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SPLITTING FAILURE OF REINFORCED CONCRETE COLUMNS

Thanh-Phuong Pham¹ and Bing Li²

Abstract:

Many earthquake reconnaissance studies have revealed a unique splitting failure mode of RC columns that has yet to be fully elucidated. Most available shear strength models in the current codes are established based on the concept of a diagonal shear plane in which shear force must be transferred through; this concept might not be as applicable to RC columns failed by vertical cracks, or splitting failure under an earthquake action. In this study an alternative load-transferring path is proposed which not only explains the failure mechanism but also allows for the estimation of shear strength of the test specimens. Experimental observations from seventeen RC columns which underwent splitting failure were used to validate the proposed model. To enhance the applicability of the model, it was further corroborated with 89 short columns screened from existing literature. The validations showed that, while simple to apply, the proposed model can give better predictions of shear strength for RC short columns than other shear strength models available in literature.

CE Database subject headings: Reinforced concrete; Short column; Shear strength; Splitting failure; Seismic loading.

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INTRODUCTION

Splitting failure in RC columns under seismic actions is described as the formation of vertical cracks along the column height. The failure has attracted attention of researchers since 1970s when researchers in Japan found that nearly one-third of 260 specimens failed in splitting along longitudinal reinforcement (Higashi *et al.* 1974). An example of this phenomenon is shown in Fig. 1 in which RC columns experienced splitting failure without remarkable signs of flexural and shear cracks. The current experiment conducted at Nanyang Technological University (NTU), Singapore on RC columns under simulated seismic loading also shows a majority proportion of specimens failing by splitting failure mode. Splitting cracks propagating along the main reinforcing bars will prevent the development of diagonal compression struts. As a result the shear strength and the post-elastic response of the columns will be affected especially for the case of short columns, and therefore this phenomenon clearly needs to be investigated. Suenaga and Ishimaru (1974) proposed the “Wedge Theory” for squat plain concrete columns, in which the mechanism of splitting failure was assumed by forming yield lines ABCD as shown in Fig. 2, which were based on the yielding condition of concrete. However, since this model ignored the stress carried by transverse reinforcement and shear stress acting along the failure plane (BC line in Fig. 2), the formation of vertical cracks could not be explained. Studies by Ichinose (1995) and Lura *et al.* (2002) demonstrated that splitting failure is related to bonding failure occurred when the local bond of longitudinal reinforcement exceeded the bond stress capacity. While this might be valid, the studies did not explain the overall performance of the whole column body. Kaminosono (2001) explained the formation of vertical cracks using an idealized deformed shape of the column as shown in Fig. 3. In this study, the author assumed the columns to be in a fully developed tensile hinge zones prior to the formation of vertical cracks. However, observations from earthquake (Fig. 1) and experiments (Tran and Li 2012; Pham and Li 2012) have shown that the vertical splitting cracks developed at very small horizontal displacement and did not require the formation of flexural cracks or a plastic hinge zone.

Although the behavior of reinforced concrete in shear has been studied for years, engineers are still unable to accurately determining the shear strength of RC columns. Most shear strength models available in literature and guidelines (ACI 318 (2008), Sezen and Moehle (2004), ASCE/SEI 41-06 (2007), Biskinis *et al.* (2004), BS EN 1998:3 (2005)) generally refer to diagonal failure and estimate the contribution of transverse reinforcement according to the Ritter-Morsch truss analogy. The shear strength models based on diagonal failure do not reflect the true behavior of columns in splitting failure. This paper aims to clarify the mechanism of splitting failure mode of RC columns through the evaluation of splitting stress along potential vertical planes. Based on experimental observations, a new model for predicting shear strength of RC columns that experienced splitting failure will be developed. Further attempts are then made to extend the model to short columns.

EXPERIMENTAL OBSERVATIONS

An experimental program consisting of nine half-scale RC columns was conducted to study splitting failure of RC columns under simulated earthquake loadings. Columns geometries and materials were selected to be representative of old type columns in existing buildings built before the major change in seismic design guidelines in 1970s. This type of columns was commonly designed with limited confining reinforcement and plain longitudinal reinforcement. Details of test specimens are summarized in Fig. 4 and Table 1.

The test columns were divided into three groups: columns with square sections, columns with rectangular sections and wall-like columns. All columns were designed with the same cross sectional area and longitudinal reinforcement ratio. The clear height of specimens was set to be either 1200 mm or 1700 mm resulting in the aspect ratios (the ratio of the shear span to the depth of the column cross section) to be 1.7, 2.4 for square and rectangular columns or 1.4 for wall-like columns. Longitudinal reinforcements used were smooth bars with a yield stress f_y of 320 MPa. Transverse reinforcement of all test specimens comprised of R6 mild steel bars with 135°-bent, spaced at 125 mm, were characterized by a yield strength f_{yt} of 500 MPa.

The test specimens were attached to a loading apparatus which enabled the columns to be displaced under double curvature bending. Two 1000-kN vertical actuators were used to control the level of axial force as well as to maintain the boundary condition of zero rotation at the top and bottom ends of the specimens. A 1000-kN horizontal actuator was used to produce cyclic shear forces to the top end of specimens. All actuators were pinned at both ends to allow for rotation during the test. The specimen base was fixed to a strong floor by four post-tensioned bolts.

The axial load was slowly applied to the specimens until the designated level was reached. During each test, the column axial load was maintained by manually adjusting the vertical actuators after each loading step. The lateral load was applied cyclically through the horizontal actuator in a quasi-static fashion with two cycles at each amplitude and controlled by drift ratio (the ratio of total horizontal displacement to column height) as 1/500, 1/300, 1/200, 1/100, 1/80, 1/70, 1/60 ...)

The test specimens were extensively installed with measuring devices both internally and externally. Lateral displacement was measured by two horizontal LVDTs (Linear Variable Differential Transformer) mounted parallel to the horizontal actuator. Shear and flexural deformation was recorded based on the reading of a number of LVDT sets located throughout the height of the specimens. Arrangement of strain gauges on both longitudinal and transverse reinforcements at critical locations is shown in Fig. 4.

It was observed that the vertical cracks formed in all specimens at an early stage. All specimens did not reach their theoretical yield strength and flexural capacity, except for SR1. Strain gauge readings also indicated that only in SR1 yielding of longitudinal reinforcement occurred prior to the application of maximum shear force. No yielding of transverse reinforcement was observed until maximum shear force was reached. Shear deformation was relatively small at 10-15% of total deformation before the maximum shear force was reached in the specimens. Response of a typical specimen is described in the following.

Specimen SP1 with an aspect ratio of 1.7 was tested under applied axial force $P/f'_c A_g$ of 0.2. Hysteretic response and the corresponding crack development are shown in Fig. 5. Vertical cracks were observed along the center bars around the mid-area of the column height at a drift ratio of 0.33%. Surprisingly, at this early state, there were only two flexural cracks with the length of one-fourth column section height at the bottom end. The appearance of the vertical cracks was the result of the formation and joining of a series of micro inclined cracks crossing against each other along the length of mid-bars. After reaching the maximum shear force at a drift ratio of 1.67%, the specimen tended to separate into two parts along the vertical cracks. There was a sudden loss of shearing capacity right after maximum horizontal load was attained. After shear failure (maximum shear force reduced by 20%) at drift ratio 2.22% the column was still able to sustain axial force till a large drift ratio of 4%. The test ended with a vertical cleavage throughout the column section that is called splitting failure. Five transverse reinforcement bars at column mid-height fractured at the end of the test as depicted in Fig. 6.

Splitting failure was also observed in the tests of columns with deformed longitudinal reinforcement conducted by Tran and Li (2012) and Pham and Li (2012) using the same test setup and loading procedure as the test program in this study. The crack patterns of typical columns (rectangular specimens tested under axial load of 0.05 and $0.5f'_c A_g$) are shown in Fig. 7. Comparing those specimens with the specimens which did not fail by splitting failure (specimens in the study of Tran, 2010), it was found that columns that did not fail in splitting failure had a higher shear strength but much smaller drift capacities (drift ratio at shear failure and axial failure).

RC columns with vertical cracks or splitting failure in other studies available are tabulated in Table 2. The observations of failure mechanism were similar to those described above. It was found that specimens that failed by splitting failure were short columns with an aspect ratio smaller than 2. It is noted that Specimen SP3 and SP4 (in the current test) with an aspect ratio of 2.43 experienced vertical cracks but did not fail by splitting failure. These two specimens failed by the formation of plastic

hinge at both column-ends causing crushing of concrete and buckling of longitudinal reinforcement. So far there is no splitting failure recorded in RC columns having an aspect ratio higher than 2.

MECHANISM OF SPLITTING FAILURE

From experimental observation, most of the specimens failed with the formation of vertical cracks. At the end of the test, one vertical plane fully developed at the mid-height of the columns separating the column body into two parts. It was observed that the vertical cracks appeared at an early stage (low drift ratio) without yielding of transverse reinforcement (indicated by strain gauges). Therefore the vertical cracks formed could be due to the shear stress acting along the potential splitting crack. Under double curvature moment and axial force as shown in Fig. 8a, the column formed two compression zones at its ends. The applied axial force and shear force will be transferred from the compression zone at the top end through the potential splitting plane to the compression zone at the bottom end. The yield lines were possibly formed as depicted in Fig. 8b. In this regard, the column might fail in one of the following mechanisms:

- Failure at compression zone due to concrete crushing;
- Yielding of transverse reinforcement;
- Splitting failure at potential sliding crack.

From Fig. 8b, the applied horizontal force was transferred by normal stress to vertical cracks along the mid-bar; the applied axial force was transferred from the top-end to the bottom end of the column through friction stress along the surface of the splitting cracks; The bending moment at the ends was modeled by a couple resultant force: the compression part F ($F=M/d$) acting on the center of the compression zone; the tensile part T_s acting on the tensile longitudinal reinforcement.

The resultant forces are shown in Fig. 8c.

The sliding shear force in the compression zone S_l is the total shear stress transferred through the diagonal plane:

$$S_l = \tau_{s1} b \frac{h}{2 \sin \alpha} \quad (1)$$

Shear stress (or sliding shear stress) acting on potential sliding plane is calculated as:

$$S_2 = \tau_{s2} b (L - h \tan \alpha) \quad (2)$$

where τ_{s1} and τ_{s2} are the average shear stresses on the diagonal and vertical plane respectively, b is the column section width, h is column section height, L is column height, α is the angle of the compression zone.

The compression force transferred through compression zone can be calculated:

$$C_c = c_{s1} b \frac{h}{2 \sin \alpha} \quad (3)$$

Neglecting the contribution of concrete in tension, the tensile stress on the vertical plane is carried out by transverse reinforcement. The resultant force T_s is, therefore, calculated as:

$$T_s = \tau_v \frac{L - h}{s} A_v \quad (4)$$

where s , A_v , τ_v is the spacing, area and tensile stress of transverse reinforcement respectively.

Equilibrium in the horizontal direction is given as:

$$V = 2S_l \cos \alpha + T_s - 2C_c \sin \alpha \quad (5)$$

where V is the applied shear force, S_l is the shear force transferred through the compression zone, T_s is the shear force carried by transverse reinforcement (ignore the tensile force contributed by concrete in tensile region), C_l is the compression force acting at compression zone.

Equilibrium in vertical direction can be given as:

$$P = 2S_l \sin \alpha + S_2 + 2C_c \cos \alpha \quad (6)$$

Where P is applied axial force, S_2 is the splitting shear force transferred through splitting cracks.

From Eq.(5) and Eq. (6) we have:

$$V = P + T_s - S_2 + 2S_1(\cos \alpha - \sin \alpha) - 2C_c(\cos \alpha + \sin \alpha) \quad (7)$$

From the experiment observations (presented in previous part) the vertical cracks generally developed at a point forming an angle of 45 to the corner of the column therefore the angle α can be reasonably assumed as 45° , then Eq. (7) can be rewritten as:

$$V = P + T_s - S_2 - 2\sqrt{2}C_c \quad (8)$$

The shear force carried by transverse reinforcement is calculated as:

$$T_s = k_1 \frac{L-h}{s} A_v f_{yt} \quad (9)$$

where L is the column height; s , A_v , f_{yt} is the spacing, area and yield stress of transverse reinforcement respectively; k_1 is a factor expressing the level of tensile stress in transverse reinforcement $k_1 \leq 1$.

Splitting shear force S_2 was taken by the aggregate interlock and dowel action developed in transverse reinforcement across vertical failure plane. However Park and Paulay (1975) suggest that dowel action is not a major component of shear-resisting mechanism across cracked interfaces at acceptable shear displacements. It is, therefore, reasonable to assume when vertical cracks occurred (at low drift ratio as observed) S_2 was mainly resisted by aggregate interlock of concrete and can be written as:

$$S_2 = k_2 f_{ct} A_{sp} \quad (10)$$

where k_2 is a factor expressing the level of tensile stress in concrete along splitting crack $k_2 \leq 1$, A_{sp} is the area of splitting plane, f_{ct} is the concrete shear stress limit at potential vertical plane. Park and Paulay (1975) proposed a simple expression for f_{ct}

$$f_{ct} = 0.166 \sqrt{f'_c} \quad (11)$$

A_{sp} is calculated as:

$$A_{sp} = (L-h)b \quad (12)$$

Eq. (10) is then expressed as:

$$S_2 = 0.166k_2(L-h)b\sqrt{f'_c} \quad (13)$$

Many researchers including Schlaich (Schlaich *et al.* 1987; Schlaich and Schaefer 1991) suggest that the compression stress in compression strut is limited to $0.4f'_c$ then the compression force C_c can be expressed as:

$$C_c = 0.4k_3bh f'_c / \sqrt{2} = 0.283k_3f'_c A_g \quad (14)$$

where k_3 is a factor expressing the level of compression stress in concrete strut $k_3 \leq 1$.

From Eq. (9), (13) and (14), Eq. (8) is then expressed as:

$$V = k_1 \frac{L-h}{s} A_v f_{yt} - 0.166k_2(L-h)b\sqrt{f'_c} + (n-0.8k_3)f'_c A_g \quad (15)$$

Parametric studies with three variables, k_1 , k_2 , k_3 , showed that for columns failed by mode 1 (splitting failure), the best match for V_{max} was achieved when k_2 exceeded 1.

k_1 is found to be a function of transverse reinforcement ratio (ρ_v %) expressing the level of stress on transverse reinforcement at maximum shear force, proportional with the ratio of transverse reinforcement:

$$k_1 = 0.9e^{-0.9\rho_v} \quad (16)$$

k_3 is the function of the applied axial force showing the level of axial stress transferred through compression zone (C_1) depending on the level of applied axial force ($n = P/f'_c A_g$) as shown in Fig. 9:

$$k_3 = 1.17n - 0.1 \quad (17)$$

Equation (15) can then be rewritten as:

$$V = 0.9 \frac{L-h}{s} A_v f_{yt} e^{-0.9\rho_v} - 0.166(L-h)b\sqrt{f'_c} + (0.08 + 0.064n)f'_c A_g \quad (18)$$

Verification of the proposed equation (Eq. (18)) by the test results from 17 RC columns with splitting cracks is shown in Table 2. The proposed equation yields good predicted shear strength with the mean difference between analytical assessments and test results is 5% and the coefficient of variation (COV) is 13.5%. This pointed out that when the specimens reached maximum shear force, the splitting stress

also reached the splitting ultimate strength while the stresses in other force transferring mechanisms (C_c and T_s) were still in elastic range.

To extend the application of this model to RC columns those which possibly failed by splitting failure, equation (15) can be further modified by verifying the parameter k_2 as a function of aspect ratio (a/d) and applied axial force ($n=P/f'_c A_g$). k_2 is obtained based on test data of the above 17 columns:

$$k_2 = n^{0.1} + 0.2 \left(\frac{a}{d} \right)^{-2} \quad (19)$$

Equation (15) can be rewritten as:

$$V = 0.9 \frac{L-h}{s} A_v f_{yt} e^{-0.9 \rho_v} - 0.166 \left[n^{0.1} + 0.2 \left(\frac{a}{d} \right)^{-2} \right] (L-h) b \sqrt{f'_c} + (0.08 + 0.064n) f'_c A_g \quad (20)$$

The verification of the proposed equation (Eq. 20) is demonstrated by comparison with published experimental results with respect to the maximum shear force obtained from the test results of 89 columns available to the authors. These specimens were selected mainly on the basis of the observed failure mode whereas the splitting failure might be occurred, which are: short columns (aspect ratio smaller than 2.5) and tested under double curvature cyclic bending. The selected specimens ranged in size from 80x80x160mm ($b \times h \times L$) to 400x500x1650mm. Concrete compressive strength ranged from 15.1 to 48.0 MPa, whereas the transverse reinforcement ratio ranged from 0.1 to 1.88%. The applied axial force ratio ($P/f'_c A_g$) ranged from 0.0 to 0.9. It is noted that of the 89 specimens, there are 17 specimens observed splitting failure. Details of selected specimens and their shear strength calculated based on the proposed equation are tabulated in **Table 3**.

Shear strength estimations were also calculated based on the methods from ACI 318 (2008), ASCE/SEI 41-06 (2007) and Biskinis *et al.*, (2004) and compared with the proposed model. The shear strength, as defined in ACI 318, based on 45-degree truss model, is given as:

$$V_n = V_c + V_s = \frac{A_v f_{yt} d}{s} + \frac{1}{6} \left(1 + \frac{P}{14 A_g} \right) b d \sqrt{f'_c} \text{ (Mpa)} \quad (21)$$

where V_c is nominal shear strength provided by concrete, V_s is nominal shear strength provided by transverse reinforcement.

Sezen and Moehle (2004) proposed a semi-empirical shear strength model based on a database of 51 test columns. This model is then adapted in ASCE/SEI 41-06 (2007) and given as:

$$V_n = V_c + V_s = k \frac{A_v f_{yt} d}{s} + \lambda k \left(\frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \text{ (Mpa)} \quad (22)$$

where k is taken as 1 for displacement ductility less than 2, as 0.7 for displacement ductility more than 6 and varies linearly for intermediate displacement ductility; M/Vd is the largest ratio of moment to shear times effective depth, and shall not be taken greater than 4 or less than 2; and λ is equal to 1 for normal-weight concrete.

Biskinis *et al.*, (2004) proposed a purely empirical shear strength model for squat columns (columns with aspect ratio less than 2.0). This model is then adapted in Eurocode 8 part 3 (2005) expressed as:

$$V_n = \frac{4}{7} (1 - 0.02 \min(5, \mu_\Delta - 1)) (1 + 1.35n) (1 + 0.45.100\rho_l) \sqrt{\min(f'_c, 40)} b z \sin 2\theta \text{ (MN, m)} \quad (23)$$

where μ_Δ is displacement ductility ratio, θ is the angle between the diagonal and the axis of the column ($\tan \theta = h/2a$), z is the internal arm and taken as $d-d'$, ρ_l is total longitudinal reinforcement ratio.

Fig 10 plots the comparisons of measured shear strength with the calculated shear strength $V_{proposed}$, V_{ACI} , $V_{Sezen\&Moehle}$ and $V_{biskinis}$. The mean ratio of measured to calculated shear strength and its coefficient of variation are 1.04 and 0.28, 1.21 and 0.31, 0.9 and 0.35 and 1.08 and 0.40 for the proposed model and models from ACI 318-08, Sezen & Moehle and Biskinis *et al.*, respectively. It is noted that although the proposed model and ACI model are dependent from displacement ductility, they provide reasonably safe estimates for shear strength of collected columns. The Sezen and

Moehle's shear strength model, originally developed for columns with light reinforcement, is sensitive with displacement ductility demand. Even after considering the ductility ratio from experimental data, the shear strength estimations from this model are inconsistent and room for improvement remains. On the contrary, the purely empirical shear strength model of Biskinis *et al.* relies on little variation the displacement ductility factor. This model ignores a very important parameter affecting the shear strength of RC columns which is transverse reinforcement content. The Biskinis *et al.* model produces a large scatter for shear strength predictions of short columns with aspect ratio equal or less than 2.5 in this study because it was built by fitting parameters with test results of columns with aspect ratio from 1 to 2. Among the four shear strength models, the proposed model gives the best match with the experimental results from the database.

To investigate the validity and applicability of the proposed equation across the range of several key parameters including axial load, aspect ratio, compressive strength of concrete and transverse reinforcement ratio, the ratio of experimental shear strength (V_u to shear strength calculated from the proposed Eq. (20)) against axial load ($P/f'_c A_g$), aspect ratio (a/d), transverse reinforcement index ($\rho_v f_{yt}/f'_c$) is plotted in Fig.11. The good correlation between the experimental and predicted strengths across the range of axial load, aspect ratio, transverse reinforcement index indicates that the proposed model well represented the effects of these key parameters. It can therefore be concluded that the proposed shear strength model is applicable and reliable.

EFFECTS OF COLUMN PARAMETERS

In this section, a parametric study is conducted to improve the understanding of the effects of various parameters on the shear strength of RC columns based on the proposed model. The parameters investigated were the aspect ratio (a/d) and column axial load ratio ($P/f'_c A_g$) concrete compressive strength (f'_c) and transverse reinforcement index ($\rho_v f_{yt}$). Specimen SP1 with an aspect ratio of 1.7 was considered as the reference specimen in the parametric study.

Influence of aspect ratio

Fig. 11 shows the influence of the aspect ratio on shear strength of RC columns. Six aspect ratios of 1.0, 1.2, 1.50, 1.8, 2.10, and 2.43 were investigated. In general, the shear strength decreased with an increase in aspect ratio. It was found that with an increase in aspect ratio from 1.00 to 1.20, 1.50, 1.80, 2.10 and 2.43, the shear strength of columns axial loads of $0.02 f'_c A_g$ dropped by approximately 12.8%, 19.4%, 28.2%, 38.5% and 54.7%, respectively. Similar trends were observed for the columns with other levels of axial load ratio. The shear strength increased by approximately 6.9%, 16.0%, 26.7%, 39.6% and 56.9% for columns under an axial load of $0.60 f'_c A_g$ with an increase in aspect ratio from 1.00 to 1.20, 1.50, 1.80, 2.10 and 2.43, respectively. This confirmed that the aspect ratio significantly influences the shear strength of RC columns.

Influence of axial force

It is generally recognized that the presence of column axial load can effectively increase the shear strength of columns. The analyses as illustrated in Fig. 11 were carried out to assess the influence of axial load ratio on shear strength. The axial load ratio was varied from 0 to 0.60. In general, the shear strength increased with the increase in axial load ratio. It is shown that with an increase in axial load ratio from 0.1 to 0.30, 0.40 and 0.60 the strength for specimens with an aspect ratio of 1.0 rose by approximately 10.3%, 16.8% and 30.5%, respectively. The trends gradually slowed down with the increase in aspect ratio. This is understandable as when the height of the columns is increased (the section is kept unchanged) the columns is more likely to be governed by buckling, then the applied axial force cannot fully affects the shear carrying mechanism. It can thus be concluded that the axial load ratio affects more significantly the shear strength of shorter columns than longer columns.

Influence of transverse reinforcement

The analyses as illustrated in Fig. 11 were conducted to assess the influence of transverse reinforcement on shear strength of short columns. Two axial loads of $0.2 f'_c A_g$ and $0.35 f'_c A_g$ were

considered. The results for these two cases were similar then only the case was demonstrated here. Transverse reinforcement ratio was varied by changing the cross sectional area or spacing of tie bars. In this study, the effect of transverse reinforcement was expressed by the transverse reinforcement index which was taken as multiplying transverse reinforcement ratio (ρ_v) by its yield strength (f_{yt}). It was found that beyond a particular ratio, the shear strength remained almost the same regardless of changing the spacing or cross sectional area of transverse reinforcement. Fig. 11 shows that with an increase in transverse reinforcement index from 0.36 to 0.9, 1.45, 1.99 and 2.53 (MPa), shear strength rose significantly by approximately 95.8%, 151%, 178%, 189%, respectively for columns under an axial load of $0.20 f'_c A_g$. This trend is similar for columns having different concrete compressive strength. The shear strength was almost unchanged when the transverse reinforcement index was increased beyond 3.1 (MPa). This suggested that the effect of transverse reinforcement index on shear strength is significant up to a certain ratio. It is worth noting that the contribution of transverse reinforcement on shear strength in most of current approaches is based on the total shear force carried by transverse reinforcement at diagonal cracking plane. Therefore shear strength is simply proportional to the transverse reinforcement ratio without any limitation. In other words shear strength will keep increasing as long as the transverse reinforcement index is increased. However for high amounts of transverse reinforcement the concrete in the inclined struts may fail before yielding of stirrup. Then the shear strength will not be governed by transverse reinforcement any longer. Therefore, certainly, there is a limit on shear strength of RC columns when its transverse reinforcement index is increased.

Influence of concrete grade

Fig. 11 illustrates the influence of concrete compressive strength on shear strength for a column under applied axial loads of $0.20 f'_c A_g$. The concrete compressive strengths investigated were 20MPa, 25MPa 30MPa, 35MPa, and 40MPa. It was found that a strong dependence of shear strength on compressive strength of concrete when transverse reinforcement index smaller than 1.0. The

parametric study showed that the changes in shear strengths of 34%, 29%, 24% and 21% when the f'_c increased from 20 to 25MPa, 25 to 30MPa, 30 to 35MPa and 35 to 40 MPa respectively when the transverse reinforcement index was 0.36. However, no significant changes on shear strength were observed when the transverse reinforcement index was higher than 1.0. This suggests that the columns may response sensitively with the change of the compressive strength of concrete at low value of transverse reinforcement index.

CONCLUSIONS

Splitting failure of RC columns was resulted from the development of vertical cracks along the mid-height at the central region of columns, and it was found only taken place in columns with an aspect ratio not higher than 2. This phenomenon appeared in both columns with deformed bars and plain bars, and was not directly related to the level of applied axial compression force. Columns that failed by splitting failure reached a lower shear capacity but higher drift capacity compared to those failed by diagonal shear failure.

An alternative load-transfer path was proposed to explain that splitting failure was due to the splitting shear stress exceeding the splitting ultimate strength along vertical potential cracks. A new shear strength model was then formulated based on the proposed failure mechanism. Comparison made between shear strength of analytical results (Eq. (18)) and test data collected on 17 columns observed splitting cracks showed a good agreement with the average difference of 1.05 and coefficient of variation of 0.135. The model also yielded good correlations with observed behavior.

The extended model could be used to estimate the shear strength of short columns. While simple, verification of the proposed shear strength model (Eq. (20) with test data collected from 89 RC short columns showed a better agreement than the models of ACI 318, Sezen & Moehle and Biskinis *et al.* The proposed model suggests that, for short columns, the transverse reinforcement index ($\rho_v f_{yt}$) could be used effectively (for improving shear strength) and economically up to 3.1(MPa).

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Notation

The following symbols are used in this paper:

A_g	=	cross sectional area;
A_v	=	total transverse reinforcement area within spacing;
a	=	the shear span of the specimen ($L/2$)
b	=	width of column section;
d	=	distance from the extreme compression fiber to centroid of tension reinforcement;
E_c	=	modulus of elasticity of concrete;
E	=	cumulative energy dissipation;
f'_c	=	compressive strength of concrete;
f_y	=	compressive strength of longitudinal reinforcement;
f_{yt}	=	compressive strength of transverse reinforcement;
f_w	=	transverse reinforcement ratio;
h	=	the height of column section;
M/V	=	ratio of moment to shear at critical section;
P	=	applied axial load;
s	=	spacing of transverse reinforcement
V_c	=	shear force carried by concrete;
V_u	=	theoretical flexural strength of columns;
V_n	=	nominal shear strength of columns;
$V_{\max}$	=	maximum of applied horizontal force;

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Table 1-Summary of test specimens

Specimen	b (mm)	h (mm)	L (mm)	a/d	A_s (mm ²)	A_v (MPa)	s (mm)	f'_c (MPa)	$\frac{P}{f'_c A_g}$
SP1-1.7-0.2	350	350	1700	1.7	2512	56.52	125	29.8	0.2
SP2-1.7-0.35	350	350	1700	1.7	2512	56.52	125	29.2	0.35
SP3-2.4-0.2	350	350	1200	2.4	2512	56.52	125	30.6	0.2
SP4-2.4-0.35	350	350	1200	2.4	2512	56.52	125	28.7	0.35
SR1-1.7-0.35	250	490	1700	1.7	2512	56.52	125	23.3	0.35
SR2-1.7-0.5	250	490	1700	1.7	2512	56.52	125	22.5	0.5
SW1-1.4-0.2	200	610	1700	1.4	2412	56.52	125	21.7	0.2
SW2-1.4-0.35	200	610	1700	1.4	2412	56.52	125	20.2	0.35
SW3-1.4-0.5	200	610	1700	1.4	2412	56.52	125	20.5	0.5

(Note: E = Cumulative Energy dissipation, 1kN = 0.2248 kip, 1mm = 0.04in, 1 MPa = 0.145 ksi)

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Table 2-Shear strength calculated by the proposed model for columns
with observed splitting cracks

<i>Author</i>	<i>Specimen</i>	<i>L</i> (mm)	<i>b</i> (mm)	<i>h</i> (mm)	<i>f_c</i> (Mpa)	<i>f_{yt}</i> (Mpa)	<i>A_v</i> (mm ²)	<i>s</i> (mm)	<i>a/d</i>	<i>n</i> = $P/f_c A_g$	<i>V_{test}</i>	<i>V_{pro}</i>	$\frac{V_{test}}{V_{pro}}$
Current tests	SP1-1.7-0.2	1200	350	350	29.8	475	56.5	125	$\frac{1.7}{1}$	0.20	238.3	208.9	1.14
	SP2-1.7-0.35	1200	350	350	29.2	475	56.5	125	$\frac{1.7}{1}$	0.35	263.2	239.3	1.10
	SP3-2.4-0.2	1700	350	350	30.6	475	56.5	125	$\frac{2.4}{3}$	0.20	193.3	135.8	1.42
	SP4-2.4-0.35	1700	350	350	28.7	475	56.5	125	$\frac{2.4}{3}$	0.35	198.0	162.0	1.22
	SR1-1.7-0.35	1700	250	490	23.3	509	56.5	125	$\frac{1.7}{3}$	0.35	263.0	274.7	0.96
	SR2-1.7-0.5	1700	250	490	22.5	509	56.5	125	$\frac{1.7}{3}$	0.50	272.0	295.4	0.92
	SW1-1.4-0.2	1700	200	610	21.7	509	56.5	125	$\frac{1.3}{9}$	0.20	246.5	284.2	0.87
	SW2-1.4-0.35	1700	200	610	20.2	509	56.5	125	$\frac{1.3}{9}$	0.35	266.0	297.0	0.90
	SW3-1.4-0.5	1700	200	610	20.5	509	56.5	125	$\frac{1.3}{9}$	0.50	289.0	323.5	0.89
Tran & Li 2012	RC-1.7-0.05	1700	250	490	32.5	392.6	56.5	125	$\frac{1.7}{3}$	0.05	283.1	216.0	1.31
	RC-1.7-0.5	1700	250	490	26.8	392.6	56.5	125	$\frac{1.7}{3}$	0.50	355.2	279.4	1.27
Moretti & Tassios 2006	specimen 7	1000	250	250	38	300	150.8	50	$\frac{2.0}{0}$	0.30	210.0	245.3	0.86
Harumi <i>et al.</i> 1990	15-0.56@90-6B	750	250	250	28.8	378	127.2	90	$\frac{1.5}{0}$	0.08	158.0	183.9	0.86
	15-0.85@60-6B	750	250	250	38	378	127.2	60	$\frac{1.5}{0}$	0.06	198.0	236.3	0.84
	15-1.28@60-6B	750	250	250	31.4	378	190.8	60	$\frac{1.5}{0}$	0.08	185.0	220.1	0.84
	30-0.56@90-6B	750	250	250	29.2	378	127.2	90	$\frac{1.5}{0}$	0.16	250.0	194.5	1.29
	45-0.56@90-6B	750	250	250	29.2	378	127.2	90	$\frac{1.5}{0}$	0.26	240.0	206.2	1.16
												Ave	1.05
												COV	0.135

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Table 3- Comparisons of shear strength calculated by proposed model and existing models for short columns

Authors	Specimens	L	b	h	f'_c	f_{yt}	A_v	s	a/d	$n = \frac{P}{f'_c A_g}$	V_{test}	$\frac{V}{[Proposed]}$	$\frac{V}{[ACI]}$	$\frac{V}{[Sezen \& Moehle]}$	$\frac{V}{[Biskinis]}$
		mm			MPa		mm^2	mm				kN			
Current tests	SP1-1.7-0.2	1200	350	350	29.8	475	56.5	125	1.71	0.20	238.3	208.9	201.2	342.8	377.9
	SP2-1.7-0.35	1200	350	350	29.2	475	56.5	125	1.71	0.35	263.2	260.5	229.0	402.2	433.8
	SP3-2.4-0.2	1700	350	350	30.6	475	56.5	125	2.43	0.20	193.3	216.1	204.1	264.5	278.3
	SP4-2.4-0.35	1700	350	350	28.7	475	56.5	125	2.43	0.35	208.1	210.1	226.4	300.0	312.5
	SR1-1.7-0.35	1700	250	490	23.3	509	56.5	125	1.73	0.35	294.4	294.4	242.0	386.6	421.4
	SR2-1.7-0.5	1700	250	490	25.7	509	56.5	125	1.73	0.50	335.8	331.0	280.7	454.1	503.4
	SW1-1.4-0.2	1700	200	610	21.7	509	56.5	125	1.39	0.20	246.5	303.5	243.1	404.9	440.0
	SW2-1.4-0.35	1700	200	610	20.2	509	56.5	125	1.39	0.35	287.0	304.1	255.7	449.4	492.3
	SW3-1.4-0.5	1700	200	610	15.1	509	56.5	125	1.39	0.50	292.6	272.9	241.1	429.7	484.1
Tran 2010	SC-2.4-0.2	1700	350	350	22.6	392.6	56.5	125	2.43	0.20	218.9	181.6	171.8	219.3	252.9
	SC-2.4-0.35	1700	350	350	24.2	392.6	56.5	125	2.43	0.35	237.6	219.0	201.7	265.1	303.5
	SC-1.7-0.05	1200	350	350	29.8	392.6	56.5	125	1.71	0.05	276.4	229.7	167.0	249.9	336.0
	SC-1.7-0.2	1200	350	350	27.5	392.6	56.5	125	1.71	0.20	294.2	221.3	190.6	319.7	384.0
	SC-1.7-0.35	1200	350	350	25.5	392.6	56.5	125	1.71	0.35	335.5	236.5	208.7	363.3	428.7
	SC-1.7-0.5	1200	350	350	26.4	392.6	56.5	125	1.71	0.50	375.6	249.2	240.6	419.8	496.2
	RC-1.7-0.05	1700	250	490	32.5	392.6	56.5	125	1.73	0.05	283.1	201.1	201.5	282.6	374.3
	RC-1.7-0.2	1700	250	490	24.5	392.6	56.5	125	1.73	0.20	305.5	239.6	207.9	322.1	386.6
	RC-1.7-0.35	1700	250	490	27.1	392.6	56.5	125	1.73	0.35	345.7	271.4	247.2	397.7	471.5
Lee 2007	RC-1.7-0.5	1700	250	490	26.8	392.6	56.5	125	1.73	0.50	355.2	288.0	274.1	444.2	533.3
	S1-0.0-N	900	300	300	27	400	140.4	180	1.50	0.10	216.0	216.6	156.3	256.4	268.9
Moretti & Tassios 2006	S2-0.0-N	900	300	300	27	400	141.8	225	1.50	0.10	200.0	195.9	141.2	241.3	268.9
	specimen 1	500	250	250	36	300	150.8	50	1.00	0.30	330.0	226.0	283.5	511.7	320.8
	specimen 2	500	250	250	48	300	150.8	50	1.00	0.60	410.0	341.0	377.1	718.6	435.6
	specimen 3	500	250	250	39	300	150.8	50	1.00	0.30	360.0	241.9	290.9	530.2	492.0
	specimen 4	500	250	250	35	300	235.6	50	1.00	0.30	360.0	210.3	388.0	612.3	466.1
	specimen 5	500	250	250	35	300	150.8	50	1.00	0.30	340.0	220.7	281.1	505.5	309.8
	specimen 6	500	250	250	39	300	150.8	50	1.00	0.30	400.0	241.9	290.9	530.2	482.0
	specimen 7	1000	250	250	38	300	150.8	50	2.00	0.30	210.0	268.0	288.5	321.3	183.9
Umemura et al. 1998	207	800	200	200	17.7	462	56.0	100	2.00	0.22	106.0	102.5	78.9	79.6	77.9
	200	800	200	200	32.9	379	26.4	120	2.00	0.12	78.0	65.2	59.3	66.0	72.7
Esaki 1996	H-2-1/5	800	200	200	23	364	51.9	50	2.00	0.20	98.8	131.5	106.8	100.4	92.5

	HT-2-1/5	800	200	200	20.2	364	77.9	75	2.00	0.20	98.5	126.3	103.3	96.3	87.9
	H-2-1/3	800	200	200	23	364	51.9	40	2.00	0.33	104.3	147.6	130.3	131.2	106.6
	HT-2-1/3	800	200	200	20.2	364	77.9	60	2.00	0.33	104.5	141.8	125.6	125.7	100.6
Amitsu <i>et al.</i> 1991	CB060C	646	278	278	46.3	414	113.0	52	1.16	0.74	509.0	471.6	485.3	631.3	636.1
Harumi <i>et al.</i> 1990	15-0.56@90-6B	750	250	250	28.8	378	127.2	90	1.50	0.08	158.0	209.6	159.4	230.0	117.4
	15-0.85@60-6B	750	250	250	38	378	127.2	60	1.50	0.06	198.0	269.6	220.6	297.4	131.5
	15-1.28@60-6B	750	250	250	31.4	378	190.8	60	1.50	0.08	185.0	247.2	295.3	367.7	121.5
	30-0.56@90-6B	750	250	250	29.2	378	127.2	90	1.50	0.16	250.0	212.1	167.6	257.0	129.8
	45-0.56@90-6B	750	250	250	29.2	378	127.2	90	1.50	0.25	240.0	216.2	175.4	279.3	141.6
Arakawa <i>et al.</i> 1989	OA2	450	180	180	31.8	249	25.1	64.3	1.25	0.18	124.3	71.9	55.4	91.3	142.4
	OA5	450	180	180	33	249	25.1	64.3	1.25	0.45	120.7	87.3	74.1	125.7	185.9
Ono <i>et al.</i> 1989	CA025C	600	200	200	25.8	426	113.0	70	1.50	0.26	130.0	156.8	165.4	172.6	118.7
	CA060C	600	200	200	25.8	426	113.0	70	1.50	0.62	135.2	172.6	185.4	205.8	161.4
Ikeda <i>et al.</i> 1986	43	1000	200	200	19.6	434	56.0	100	2.50	0.10	74.0	114.6	71.2	72.9	53.0
	44	1000	200	200	19.6	434	56.0	100	2.50	0.10	77.0	114.6	71.2	81.0	54.7
	45	1000	200	200	19.6	434	56.0	100	2.50	0.20	82.0	110.3	74.8	89.2	62.3
	46	1000	200	200	19.6	434	56.0	100	2.50	0.20	81.0	110.3	74.8	89.2	62.9
	62	1000	200	200	19.6	348	56.0	100	2.50	0.10	58.0	87.7	62.9	50.8	50.9
	63	1000	200	200	19.6	348	56.0	100	2.50	0.20	69.0	83.4	66.5	64.7	59.4
	64	1000	200	200	19.6	348	56.0	100	2.50	0.20	69.0	83.4	66.5	60.7	58.4
Kokusho <i>et al.</i> 1986	372	1000	200	200	19.9	524	62.0	100	2.50	0.20	74.0	129.5	87.8	80.2	44.6
	373	1000	200	200	20.4	524	62.0	100	2.50	0.20	88.0	152.3	88.4	93.3	60.8
	452	1000	200	200	21.9	395	62.0	100	2.50	0.45	110.0	111.7	87.1	99.0	99.2
	454	1000	200	200	21.9	395	62.0	100	2.50	0.45	110.0	111.7	87.1	99.0	118.6
Zhou <i>et al.</i> 1985, 1987	No. 806	160	80	80	32.3	341	25.1	80	1.00	0.60	30.6	22.9	19.4	36.8	17.7
	No. 1007	160	80	80	34	341	25.1	80	1.00	0.70	33.2	25.3	21.3	40.3	19.4
	No. 1309	160	80	80	32.8	341	25.1	80	1.00	0.90	32.1	26.9	23.2	43.5	21.8
	No. 104-08	320	160	160	19.8	559	39.3	40	1.00	0.80	82.8	89.4	114.5	160.2	134.3
	No. 114-08	320	160	160	19.8	559	39.3	40	1.00	0.80	90.6	89.4	114.5	160.2	134.3
	No. 124-08	320	160	160	19.8	559	66.9	40	1.00	0.80	108.0	96.5	169.6	202.6	134.3
Bett <i>et al.</i> 1985	No. 1-1	914	305	305	29.9	414	56.5	210	1.50	0.10	209.5	159.1	122.2	175.9	299.1
Ohue <i>et al.</i> 1985	2D16RS	800	200	200	32	316	47.5	50	2.00	0.14	96.6	127.9	99.8	127.5	100.6
	4D13RS	800	200	200	29.9	316	47.5	50	2.00	0.15	106.0	123.3	98.2	119.6	97.0
Umehara and Jirsa 1984	CUS	910	410	230	34.9	414	56.5	89	1.98	0.16	296.0	195.5	312.4	291.1	480.3
	CUW	910	230	410	34.9	414	56.5	89	1.11	0.16	254.0	293.7	113.8	394.4	202.2
	2CUS	910	410	230	42	414	113.0	89	1.98	0.27	375.5	312.9	500.1	459.4	575.3
Woodward	C-86-14	910	300	300	34.8	414	56.5	65	1.52	0.20	267.0	282.9	167.5	172.5	141.2

and Jirsa 1984	C-86-09	910	300	300	32.1	414	56.5	101	1.52	0.22	289.3	223.0	144.5	151.4	137.9
	C-86-03	910	300	300	42.1	414	56.5	303	1.52	0.14	324.9	209.3	127.0	130.4	142.0
	C-84-32	910	300	300	33.5	414	56.5	28	1.52	0.21	280.4	371.9	231.1	237.0	139.6
	C-84-21	910	300	300	33.5	414	56.5	43	1.52	0.21	275.9	323.5	190.8	196.8	132.3
	C-84-14	910	300	300	37.6	414	56.5	65	1.52	0.18	271.5	298.3	172.0	175.1	136.9
Imai and Yamamoto 1983	No. 1	1650	400	500	27.1	336	127.2	100	1.65	0.07	442.9	506.0	372.2	527.3	619.6
Arakawa <i>et al.</i> 1982	No. 102	750	250	250	20.6	323	71.2	32	1.50	0.33	159.7	185.3	182.0	282.4	98.9
Nagasaka <i>et al.</i> 1982	HPRC10-63	600	200	200	21.6	344	47.5	35	1.50	0.17	87.2	115.7	120.4	164.4	82.7
	HPRC19-32	600	200	200	21	344	47.5	20	1.50	0.35	111.7	128.2	190.9	248.7	97.7
Ramirez and Jirsa 1980	00-U	915	305	305	34.5	455	63.3	65.3	1.50	0.00	265.0	422.2	199.7	264.8	281.7
	120C-U	915	305	305	30.7	455	63.3	65.3	1.50	0.19	290.0	292.7	226.8	360.0	332.8
Ave (V_i/V_{Test})												1.04	1.21	0.90	1.08
COV (V_i/V_{Test})												0.28	0.31	0.35	0.40

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