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RESIDUAL AXIAL LOAD CARRYING CAPACITY OF SHEAR DAMAGED RC COLUMNS

Y. Yang[1], K. Matsukawa[2], H. Choi[3], and Y. Nakano[4]

[1] *Project researcher, Institute of Industrial Science, The University of Tokyo, yangyong@iis.u-tokyo.ac.jp*
[2] *Research associate, Institute of Industrial Science, The University of Tokyo, mtkw@iis.u-tokyo.ac.jp*
[3] *Research associate, Institute of Industrial Science, The University of Tokyo, choiho@iis.u-tokyo.ac.jp*
[4] *Professor, Institute of Industrial Science, The University of Tokyo, iisnak@iis.u-tokyo.ac.jp*

Abstract

In this paper, an arch resistance model, which can be applied to the evaluation of residual axial load carrying capacity of Reinforced Concrete (RC) columns with shear damage, is proposed based on the theory of mechanics. The proposed arch resistance model can give a better understanding of the loss of axial load carrying capacity of shear-damaged RC columns. The evaluation formula of residual axial load carrying capacity of RC column, which consists of contributions by longitudinal reinforcement and core concrete, is derived based on the proposed arch resistance model. In addition, to verify the accuracy of the formula proposed in this paper, a database of RC columns experimentally damaged in shear is compiled and estimation results obtained by the proposed formula are compared with experimental results. The results obtained by the proposed formula shows a good agreement with the compiled experimental results.

Keywords: RC column; Shear-damaged; Axial collapse; Residual axial load carrying capacity; Arch resistance model

1. Introduction

From post-earthquake reconnaissance surveys carried out in the past, it was observed that reinforced concrete (RC) short columns and RC columns with light or widely spaced transverse reinforcement were vulnerable to shear failure. In some cases, axial load carrying capacity was severely deteriorated due to shear damage which led to global or partial building collapse (Fig. 1). For columns vulnerable to shear failure included in the existing RC buildings, it is therefore necessary to rationally evaluate residual axial load carrying capacity and to give advice for seismic resistance evaluation or seismic retrofit.

Up to date, several evaluation models have been proposed in the past researches to account for loss of axial load carrying capacity for RC columns prone to shear failure. These evaluation models can be divided into three types: (1) theoretical model (Uchida and Uezono 2003[1]; Elwood and Moehle 2005[2]; Ousalem and Kabeyasawa 2006[3]; Tran 2010[4]; Matsukawa 2013[5]); (2) semi-empirical model (Kato et al. 2006[6]; Takaine and Yoshimura 2007[7]); (3) empirical model (JBDPA 2001[8]). By these evaluation models, the residual axial load carrying capacity of shear-damaged RC columns can be estimated to a certain extent. However, it is not easy to obtain an intuitive understanding regarding loss of axial load carrying capacity through these models.

Hence, in this paper, an arch resistance model is presented for shear-damaged RC columns, which can give a better understanding of the loss of axial load carrying capacity. The evaluation formula of residual axial load carrying capacity of column, consisting of contributions by longitudinal reinforcement and core concrete, is also derived based on the proposed arch resistance model. In addition, in order to investigate the accuracy of evaluation formula, the database of shear-damaged RC columns is also compiled from several loading tests related to residual axial load carrying capacity carried out by other researchers.

Fig. 1 Axial collapse (1995 Kobe Earthquake)

2. Development of arch resistance model

The force transmission between core concrete and reinforcement due to bond action of them depends on the nature of cracking in concrete. If the crack pattern in the shear-damaged region consists only of main diagonal cracks, the bond action between core concrete and reinforcement exists and the force transmission between them should not be neglected. So, the contribution of force transmission between core concrete and reinforcement in the residual axial load carrying capacity can be expected. However, in a severely damaged column, as shown in Fig. 2, where the cover concrete totally spalled off with completely crushed core-concrete, no bond action exists and the contribution of force transmission due to bond action in residual axial load carrying capacity cannot be expected. In this research, the arch resistance model is developed to estimate the residual axial load carrying

Fig. 2 Shear-damaged pattern with crushed core-concrete (1995 Kobe Earthquake)

capacity neglecting any force transmission due to bond action between the longitudinal bars and crushed core-concrete.

2.1 Definition of limit state of axial collapse

A schematic model (Fig. 3) of shear-damaged region of RC column is established based on shear damage pattern shown in Fig.2. Although the cover concrete of shear-damaged region is totally removed and the core concrete is completely crushed, under assumption that the core concrete could be confined perfectly by transverse reinforcement, it could be still considered that the crushed core-concrete could transmit axial load from upper to lower part of column. With consideration of restrain due to the upper and lower floor slabs or non-structural walls, it is reasonable to assume that the end rotation of the shear-damaged region is negligible and the internal moments (M) at the end sections are equal to each other. Thus, for the shear-damaged region, the equilibrium equation of moment can be written as Eq. (1) and the internal moment (M) at the end sections can be expressed as Eq. (2).

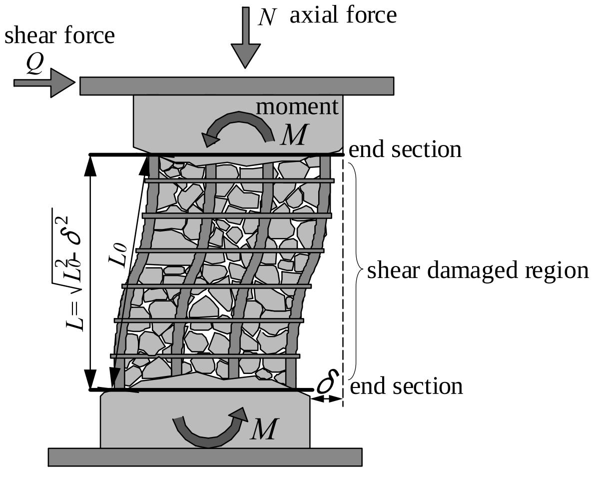


Fig. 3 Schematic model of shear-damaged region with crushed core-concrete

$$2M = N\delta + QL \tag{1}$$

$$M = 0.5N\delta + 0.5QL \tag{2}$$

The limit state of axial collapse of shear-damaged RC column could be defined by analyzing the change in internal moment (M) acting on the end section of shear-damaged region with horizontal displacement (δ) based on Eq. (2). Under a constant axial force (N), the variation of internal moment (M) with horizontal displacement (δ) is shown in Fig. 4. The bending moment ($0.5N\delta$) contributed by axial force behaves linearly with horizontal displacement (δ). However, the bending moment ($0.5QL$) provided by shear force (Q) is non-linear because the yielding occurs at both ends and it decreases along with increase in horizontal displacement (δ). When the shear force (Q) degrades to zero with increase in horizontal displacement (δ), the bending moment (M) acting on the end sections is equal to the maximum moment capacity of column section (as shown by (A) in the Fig. 4). If the horizontal displacement (δ) increases further, the equilibrium between internal and external forces will be lost and the axial collapse would take place. Thus, from the analysis of the change of internal moment (M) with horizontal displacement (δ), it can be concluded that the state of shear force equal to zero is the limit state of equilibration, at which the horizontal displacement (δ) reaches maximum value.

Therefore, in this research, for a shear-damaged RC column subjected to an increasing horizontal displacement with a constant axial force, the state of shear force equal to zero is defined as the limit state of axial collapse. At the defined limit state of axial collapse, the residual axial load carrying capacity can be considered equal to the constant axial force acting on the end section of shear-damaged region.

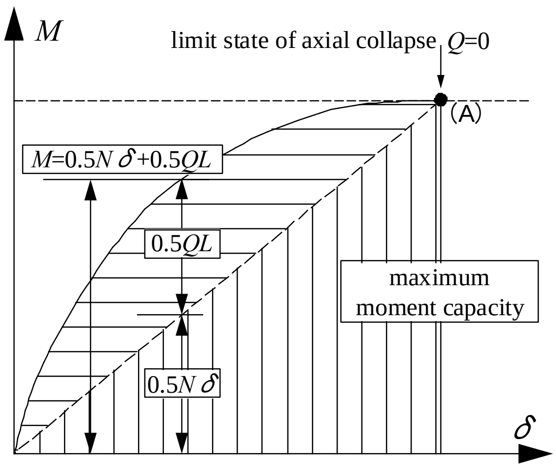


Fig. 4 Relationship between internal moment and horizontal displacement

2.2 Arch resistance model

In order to obtain a better understanding of the loss of axial load carrying capacity of shear-damaged RC columns, the arch resistance model would be proposed based on the schematic model (Fig. 3) of shear-damaged region with the following assumptions.

1). The bond strength between reinforcing bars and concrete can be neglected and no force transmission due to bond action is expected between longitudinal reinforcement and crushed core-concrete.

2). No rotation is expected at the ends of longitudinal steel bars within the shear-damaged region of RC column. These longitudinal steel bars have the same internal forces.

3). The buckling of longitudinal steel bars does not occur prior to axial collapse, with consideration of confinement provided by core concrete and transverse reinforcement.

4). Core concrete confined within shear-damaged region cannot resist any bending moment as it is crushed completely under earthquake motions.

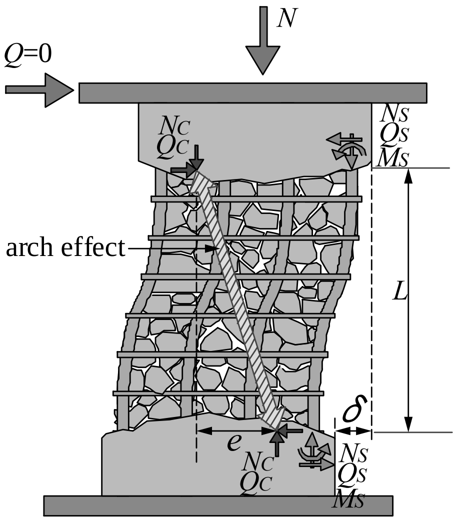
Under the above assumptions, the arch resistance model of shear-damaged region of RC column at the defined limit state of axial collapse is developed as shown in Fig. 5. According to the definition of the limit state of axial collapse at which external force Q is equal to zero, the force equilibrium of shear-damaged region in horizontal direction is shown in Eq. (3). The moment equilibrium of shear-damaged region is shown in Eq. (4a). When the column is at the limit state of axial collapse, the moment equilibrium Eq. (4a) can be rewritten as Eq. (4b).

As shown in Eq. (4a), at the limit state of axial collapse, the P-Δ effect ($nN_S\delta$) of longitudinal steel bars is resisted by the moment ($2nM_S$) acting on the end sections of longitudinal steel bars and the moment ($N_C\,e$) due to the axial forces acting on the end sections of core concrete. It should be noted that the moment ($N_C\,e$) due to axial forces (N_C) of core concrete can be considered as a force couple, which is acting on the core concrete with eccentricity e. In this research, the force couple due to axial forces of core concrete is called ñarch effectò and it can be considered as the interaction between core concrete and longitudinal steel bars.

Moreover, based on the moment equilibrium of shear-damaged region (Eq. (4)), the mechanism of axial collapse of shear-damaged RC column can be explained as follows.

1). When $nN_S\delta < 2nM_S+N_C\,e$, the static equilibrium of shear-damaged RC column can be maintained and the residual axial load carrying capacity of shear-damaged column can be considered greater than N.

2). When $nN_S\delta = 2nM_S+N_C\,e$, it is the limit state of static equilibrium and the shear-damaged RC column reaches the limit state of axial collapse. The residual axial load carrying capacity can be considered equal to N.



(The symbols of internal forces at end sections of longitudinal bars are only indicated for one of them.)

Fig. 5 Arch resistance model

$$nQ_S - Q_C = Q = 0 \tag{3}$$

$$nN_S\,\delta + Q_C L = 2nM_S + nQ_S L + N_C\,e \tag{4a}$$

$$nN_S\,\delta = 2nM_S + N_C\,e \tag{4b}$$

Where, N: axial load (equal to residual axial load carrying capacity); N_S: axial force carried by each longitudinal steel bar; Q_S: shear force carried by each longitudinal steel bar; M_S: bending moment carried by each longitudinal steel bar; N_C: axial force carried by confined core concrete; Q_C: shear force carried by confined core concrete; e: eccentricity of axial forces carried by confined core concrete; δ: horizontal displacement; n: number of longitudinal steel bars.

Thus, based on the arch resistance model propose above, the reason of loss of axial load carrying capacity of shear-damaged RC column can be explained as the decrease in moment carrying capacity ($2nM_S$+$N_C\,e$) of column section with increase in horizontal displacement due to shear damage, which can resist the P-Δ effect of longitudinal steel bars. The decrease of moment carrying capacity ($2nM_S$+$N_C\,e$) of column section can also be divided into two parts: one is the reduction of moment carrying capacity ($2nM_S$) of longitudinal steel bars due to the yielding of end sections, and the other is the reduction of arch effect ($N_C\,e$) of core concrete.

2.3 Residual axial load carrying capacity N_C by core concrete

As shown in Fig. 5, the residual axial load carrying capacity of shear-damaged RC columns (N) can be divided into residual axial load carrying capacity of core concrete (N_C) and longitudinal reinforcing bars (N_S) parts. The evaluation formulas of contributions of core concrete and longitudinal steel bars will be established in section 2.3 and 2.4, respectively.

The evaluation formula of residual axial load carrying capacity of core concrete is deduced based on the static equilibrium of forces in horizontal and vertical directions with the following two assumptions of free body of core concrete shown in the Fig. 6. It should be noted that, there is no forces due to bond action between longitudinal steel bars and core concrete, which are acting on the free body of core concrete.

1). At the oblique and horizontal cutting plane, the relationship of forces parallel and perpendicular to the cutting plane is subject to the Coulombôs law of friction.

2). None of transverse reinforcement within shear-damaged region is ruptured under earthquake motions and it could confine core concrete perfectly. Moreover, at the limit state of axial collapse, all of transverse reinforcement can develop fully plastic strength under tensile action induced by axial loaded crushed core concrete.

The static equilibrium equations of horizontal and vertical direction of forces shown in the free body diagram (Fig. 6) can be given as Eq. (5) and Eq. (6), respectively. By solving Eq. (5) and Eq. (6), the residual axial load carrying capacity of core concrete is determined and the expression is shown in Eq. (7). Moreover, based on static moment equilibrium (Eq. (8)) at point O (Fig. 6), the eccentricity e of the axial load acting on the end sections of confined core could be evaluated by Eq. (9).

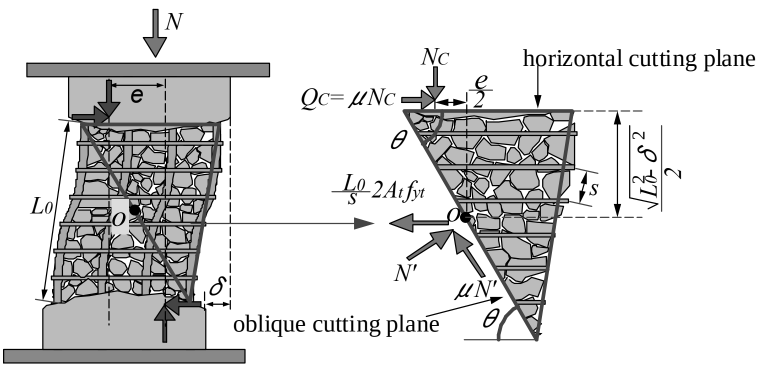


Fig. 6 Free body diagram of core concrete

$$\mu N_C + N' \sin\theta = \mu N' \cos\theta + \frac{L_0}{s} 2 A_t f_{yt} \tag{5}$$

$$N_C = \mu N' \sin\theta + N' \cos\theta \tag{6}$$

$$N_C = \frac{L_0}{s} 2 A_t f_{yt} \frac{\mu \sin\theta + \cos\theta}{\mu^2 \sin\theta + \sin\theta} \tag{7}$$

$$N_C (\frac{e}{2}) = \mu N_C (\frac{\sqrt{L_0^2 - \delta^2}}{2}) \tag{8}$$

$$e = \mu \sqrt{L_0^2 - \delta^2} \tag{9}$$

Where, θ: angle forming by the oblique and horizontal cutting plane; s: spacing of transverse reinforcement; A_t: section area of transverse reinforcement; f_{yt}: yield strength of transverse reinforcement; N': compressive force perpendicular to the oblique cutting plane; μ: friction factor of cutting planes.

2.4 Residual axial load carrying capacity N_S by longitudinal steel bars

As mentioned in the section 2.1, at the limit state of axial collapse, the end section of shear-damaged region can develop moment carrying capacity completely (as shown by (A) in the Fig. 4). So, it is reasonable to assume that the end section of longitudinal bars within shear-damaged region can reach a fully plastic stress condition (Fig. 7) at the limit state of axial collapse. However, for ease in developing the formula of residual axial load carrying capacity of longitudinal bars, the circular cross section is applied by neglecting the effect of steel ribs. The elastoplastic mechanical property is also applied to the end section of longitudinal bars within shear-damaged region neglecting effects of strain hardening.

By stress integration method, the bending moment and axial force acting on the end sections of longitudinal bars can be expressed as Eqs. (10) and (11). From them, the initial interaction relationship between axial force and moment of longitudinal bars can be obtained and shown in Eq. (12) with unknown parameter h. The unknown parameter h (shown in Fig. 7, a distance from centroid of tensile area A to the cross section centreline) can be expressed as Eq. (13) in term of unknown parameter x (shown in Fig. 7, distance between neutral axis and centerline). The axial force can also be obtained by stress integration over compressive part of end section, as shown in Eq. (14). By combining three equations (Eqs. (12), (13) and (14)) to eliminate the unknown parameters x and h , the final interaction relationship between axial force and moment of longitudinal bars can be shown as Eq. (15).

For a shear-damaged RC column with specific horizontal displacement, by solving the equations (Eqs. (4), (7), (9) and (15)), the residual axial load carrying capacity of longitudinal bars can be determined. However, it is difficult to obtain an explicit expression for residual axial load carrying capacity of longitudinal bars in terms of horizontal displacement and it is too complex for practical applications. So, for the actual practice of seismic resistance evaluation or seismic retrofit, the interaction relationship between axial force and moment needs to be simplified to obtain an explicit function for residual axial load carrying capacity of longitudinal bars.

Thus, as shown in Fig. 8, by using the ellipse diagram (Eq. (16)) instead of the above interaction diagram between axial force and moment (Eq. (15)), the explicit expression for residual axial load carrying capacity of longitudinal bars can be obtained and shown in Eq. (17). Although the evaluation formula (Eq. (17)) obtained from approximated method of ellipse diagram is complex, it has a relatively high accuracy compared with other approximated method, such as linear approximation method [2] as shown in Fig. 8.

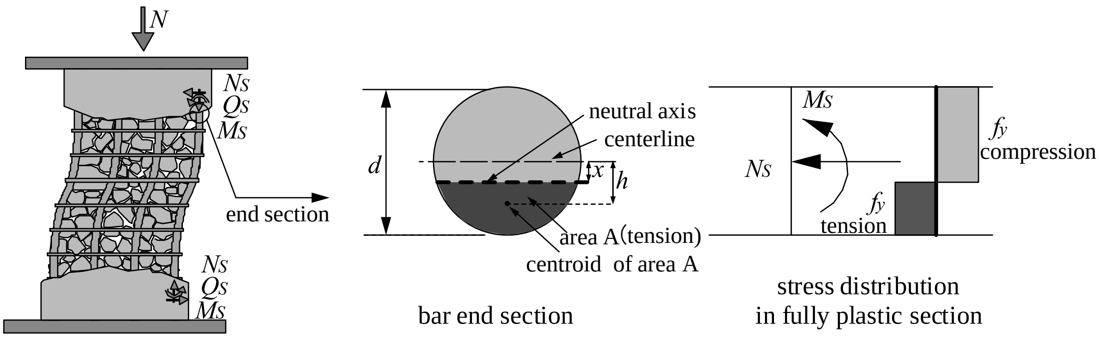


Fig. 7 Ideal plastic section at the ends of longitudinal bars within shear-damaged region

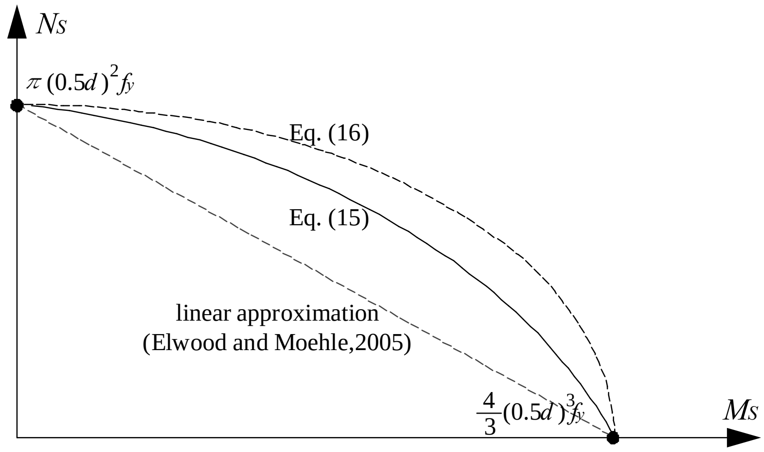


Fig. 8 Axial force and moment interaction relationship

$$M_S = 2Af_y h \tag{10}$$

$$N_S = (\pi\left(\frac{d}{2}\right)^2 - 2A)f_y \tag{11}$$

$$M_S = (\pi\left(\frac{d}{2}\right)^2 f_y - N_S)h \tag{12}$$

$$h = \frac{\frac{2}{3}\left[\left(\frac{d}{2}\right)^2 - x^2\right]^{\frac{2}{3}}}{\frac{1}{2}(\pi\left(\frac{d}{2}\right)^2 - \frac{N_S}{f_y})} \tag{13}$$

$$N_S = 2(2\int_0^{\arcsin(\frac{x}{\frac{d}{2}})} \frac{1}{2}(\frac{d}{2})^2 d\theta + x\sqrt{(\frac{d}{2})^2 - x^2})f_y \tag{14}$$

$$\left(\frac{d}{2}\right)^2 \times \arcsin(\frac{\sqrt{\left(\frac{d}{2}\right)^2 - (\frac{3M_S}{4f_y})^{\frac{2}{3}}}}{\frac{d}{2}}) + \sqrt{\left(\frac{d}{2}\right)^2 - (\frac{3M_S}{4f_y})^{\frac{2}{3}}}(\frac{3M_S}{4f_y})^{\frac{1}{3}} = \frac{N_S}{2f_y} \tag{15}$$

$$\frac{N_S^2}{(\pi\left(\frac{d}{2}\right)^2 f_y)^2} + \frac{M_S^2}{(\frac{4}{3}(\frac{d}{2})^3 f_y)^2} = 1 \tag{16}$$

$$N_S = \frac{\frac{N_C e\delta}{2n(\frac{4}{3}(\frac{d}{2})^3 f_y)^2} + \sqrt{\left[\frac{N_C e\delta}{2n(\frac{4}{3}(\frac{d}{2})^3 f_y)^2}\right]^2 - 4\left[\frac{1}{(\pi\left(\frac{d}{2}\right)^2 f_y)^2} + \frac{\delta^2}{(\frac{8}{3}(\frac{d}{2})^3 f_y)^2}\right]\cdot\left[\frac{N_C^2 e^2}{(\frac{8}{3}(\frac{d}{2})^3 f_y n)^2} - 1\right]}}{2\left[\frac{1}{(\pi\left(\frac{d}{2}\right)^2 f_y)^2} + \frac{\delta^2}{(\frac{8}{3}(\frac{d}{2})^3 f_y)^2}\right]} \tag{17}$$

Where, d: diameter of longitudinal bar; f_y: yield strength of longitudinal bar; A: area of tensile part of end section; h: distance from centroid of area A to the centerline; x: distance between neutral axis and centerline of cross section.

2.5 Residual axial load carrying capacity of shear-damaged RC column

Therefore, the residual axial load carrying capacity of shear damaged RC column can be evaluated as Eq. (18), consisting of core concrete and longitudinal reinforcing bar contributions.

$$N = N_C + nN_S \tag{18}$$

3. Accuracy investigation of arch resistance model

The accuracy of the arch resistance model proposed in this research is verified against the compiled experimental database, shown in table 1. In addition to that, the evaluation accuracy comparison, between arch resistance model and other evaluation models [2], proposed by Elwood and Moehle, is also conducted.

In the estimation of residual axial load carrying capacity of these two evaluation models [2], the basic parameters, such as section dimensions and reinforcement details, can be set based on the compiled database. The other parameters, such as the friction coefficient μ of cutting planes and length L_0 of shear-damaged region in axial direction, are set as follows for two models, respectively.

For the arch resistance model proposed in this research, to simplify the application process, the friction coefficient μ of cutting planes is set as a constant 0.6 according to design guideline [9]. The length L_0 of shear-damaged region in axial direction can be determined based on the critical shear crack angle (the angle between the crack surface and horizontal cutting plane when the column fails in shear) and the depth of column section. In this research, the shear crack angle is assumed to be 60° by referring to the past research [6].

For the evaluation model [2] proposed by Elwood and Moehle, although it is reported that the friction coefficient μ changes with column horizontal displacement, it is also set to a constant value 0.6 to simplify the calculation process in this application. The length L_0 of shear-damaged region is set based on the critical shear crack angle equal to 65° recommended in the original reference [2].

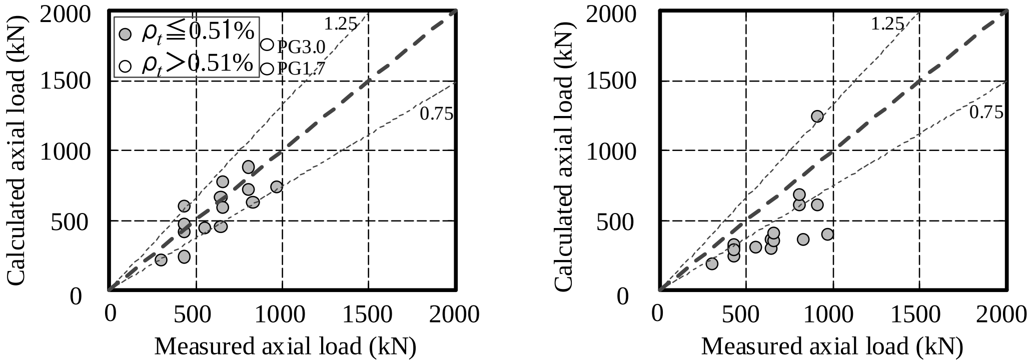
Table 1 Database of shear damaged column specimens

Specimen	D (mm)	b (mm)	h (mm)	d (mm)	n	f_y (MPa)	ρ (%)	d_t (mm)	s (mm)	f_{yt} (MPa)	ρ_t (%)	N_R (kN)	δ (mm)
Ryu Y, Nakamura T, Yoshimura M, 2001 [10]													
N27C	300	300	900	15.9	12	380	2.65	6.35	100	375	0.21	644	27
N18M	300	300	900	15.9	12	380	2.65	6.35	100	375	0.21	429	83
N27M	300	300	900	15.9	12	380	2.65	6.35	100	375	0.21	644	42
Ishigami S, Owa S, Nakamura T, Yoshimura M, 2002 [11]													
2C	300	300	600	15.9	12	396	2.65	6.35	100	392	0.21	431	47
3C	300	300	600	15.9	12	396	2.65	6.35	100	392	0.21	657	32
2M	300	300	600	15.9	12	396	2.65	6.35	100	392	0.21	431	40
3M	300	300	600	15.9	12	396	2.65	6.35	100	392	0.21	657	22
2M13	300	300	600	12.7	12	350	1.69	6.35	100	392	0.21	431	22
Yoshimura M, Takaine Y, Nakamura T, 2004 [12]													
NO.3	300	300	1200	15.9	12	402	2.65	6.35	200	392	0.11	553	23
NO.4	300	300	1200	15.9	12	402	2.65	6.35	100	392	0.21	829	30
NO.5	300	300	1200	15.9	12	402	2.65	6.35	100	392	0.21	967	24
Yamanaka N, Yoshimura M, 2000 [13]													
S1	400	400	900	22.2	16	547	3.87	9.53	180	355	0.20	803	88
S2	400	400	900	22.2	16	547	3.87	9.53	180	355	0.20	803	71
Kato D, Li Z, Nakamura Y, Honda Y, 2006 [14]													
D13W-1	180	180	360	12.7	4	335	1.56	6.35	70	335	0.51	300	12
Nakamura T, Muto S, Ito S, Yoshimura M, 2011 [15]													
PG1.7	450	450	900	19.1	12	390	1.70	9.53	60	390	0.53	911	69
PG3.0	450	450	900	25.4	12	390	3.00	9.53	60	390	0.53	911	127

Note: D: column section depth; b: column section width; h: column interior net height; d: diameter of longitudinal bar; n: number of longitudinal bar; f_y: yield strength of longitudinal bar; ρ: longitudinal reinforcement ratio; d_t: diameter of transverse reinforcement; s: spacing of transverse reinforcement; f_{yt}: yield strength of transverse reinforcement; ρ_t: transverse reinforcement ratio; N_R: residual axial load carrying capacity (at the defined limit state of axial collapse equal to constant axial force); δ: horizontal displacement (convert from the drift angle at which the shear force equal to zero).

For the proposed arch resistance model and the evaluation model proposed by Elwood and Moehle, the comparisons between calculated and measured results of residual axial load carrying capacity are shown in Fig. 9(a) and 9(b), respectively. For most specimens contained in the experimental database, the ratio of calculated-to-experimental residual axial load carrying capacity by the proposed arch resistance model is about 0.75~1.25 as shown in Fig. 9(a). However, the ratio based on the evaluation model (Elwood and Moehle 2005) is less than 0.75 as shown in Fig. 9(b) and the residual axial load carrying capacity is underestimated for most specimens. It is primarily because that, the interaction between core concrete and longitudinal steel bars, which is called ñarch effectò in this research, is not considered in the evaluation model of Elwood and Moehle. Therefore, it shows that, for most specimens included in the experimental database, the residual axial load carrying capacity estimated by the proposed arch resistance model have a better agreement with the measured results than the evaluation model of Elwood and Moehle.

It should also be noted that, for specimens with transverse reinforcement ratio more than 0.51% (PG1.7 and PG3.0) as shown in Fig. 9(a), the proposed arch resistance model leads to overestimation of residual axial load carrying capacity. It is possibly due to the assumption made for transverse reinforcement stress state at limit state of axial collapse in section 2.3, which is not proper for RC column with high transverse reinforcement ratio. That is, in loading test the transverse reinforcement within shear-damaged region does not develop their full plastic capacity, while in the arch resistance model it is assumed that all of transverse reinforcement within shear damaged region can achieve their full plastic capacity. Thus, it is necessary to further discuss the range of transverse reinforcement ratio in which this assumption is valid.



(a) Arch resistance model (b) Elwood and Moehle 2005[2]

Fig. 9 Comparison of calculated-to-measured residual axial load carrying capacity

4. Conclusions

In this research, the arch resistance model is developed based on the theory of mechanics to predict residual axial load carrying capacity of shear-damaged RC column. By the proposed arch resistance model, the mechanism of axial collapse could be explained reasonably and the loss of axial load carrying capacity could also be understood intuitively. The evaluation accuracy of the arch resistance model is investigated through experimental database and it shows that the estimated results of residual axial load carrying capacity have a good agreement with the measured results for most specimens. Moreover, the proposed arch resistance model has higher evaluation accuracy as compared with the other evaluation model because the arch effect of core concrete is considered into the model.

However, the arch resistance model overestimates the results of residual axial load carrying capacity for RC columns with high transverse reinforcement ratios. To obtain a better estimation of residual axial load carrying capacity for RC columns with such a high transverse reinforcement ratio, the stress state assumption of transverse reinforcement needs to be improved.

5. Acknowledgements

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