b) Total collapsed frame (T03)

Figure 4.1. Span width and story height for frames (unit; mm)

Table 4.1. member parameter for frames

T03		
Column	Cross section	800x800
	Main bar	16-D38(SD490)
	Hoop	4-D16@60(SD590)
	F_c (N/mm^2)	24
Beam	Cross section	800x600
	Main bar	16-D38(SD490)
	Hoop	4-D16@60(SD590)
	F_c (N/mm^2)	24

Column cross section(T03) Beam cross section(T03)

Figure 4.2. Cross section of Column and Beam

Push-over analysis is conducted for 3 frames which are explained above. In the analysis, it is assumed that external force follows A_i distribution and floor stiffnesses are sufficiently large. Each member of the frame is expressed by spring model, which has two flexural springs and one shear spring. The skeleton curves of each spring are set based on the size of cross section, bar arrangement and material strength of the member.
The skeleton curves of the flexural springs in T03 are set, which are represented as tri-linear curves as shown in Figure 3.2.. The skeleton curve of S06's column is set as the same with that of T03's.
S03's column is set, where yield strength of S03 of 1st story is set in proportion to the ratio of S03's and S06's base shear coefficient. S06 and S03's skeleton curves of beams are set so that the displacements at the time of cracking and yielding fit with those T03's, and yield strength is sufficiently large. The stiffness reduction rates after cracking or after yielding are assumed to 0.300 and 0.001 for flexural springs, to 0.500 and 0.001 for the shear spring.

4.2. Seismic performance of frames obtained by Push-over analysis

In the push-over analysis, S03 and S06 frame reached to story collapse mechanism by the yielding of top and bottom of 1st story columns. T03 frame reached to total collapse mechanism by the yielding of ends of all beams. Figure 4.3. shows story shear force coefficient-displacement relationship of S06 and T03 as an example. It is confirmed that the displacement of S06 frame is concentrated on 1st story, and base shear coefficient is reached about 0.6 and that displacement of T03 frame is dispersed in all floors and base shear coefficient is reached about 0.3.
For confirming the seismic performance of the frames, "seismic capacity index" in "Guidelines for Performance Evaluation of Earthquake Resistant Reinforced Concrete Buildings (Draft)" of AIJ (2004) is introduced. This index, which represents seismic performance of R/C structure as definite value, is the ratio of intensity of ground motion that arise the limit states for the structure to the intensity of standardized ground motion. The limit states are considered about serviceability limit state and safety limit state which is defined in 4.1. The seismic indices of these frames, evaluated based on the guidelines, are 0.25, 0.48, 0.22 at reparability limit state, and 0.59, 0.74, 0.86 at safety limit state. The seismic performance of S03 at the serviceability limit state frame can be considered nearly the same with T03 and the seismic performance of S06 at the safety limit state can be considered nearly the same with T03.

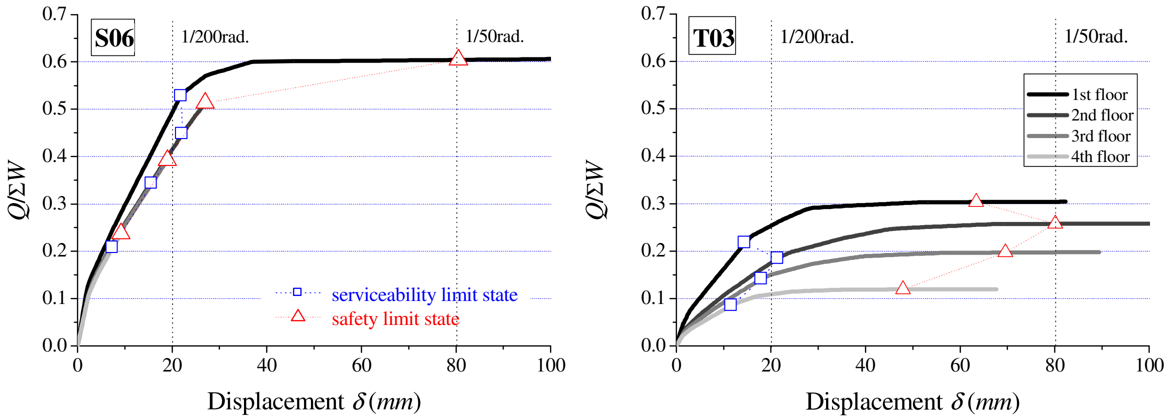


Figure 4.3. Story shear force coefficient-displacement relationship

4.3. Amount of damage and repair cost analysis

4.3.1. Total crack length and spalling area

Figure 4.4. shows the relationship between total crack length and representative displacement of the frame. The total crack length is calculated by the deformation of springs in push-over analysis and the representative displacement is calculated by regarding the frame model as single mass system. It is found that, for the same representative displacement, T03 frame with total collapse mechanism has longer crack length than S03 or S06, story collapsed frame, except the range of very small

displacement. This is because the total collapsed frame has much more damage area than the story collapsed frame. On the other hand, S03 and S06 with story collapse mechanism have severe cracks, whose width is over 5 mm, instead of a lot of slight cracks.

4.4.2. Repair cost of structure

Figure 4.5. shows the relationship between repair cost of frame and representative displacement. The damage repair cost of the member with each damage class is calculated based on the unit costs of each kind of damages in Table 4.5.. The values of the table are decided by the study on repair cost for R/C school building damaged by earthquake (Maeda et al. 2002). Restoration of the member is assumed to be started when over 0.2 mm width of cracks appears in the surface of members because it is not general to repair cracks under 0.2 mm width. The member of the frame, which is regarded as IV or V of damage class, requires additional costs for large-scale repair construction such as an exchange of main bar and/or hoop, temporary constructions, formwork and so on. As a result, the repair cost of the frame T03 with total collapse mechanism is twice of that of the frame with story collapse mechanism.

Table 4.5. Repair method and repair cost by damage class

Damages		Repair method	Unit cost of repair cost (Japanese Yen)
1)	Cracks (width of under 0.2mm)	Rub epoxy resin	¥2,000
2)	Cracks (width of 0.2-1.0mm)	Inject epoxy resin	¥7,300
3)	Cracks (width of 1.0-3.0mm)	Inject epoxy resin	¥9,000
4)	Cracks (width of 3.0-5.0mm)	Inject epoxy resin	¥13,000
5)	Cracks (width of upper 5.0mm)	Inject epoxy resin	¥15,000
6)	Concrete spalling	Restore concrete	¥67,000
7)	Damage classed as IV	Change hoop and so on	¥677,818
8)	Damage classed as V	Re-construct a member	¥246,000

Table 4.6. Criterion of damage class and repair method assumed in analysis

Damage class	Criterion for deciding Damage class	Repair method of member (Table 4.5)
I (Slight)	cracks with under 0.2mm width appear	1)
II (Minor)	cracks with upper 0.2mm width appear	1), 2)
III (Moderate)	cracks upper 1.0mm width appear	1), 2), 3)
IV (Severe)	cracks upper 2.0mm width or concrete spalling appear	1), 2), 3), 4), 5), 6), 7)
V (Collapse)	reach end strain of core concrete(ε=0.003)	1), 2), 3), 4), 5), 6), 8)

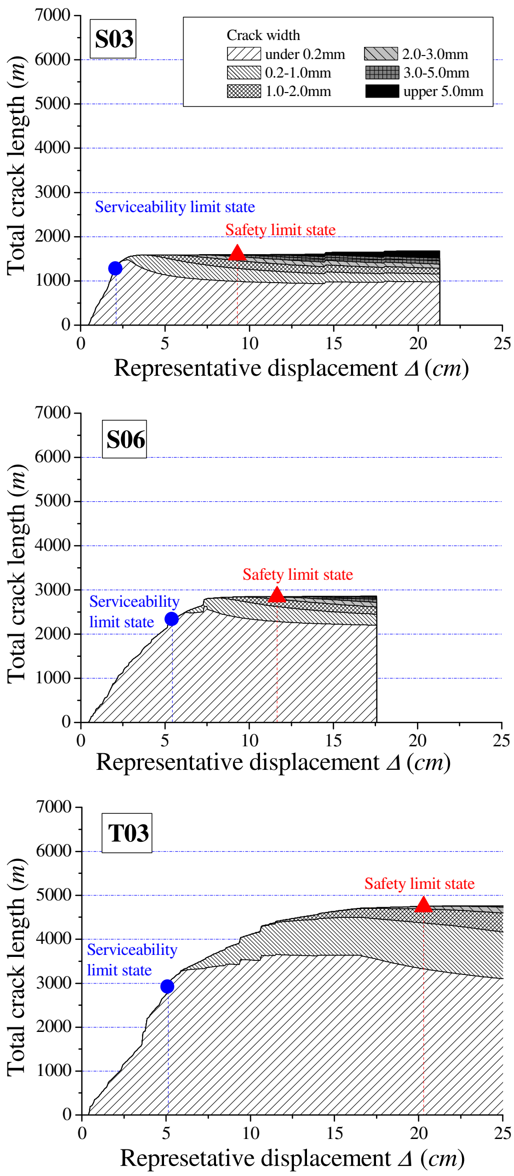


Figure 4.4. Relationship between representative displacement and total crack length of the frame

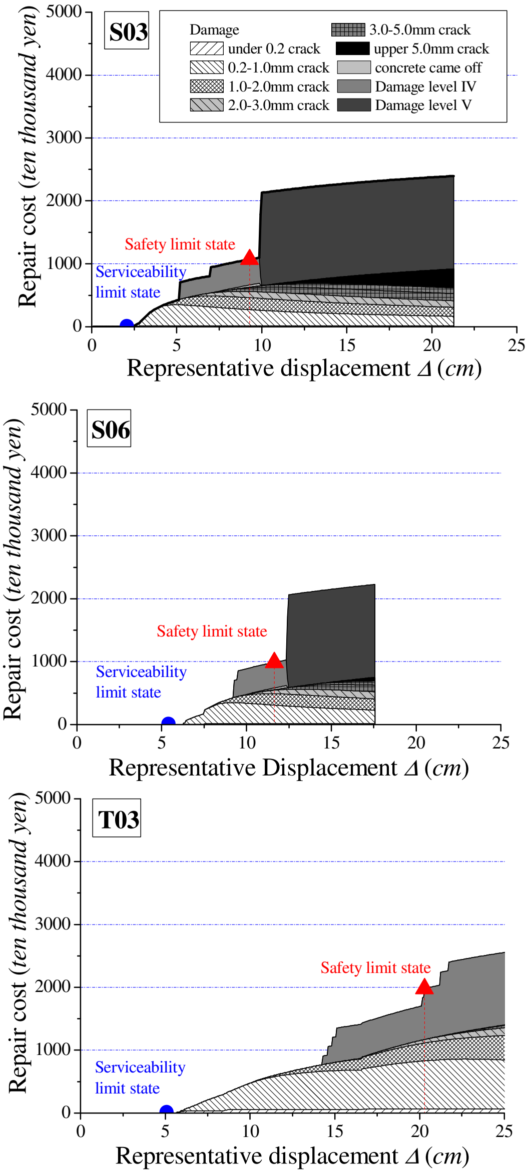


Figure 4.5. Relationship between representative displacement and total repair cost of the frame

4.4. Assumptions in seismic performance evaluation of a building

In order to assess seismic reparability performance of frames, repair cost and economic loss should be calculated. So, "apartment building" and "office building" is selected as the building use for these frames. Under these conditions, the index of seismic reparability performance, R_r-index, are evaluated for 3frames and compared with each other. Firstly, the assumptions which are necessary to calculate R_r-index are mentioned in the following.

4.4.1. Repair cost C_r

The repair cost which calculated by the method proposed in chapter 4.3.2 represents the cost for repairing structural members such as columns and beams. Thus, it is necessary that the structural repair cost is converted into total repair cost of the building including all components of a building such as utilities or finishing. In this paper, the total repair cost of the building is calculated by using "property rate" which means the property assets amount ratio of each component to whole building. The property rate of apartment building and office building are shown in Figure 4.6. The repair cost of whole building is assumed to represent by property rate and repair cost of structure in Eqn. 4.1..

total repair cost of building=repair cost of structure/ property rate of structure to whole building (4.1)

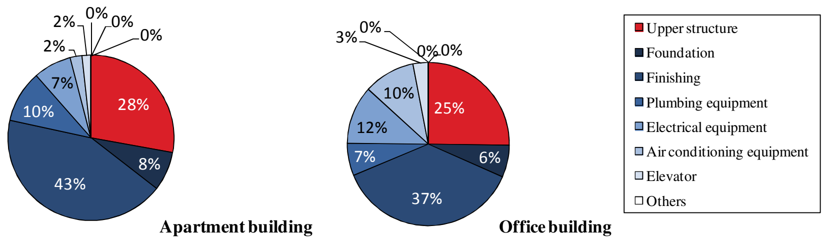


Figure 4.6 Property rate of the newly-build building

4.4.2. Re-construct cost C_n

The cost for re-constructing new building is decided based on the unit cost for newly-build which is generally used for seismic risk management. The unit cost of apartment building is 300,000 yen/m^2 and that of office building is 240,000 yen/m^2. Consequently, it is assumed that the re-construction cost of subject frame is 384,000,000 yen in the case of apartment building use and 307,200,000 yen in the case of office building use.

4.4.3. Period for re-construction

In order to calculate economic loss due to recovery time of damaged building, re-construction period is considered. According to the study by Kato et al. (2004), the period for constructing a new building is calculated by the Eqn. 4.2. and Eqn. 4.3..

The period for new construction of apartment building ;

$$Z = 39.764X^{0.2160}(Y_1+2)^{0.0684}(Y_2+1)^{0.1392} \tag{4.2}$$

The period for new construction of office building ;

$$Z = 77.035X^{0.1501}(Y_1+2)^{0.1365}(Y_2+1)^{0.0919} \tag{4.3}$$

where, Z: construction period (days), X: total area of floor (m^2), Y_1: number of underground floors, Y_2: number of above-ground floors.

In this study, the re-construction period for assuming economic loss are considered not only the construction period calculated by equation above but also the building design period and the application period. Consequently, the period for re-constructing apartment building is found to be 335 days and the period for re-constructing new office building is found to be 378 days.

4.4.4. Period of recovery of the building

Recovery period of the buildings are calculated by reducing the re-construction period showed in chapter 4.4.3. The ratio of recovery period to re-construction period is expressed as the parameter "w" which is represented by Eqn. 4.4 . Here, damage class is represented as "i" and it is assumed that the ratio of recovery period of the members with each damage class to re-construction period is expressed as "D_i". The parameter "w" is evaluated with "D_i" and the rate of members which is categorised each damage class "r_i". It is difficult to precisely decide the effect coefficient D_i, whereas, it is assumed as listed in Table 4.7.. Table 4.8. shows the recovery period when the building reached safety limit state. As a result, the recovery period of total collapse mechanism frame is longer than that of story collapse frame, because total collapsed frame has more damage areas than story collapsed frame.

$$w = \sum (D_i \times r_i) \tag{4.4}$$

$$\textit{recovery period} = w \times \textit{re-construction period} \tag{4.5}$$

Table 4.7. Ratio of recovery period of members with each damage class to re-construction period

	Damage class of member					
D_i	O(None)	I(Slight)	II(Minor)	III(Moderate)	IV(Severe)	V(Collapse)
	0.0	0.0	0.1	0.2	0.8	1.0

Table 4.8. Recovery period of subject buildings at the safety limit state

	Total number of members		O D_i=0.0	I D_i=0.0	II D_i=0.1	III D_i=0.2	IV D_i=0.8	V D_i=1.0	w	Recovery period: apartment building	Recovery period: office building
S03	36	number of members	8	23	0	0	5	0	0.11	37	42
		Rate	**22.2**	**63.9**	**0.0**	**13.9**	**0.0**	**0.0**			
S06		number of members	4	27	0	0	5	0	0.11	37	42
		Rate	**11.1**	**75.0**	**0.0**	**0.0**	**13.9**	**0.0**			
T03		number of members	0	20	4	0	12	0	0.28	93	105
		Rate	**0.0**	**55.6**	**11.1**	**0.0**	**33.3**	**0.0**			

4.4.4. Economic loss due to re-construction L_n and economic loss due to recovery L_r

Economic loss of subject buildings is set based on rental fee or on average annual profits. Apartment building is considered as rental use, and the rental fee is assumed to 3,500 yen/m^2/month. The average annual profit of office building is considered as 15,000,000 yen. Thus, economic loss of these buildings is calculated from the recovery period and rental fee or annual profit.

4.5. Seismic reparability performance of subject buildings

Using these assumptions, Index-R_r of subject buildings is calculated and the relationship between the ratio of input seismic motion to basic seismic motion and Index-R_r are compared in Figure 4.9. As for the apartment building, index-R_r of T03 with the type of total collapse mechanism is larger than that of S03 and S06 frame at safety limit state. Especially, althougth seismic performance of T03 are nearly the same with S06, Index-R_r of T03 frame shows the twice with that of S06's. The same trends can be seen for apartment building and office building. Thus, even if the intensity of input seismic motion is nearly the same, the reparability performance differs largely because of differences of collapse mechanism. The reparability performance of total collapse mechanism, which is recommended when structural design is conducted, can be lower evaluated than that of story collapse mechanism.

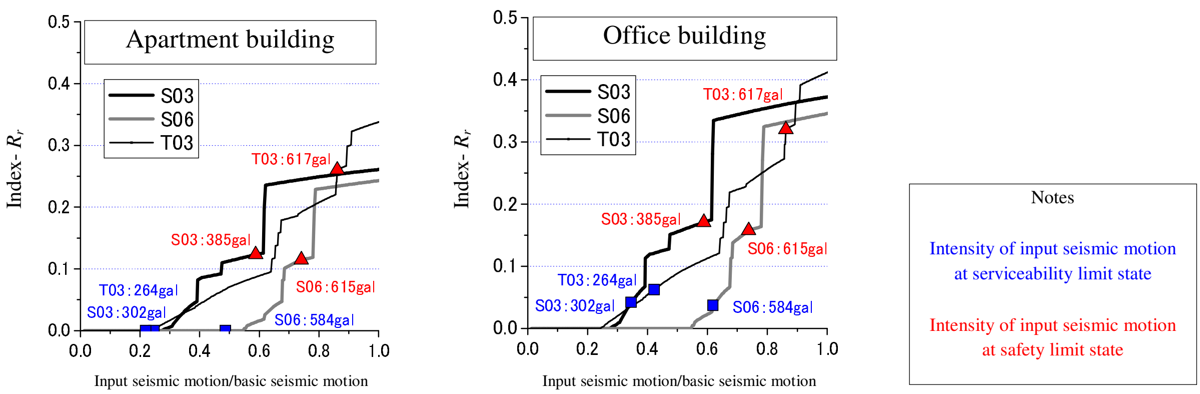


Figure 4.9. Property rate by the difference of using purpose

5.DISCUSSION OF SEISMIC REPARABILITY LIMIT STATE OF THE WHOLE BUILDING

In this study, the reparability performance of the building is decided from the repair cost and economic loss of the building. Another method of determining seismic limit state of R/C structure, which is described in "Guidelines for Performance Evaluation of Earthquake Resistant Reinforced Concrete Buildings (Draft) "(AIJ, 2004), is introduced. According to the method, the limit state of the structure is decided from the story limit state, which is determined from the rate of each damage class of the member. Safety limit state, which evaluates the dangerousness of structural collapse, can be decided from the safety limit state of one story which has the largest damage. On the other hand, it is rational that the reparability limit state is decided from the amount of damage and/or repair cost of whole building. Thus, in the paper, the reparability limit state is evaluated considering damage class of all the members and is decided from the rate of the member which is categorized to each damage class.

In this paper, the criteria of seismic reparability limit state I is defined as the point when no member reached to damage class III and the criteria of seismic reparability limit state II is defined as the point when no member reached to damage class IV.

According to these criterions, the relationships between index-R_r and the ratio of members which is categorised each damage class are shown in Figure 5.1.. The lines of each safety limit state determined by push-over analysis are also shown. Figure 5.1. shows the result of apartment building. The index-R_r of S03 and S06, which are story collapse buildings, show about 3 % when they reached to reparability limit state I and show about 5 % when they reached to reparability limit state II. On the other hand, the index-R_r of T03, which is total collapse building, shows about 4.5 % when it reached to reparability limit state I and show about 9 % when it reached to reparability limit state II. It can be said that reparability performance of building is strongly affected by the type of collapse mechanism. Story collapsed building has lower index-R_r than total collapsed building, because it has fewer damage areas are , is evaluated lower of index -R_r than that of total collapsed building.

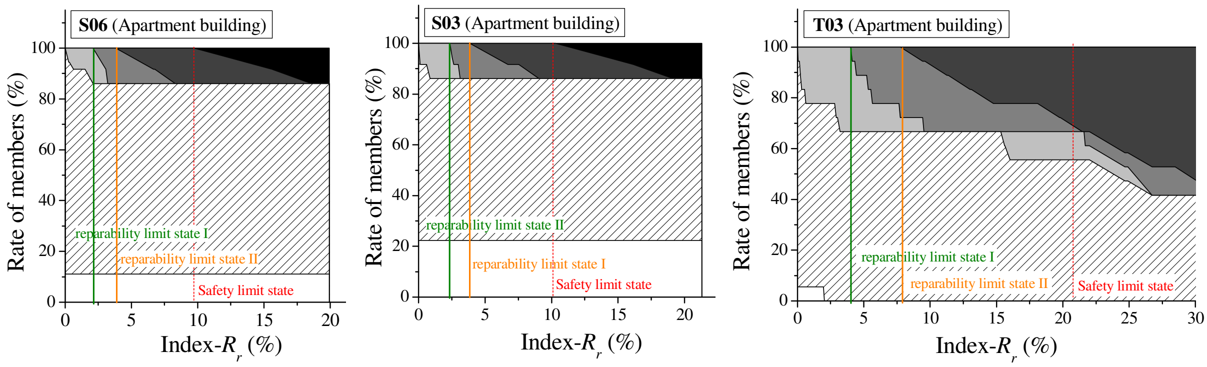


Figure 5.1. Rate of members with each damage class - Index-R_r relationships

6. CONCLUSION

Damage evaluation method for R/C frame structure is proposed and applied to the some frames. It was found that the seismic reparability performance calculated from repair costs and economic loss of damaged structure are strongly affected by the type of collapse mechanism. Reparability performance of reinforced concrete buildings with total collapse mechanism, which is recommended as the point of structural safety, is found to be inferior to that of buildings with story collapse mechanism in this study. Although some assumptions used in this study are need to be examined in detail in future, the evaluation based on damages of the building can be useful by consider the type of collapse mechanism of the building.

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