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#### A Simplified Axial-Shear-Flexure Interaction Approach for Load and Displacement Capacity of Reinforced Concrete Columns

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#### ABSTRACT

A simple performance-based evaluation approach is presented based on interactions of axial, shear, and flexure mechanisms to estimate the axial and lateral deformation and load capacities of reinforced concrete columns. The developed model is based on a simplification of a consistent but relatively more complex approach known as the axial-shear-flexure interaction (ASFI) method, which is able to predict the full load-deformation response of reinforced concrete columns subjected to axial, flexure and shear force. The analytical model was developed by coupling an axial-shear model, which is a biaxial shear model, and an axial-flexure model, which is the traditional section analysis. Axial deformation interaction is the main compatibility condition taken into account in this approach. Equilibrium conditions are satisfied through a simple shear and flexure stress relation. A series of reinforced concrete columns subjected to axial and lateral loads, tested by various researchers, were evaluated by the simple developed approach and the results were compared with the test data; consistent correlation and agreement were achieved. This paper describes the formulation, implementation and verification of the modified approach. A future attempt is to modify the ASFI method for response estimation of reinforced concrete columns in fire under axial load and lateral deformation induced by thermal expansion.

#### INTRODUCTION

The performance of reinforced concrete columns under axial, flexural and shear loads has been studied for many years. However, a remaining challenge has been the development of a reliable methodology for estimating the ultimate deformation capacity of columns. Several experimental studies, such as those by Elwood and Moehle (2005) and Park et al. (1982), showed that reinforced concrete columns subjected to axial load and lateral load with similar ultimate strength may fail in significantly different ultimate deformations. Lynn et al. (1996) tested two columns with similar material and mechanical properties but with different ratios of longitudinal reinforcement. The results showed that although the difference between the ultimate strength of the two columns was only 8%, completely different ultimate deformations for performance-based design, one of the main performance properties in the design process is the ductility and

deformability of the structure. The more ductility the structure possesses, the better the performance and the more economical the design. Therefore, it is essential to have and apply a suitable analytical tool to accurately estimate the ultimate deformation or ductility of reinforced concrete column elements.

Mostafaei and Kabeyasawa (2007) developed a displacement-based analytical method for modeling the load-deformation response of reinforced concrete columns under axial and lateral loads. The model was developed to include the effects of shear deformations in sectional analyses through a method called Axial-Shear-Flexure Interaction (ASFI). The main deformation component of the interaction was the axial deformation, which was extracted from an axial-flexure model and manipulated into an axial-shear model. This was done by satisfying compatibility and equilibrium conditions for both the flexure and shear mechanisms employed in the ASFI method. In the method, the flexure mechanism was modeled by applying traditional section analysis techniques, and the shear behavior was modeled based on the Modified Compression Field Theory (MCFT), (Vecchio and Collins 1986). The approach was implemented and verified for a number of reinforced concrete columns tested with different axial loads, transverse reinforcement ratios, longitudinal reinforcement ratios, and scales ranging from onethird to full-scale specimens. Later, the shear model of the ASFI method was simplified and a method called the Uniaxial-Shear-Flexure Model (USFM) was developed (Mostafaei and Vecchio 2008). In the USFM method, a compression softening factor is employed for degradation of compressive concrete of the stress block, the value of which is determined according to a simplification on the Modified Compression Field Theory. The USFM method is also capable of producing the load-deformation response of the reinforced concrete columns with relatively moderate to high applied shear stress.

Later, further simplifications were implemented in the ASFI and the USFM models by defining three general failure criteria for reinforced concrete columns (Mostafaei et al. 2009). The three main failures, for typical reinforced concrete columns in buildings, are tension-shear failure across cracks, loss of concrete compression strength, and compression-shear failure, for both shear- and flexure-dominated members.

One of the dilemmas for the simplified ASFI methods was its application for columns with very low shear stress; columns with high shear strength and substantially low flexural capacity. This resulted in overestimating the ultimate deformation for such columns by the approach. The suggestion provided for these columns with low shear stress was to terminate analysis when compression softening factor reached less than 0.15, (Mostafaei, et al. 2009). A modification is presented in this study to overcome this limitation based on defining a plastic hinge length and calculating a shear crack angle for the columns with very low shear stress.

Further modifications are implemented currently to employ the ASFI method for response prediction of reinforced concrete columns in fire and after fire exposure. This includes post-fire seismic capacity and thermal lateral deformation capacity of the reinforced concrete columns. Studies are suggested on developing a suitable method to estimate length of the plastic zone in the axial-flexure model and the shear crack angle at the failure stage in the axial-shear model.

#### CONCEPT OF THE ASFI AND THE USFM METHODS

The main concept and methodology of both the ASFI and the USFM methods are based on the axial deformation interaction between the two models: a flexure model based on traditional uniaxial section analysis principles, and a shear model based on a biaxial shear element approach. Experimental results of reinforced concrete columns subjected to axial and lateral loads typically showed that the first cracks to appear on column were flexural cracks and shear cracks were observed afterward (Ousalem et al. 2003). In other words, the preliminary behavior of reinforced concrete columns under lateral load is flexural; but, they ultimately fail in shear. Columns with low shear capacity may fail soon after the initial flexural deformations; however, those with higher amounts of lateral reinforcement may fail after large lateral deformation in shear.

Fig. 1 provides a conceptual illustration of the ASFI approach. The figure shows that the flexural deformation/crack interacts with the shear deformation/crack at the centroid of the section in the axial direction; the flexure deformation results in an increase in the centroidal strain which in turn enlarges the shear crack and deformation. The centroidal strain in the flexure mechanism,  $\varepsilon_{cf}$ , of the axial-flexure model, is composed of the pure axial strain,  $\varepsilon_{xaf}$ , due to only the applied axial load, and flexural-axial strain,  $\varepsilon_{xf}$  due to the flexure deformation/crack. On the other hand, centroidal strain in shear mechanism,  $\varepsilon_{cs}$ , of the axial-shear model, is composed of the pure axial strain,  $\varepsilon_{xas}$ , due to only the applied axial load, and shear-axial strain,  $\varepsilon_{xs}$ , due to the shear deformation/crack. The compatibility condition requires identical axial deformation due to the applied axial load for the two mechanisms; thus,  $\varepsilon_{xa} = \varepsilon_{xaf} = \varepsilon_{xas}$ . Therefore, the total column's axial deformation,  $\varepsilon_x$ , is defined as.

$$\varepsilon_x = \varepsilon_{xa} + \varepsilon_{xs} + \varepsilon_{xf} \tag{1}$$

To obtain  $\varepsilon_x$  in Eq. (1),  $\varepsilon_{xf}$  must be extracted from  $\varepsilon_{cf}$  and added to  $\varepsilon_{cs}$ . The total lateral drift of a column,  $\gamma$ , is defined as the sum of shear strain,  $\gamma_s$ , and the flexural drift ratio,  $\gamma_f$  between the two sections.



Fig. 1 Conceptual illustration for effect of flexural deformation on shear crack widening in a reinforced concrete column

$$\gamma = \gamma_s + \gamma_f \tag{2}$$

Lateral deformation due to pullout of tensile reinforcement is ignored in this study. However, a suitable pullout model can be applied to include this effect, which typically results in a larger ultimate deformation (Mostafaei and Kabeyasawa 2007). Equilibrium of the shear and axial stresses from the axial-flexure model,  $\tau_f$  and  $\sigma_{xf}$ , and from the axial-shear model,  $\tau_s$  and  $\sigma_{xs}$ , respectively, must be satisfied simultaneously through the analysis. That is,

$$\sigma_{xf} = \sigma_{xs} = \sigma_x \tag{3}$$

$$\tau_f = \tau_s = \tau \tag{3}$$

where  $\sigma_{xf}$  = axial stress in the axial-flexure mechanism,  $\sigma_{xs}$  = axial stress in the axialshear mechanism,  $\sigma_x$  = applied axial stress,  $\tau_f$  = shear stress in the axial-flexure mechanism,  $\tau_s$  = shear stress in the axial-shear mechanism, and  $\tau$  = applied shear stress. Stresses in axes perpendicular to the longitudinal axis of the column (i.e., the clamping stresses  $\sigma_y$  and  $\sigma_z$ ) are ignored by assuming equilibrium between the confinement pressure and the hoops stresses.

$$\sigma_{v} = \sigma_{z} = 0 \tag{5}$$

Fig. 2 illustrates the ASFI method for a reinforced concrete column with two end sections, including the equilibrium and compatibility conditions.



Fig. 2 Axial-shear-flexure interactions in ASFI method

In the USFM, an assumption is made for the average concrete compression strain. Consider a reinforced concrete column of moderate height, fixed against rotation and translation at the bottom and free at the top, subjected to in-plane lateral load and axial load as shown in Fig. 3. Given its pattern along the column (see Fig. 3-a), the concrete principal compression strain for a shear element between the two sections,  $\varepsilon_2$ , may be determined based on average values of the concrete uniaxial compression strains corresponding to the resultant forces of the concrete stress blocks.

$$\varepsilon_2 = 0.5(\varepsilon_{ci} + \varepsilon_{ci+1}) \tag{6}$$

This is the main hypothesis of the USFM; this assumption simplifies the shear model significantly from a biaxial to a uniaxial mechanism. For the column in Fig. 3, the compression strain obtained from the above equation is set equal to the average principal compression strain of the element between the two sections i and i+1.

The shear mechanism in the ASFI and the USFM methods is modeled according to the Modified Compression Field Theory (MCFT), (Vecchio and Collins 1986). It is a suitable displacement-based evaluation approach for predicting the load-deformation response of reinforced concrete membrane elements subjected to shear and normal stresses. The MCFT is essentially a smeared rotating crack model. It includes compression softening effects, tension stiffening effects, and consideration of local conditions at cracks. The MCFT is based on orientations of the principal average strains in an element leading to the calculation of principal average stresses in concrete through concrete constitutive relationships. Transforming the average concrete principal stresses to the global coordinate axes and adding them to the average stresses in the reinforcement gives the total average stresses in the element. There are two checks in the calculation process relating to the crack zones. The first is to ensure that tension in the concrete can be transferred across the crack. The second is to ensure that the shear stress on the surface of the crack dose not exceed the maximum shear resistance provided by aggregate interlock.



Fig. 3 A reinforced concrete column subjected to shear and axial Loads; a) Concrete principal compression stress pattern, b) Cross section, and c) Stress blocks and strains at two adjacent sections

#### ANALYTICAL MODEL

Details of the ASFI and the USFM are described by Mostafaei and Kabeyasawa (2007), Mostafaei and Vecchio (2008), and Mostafaei et al. (2009). Here, a summary of the analytical model and the process are provided.

Three shear failure conditions are defined for a reinforced concrete column under axial and shear load: shear failure at a crack (Mode 1 Failure); failure due to loss of compression strength (Mode 2 Failure); and shear-compression failure (Mode 3 Failure). In the simplified ASFI method, these three failure modes are described for a

column, similar to the one shown in Fig. 1, with a flexure section at one end, a section at the inflection point and a shear model between the two sections.

Shear failure at a crack, or Mode 1 Failure, which is typically the governing case for columns with low transverse reinforcement ratios, occurs when:

$$\tau_f = \frac{M}{bdL_{in}} \ge \tau_i + f_{syy}\rho_{sy}\cot\theta_c \tag{7}$$

where  $\tau_f$  is shear stress due to flexure mechanism, *M* is the end-moment of the column, *d* is the effective depth of the section, *b* is the width of the section,  $L_{in}$  is the length of the column from the inflection point to the end section,  $\theta_c$  is the crack angle,  $f_{syy}$  is the yield stress of transverse reinforcement,  $\rho_{sy}$  is the reinforcement ratio in the y (transverse) direction, and  $\tau_i$  is the shear stress transferred by aggregate interlock across the crack surface, determined by Walraven's equation, Eq. (8).

$$\tau_{i} \leq \frac{0.18\sqrt{f_{c}'}}{0.31 + \frac{24w}{a_{g} + 16}}$$
(MPa, mm) (8)

with  $w = s_{\theta} \varepsilon_1$ , and  $s_{\theta} = \frac{1}{\frac{\sin \theta_c}{s_x} + \frac{\cos \theta_c}{s_y}}$ , where,  $f'_c$  is the concrete compressive strength,

*w* is the average crack width,  $\varepsilon_1$  is the concrete tensile strain in shear element,  $s_x$  and  $s_y$  are the average crack spacings in the x- and y-directions, respectively and  $a_g$  is the maximum aggregate size. In this study  $s_x$  and  $s_y$  are the same as the maximum reinforcement spacing in the x- and y-directions, respectively.

Columns under high shear force, such as short columns, if not failing via Mode 1, will lose compression strength,  $f_2$ , due to shear deformation and fail before the concrete peak strain is achieved,  $\varepsilon_2 \leq \varepsilon'_c$ . This failure condition, Mode 2, takes place when:

$$\tau_f = \frac{M}{bdL_{in}} \ge \frac{(f_{c1} - f_{c2})}{(\tan\theta_c + 1/\tan\theta_c)} \tag{9}$$

Where,  $f_{c1}$  and  $f_{c2}$  are the tensile stress and compression stress in the concrete according to the shear model.

Columns and beams with a ductile flexure performance, when having sufficient transverse reinforcement and relatively low shear stress, will fail in shear when  $\varepsilon_2 = \varepsilon'_c$  via Mode 3.

$$\tau_f = \frac{M}{bdL_{in}} \tag{10}$$

After Mode 3, a column likely sustains larger lateral deformation before the complete shear failure, Mode 1 or 2; however, the lateral load may drop depending on the level of the column's confinement and the level of the damage caused to the confinement as the result of a cycling loading.

In this approach, the concrete compression softening factor was employed only within the MCFT-based shear model. This is because at the compression block of the flexure section, crack angle is nearly zero.

#### ANALYTICAL STEPS

Based on the shear failure criteria described in the previous section, an analytical procedure can be constructed to estimate the ultimate deformation of a reinforced concrete column subjected to both axial and lateral loads.

In step-by-step fashion, the procedure is as follows:

1. Assume an initial value for the concrete compression strain of the flexure section,  $\varepsilon_c$ ; for example,  $\varepsilon_c = 0.5\varepsilon'_c$ .

2. Employ a section analysis for the end section of the column and determine the centroidal strain of the section,  $\varepsilon_{cf}$ , in Fig. 2 (Mostafaei et al. 2009).

3. Determine the axial strain at the inflection point with zero moment,  $\varepsilon_{xa}$ , in Fig. 2. This is the axial deformation of the column when it is subjected only to axial load.

4. Compute the average concrete principal compression strain,  $\varepsilon_2$ , and average axial strain,  $\varepsilon_x$ , for the shear model.

$$\varepsilon_2 = \frac{\varepsilon_c + \varepsilon_{xa}}{2} \tag{11}$$

$$\varepsilon_x = \frac{\varepsilon_{cf} + \varepsilon_{xa}}{2} \tag{12}$$

5. Determine  $\varepsilon_1$  and  $\varepsilon_y$ . If the transverse bars are yielded before the longitudinal bars, Eq. (13) can be employed to determine  $\varepsilon_1$ .

$$\varepsilon_1 = \frac{(\varepsilon_x - \varepsilon_2)(f_{c1} - f_{cx})}{(f_{c1} + \rho_{sy}f_{syy})} + \varepsilon_x \text{ if } F_{cy} > f_{syy}\rho_{sy}$$
(13)

where,  $F_{cy} = -f_{cy} = [\tau_f^2 / (f_{c1} - f_{cx}) - f_1]$ ;  $f_{cx}$  and  $f_{cy}$  are concrete stresses in the x-and y-direction, respectively;  $f_{cx} = \sigma_x - \rho_{sx} E_{sx} \varepsilon_x$ ;  $E_{sx}$  is the modulus of elasticity of the main reinforcement steel, and  $\rho_{sx}$  is the reinforcement ratio in the x-direction (main bars). An average initial value of  $f_{c1} = 0.44 f'_t$  can be assumed for Eq. (13) (Mostafaei et al. 2009). If an iteration process is implemented, the value of  $f_{c1}$  can be determined according to the concrete tensile strain,  $\varepsilon_1$ , obtained from the previous iteration. On the other hand, if  $F_{cy} \leq f_{syy}\rho_{sy}$  then,  $\varepsilon_1 = \varepsilon_x + f_{syy} / E_{sy} - \varepsilon_2$ ; this is a failure condition when transverse bars yield after yielding the longitudinal bars.  $E_{sy}$  is the modulus of elasticity of the transverse bars.

6. Calculate  $\tan \theta_c$ 

$$\tan^2 \theta_c = \frac{\varepsilon_x - \varepsilon_2}{\varepsilon_y - \varepsilon_2}, \text{ for } F_{cy} \le f_{syy} \rho_{sy}, \text{ and } \tan \theta_c = \frac{f_{c1} + \rho_{sy} f_{syy}}{\tau_f} \text{ for } F_{cy} > f_{syy} \rho_{sy} \quad (14)$$

where,  $\varepsilon_y = \varepsilon_1 + \varepsilon_2 - \varepsilon_x$ .

7. Using Eqs. (7) (9), and (10), check for shear failure. If no failure has occurred, then increase  $\varepsilon_c$  and repeat the above steps. If, on the other hand, Eq. (7) shows a shear failure occurring at the crack, then  $\varepsilon_c$  must be reduced until all three equations provide greater or equal values.

8. Determine the ultimate deformation using Eq. (2), when:

$$\gamma_f = \frac{\delta}{L_{in}} = \frac{1}{L_{in}} \int_{0}^{L_{in}} x \phi dx$$
, and  $\gamma_s = \frac{2(\varepsilon_x - \varepsilon_2)}{\tan \theta_c}$ 

10. Finally, the ultimate lateral load capacity is obtained by

$$V_{u} = \tau_{f} b(h/d) [L_{in}/(L_{in} - d')]$$
(15)

where, h is the depth of the section, and d is the cover concrete. Shear force in Eq. (15) has been increased for consideration of the support confinement effect. Plastic hinge is determined by Eq. (16).

$$L_p = h / (2 \tan \theta_c) \le L_{in} \tag{16}$$

Furthermore, other possible failure modes such as buckling of the compression bars, bond failure, failure of the cover concrete and rupture of tensile bars must be checked for the columns.

Using the above process, ultimate deformations were estimated for several reinforced concrete column specimens, previously tested by different authors as listed in Table 1. Comparisons between the experimental and analysis are plotted in Fig. 4, indicating consistently accurate correlations. Since the shear capacity, obtained from the analysis, is based on the section moment capacity without consideration of geometrical nonlinearity, the P- $\Delta$  effect due to drift is determined and employed for the flexural columns, which reduces the calculated shear capacity. Failure modes are determined and given in Table 1 for all the reinforced concrete column specimens.

#### CONCLUSIONS

An analytical approach was presented to estimate the ultimate deformation and load capacity of reinforced concrete columns based on a simplified axial-shear-flexure interaction approach. Shear failure was the main failure criteria for both flexure- and shear-dominant specimens. In this approach, the concrete compression softening factor was employed only within the MCFT-based shear model. Axial strain and concrete compression strain of the shear model were determined according to centroidal and average concrete strains in the flexure model. Three failure modes were defined as the main ultimate state conditions; shear failure at the cracks, loss of concrete compression strength before the peak, and finally shear-compression failure when  $\varepsilon_2 = \varepsilon'_c$ . A modification is implemented and provided for the USFM method on defining the plastic hinge length and determining the concrete tensile strain and crack angle at failure. The

ultimate deformation and load capacity results obtained by the modified approach were verified against experimental data, and consistent correlations between the analytical and experimental results for a series of reinforced concrete columns were obtained.

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Specime	Tuna	b	h	$2L_{in}$	$S_h$	$ ho_{g}$	$ ho_w$	f <sub>syx</sub>	f <sub>syy</sub>	f'c	Р	Failure
n	rype	mm	mm	mm	mm	%	%	MPa	MPa	MPa	kN	mode
No.12 <sup>1</sup>	DC	300	300	900	150	2.26	0.14	415	410	28	540	1
No.15 <sup>1</sup>	DC	300	300	900	50	2.26	0.85	415	410	26	540	1
No.16 <sup>1</sup>	DC	300	300	600	50	1.8	0.43	450	410	27	540	2
2CLH18 <sup>2</sup>	DC	457	457	2946	457	2	0.1	330	400	33	500	3
3CLH18 <sup>2</sup>	DC	457	457	2946	457	3	0.1	330	400	25.6	500	1
$No.2^3$	DC	457	457	2946	305	2.5	0.17	434	476	21.1	2650	3
$N18M^4$	DC	300	300	900	100	2.7	0.19	380	375	26.5	429	1
No.1 <sup>5</sup>	DC	200	400	1000	128	2.53	1	360	345	45	0	1
No.4 <sup>6</sup>	DC	400	400	3200	80	1.57	1.1	474	333	25.6	819	3
No.7 <sup>6</sup>	SC	550	550	3300	90	1.25	1	511	325	32.1	2913	3
$B2^7$	DC	250	250	1000	40	2.43	0.4	379	774	99.5	2176	3
TP-30 <sup>8</sup>	SC	242	242	1250	40	2.43	0.8	461	486	37.6	705	3
D1N3 <sup>9</sup>	SC	242	242	1250	40	2.43	0.8	461	486	37.6	705	3
D1N6 <sup>9</sup>	SC	242	242	1250	40	2.43	0.8	461	486	37.6	1410	3

Table 1 Material property of the test specimens

**Footnotes:** DC= double curvature, or with two fixed ends, SC=single curvature, or cantilever, b=width of the section, h= Depth of the section,  $L_{in}$ = length of the column from the inflection point to the end section,  $S_h$ = hoop spacing,  $\rho_g$ =longitudinal reinforcement ratio,  $\rho_w$ = transverse reinforcement ratio,  $f_{syx}$ = longitudinal reinforcement yield stress,  $f_{c}$ = concrete compression strength , P=axial load, Failure mode 1: shear failure at crack  $\epsilon_2 < \epsilon'_c$ , Failure mode 2: loss of compression strength  $\epsilon_2 < \epsilon'_c$ , and Failure mode 3: shear-compression failure  $\epsilon_2 = \epsilon'_c$ , Test results by: <sup>1</sup>Ousalem et al. (2003), <sup>2</sup>Lynn (1996), <sup>3</sup>Sezen (2000), <sup>4</sup>Nakamura and Yoshimura (2002), <sup>5</sup>Umemura et al. (1977), <sup>6</sup>Tanaka and Park (1990), <sup>7</sup>Sakai et al. (1990), <sup>8</sup>Nagaya and Kawashima (2001), and <sup>9</sup>Kono and Watanabe (2000).



Fig. 4 Comparison of experimental and analytical results



Fig. 4 Continued