- Technical Paper -

INVESTIGATING SHEAR TRANSFER ACROSS CRACKS IN HIGH PERFORMANCE STEEL FIBER-REINFORCED CONCRETE

Benny SURYANTO[*1], Kohei NAGAI[*2], and Koichi MAEKAWA[*3]

ABSTRACT

This paper presents a rational shear transfer model for *high-strength* steel fiber-reinforced concrete (HSSFRC). The model inherently considers shear transfer across smooth cracks, bridged by stiff steel fibers, and the resulting crack slip. When implemented into a nonlinear FE algorithm, the model is shown to perform well in simulating the response of uniaxially reinforced HSSFRC panels in shear, including load-deformation response and inclination of principal stress-strain fields. It is shown that while steel fibers are effective in limiting crack slip, they may also deteriorate shear transfer capacity.

Keywords: steel fiber, shear transfer, slip, panel test, principal strain, stress rotation

1. INTRODUCTION

A considerable amount of research has been conducted at the University of Toronto, aiming at investigating the shear characteristics of reinforced concrete (RC) panels subjected to various conditions of in-plane stresses [1,2,3,4, amongst other]. The panels tested covered a wide range of test parameters, and hence, exhibited various responses and failure modes.

The most common way to evaluate the response of the panels is by looking at the load-deformation response. A ductile response corresponds to a panel that the failure is dictated by steel yielding, while a brittle response corresponds to that dictated by concrete compression failure. While this evaluation way provides direct and useful information, it does not provide insight on the mechanism of stress transfer.

To date, considerably less attention has been devoted to evaluate the angles of principal stress and principal strain fields of the panels. It will be shown here that they indeed provide useful and valuable insights on the mechanism of crack-shear transfer. This is particularly true in panels with little or no transverse reinforcement (e.g.:$\rho_x>\rho_y$; see Fig. 1) [1,5].

For clarity, consider Panel PV10, tested by Vecchio and Collins [1], shown in Fig. 1(a). As shown, the inclinations of the strain field after first cracking are always in a larger angle than those of the stress field. Vecchio clearly demonstrated that the lag angle between the two fields was attributed to crack-shear slip, which soon became the fundamental assumption of an alternative, rational theory called DSFM [5,6].

Recently, eight *high-strength* fiber-reinforced concrete (HS-SFRC) panels were tested in pure shear by Susetyo [7]. The panels tested were uniaxially reinforced and cast with a newly-developed material, falling in the category of high performance fiber-

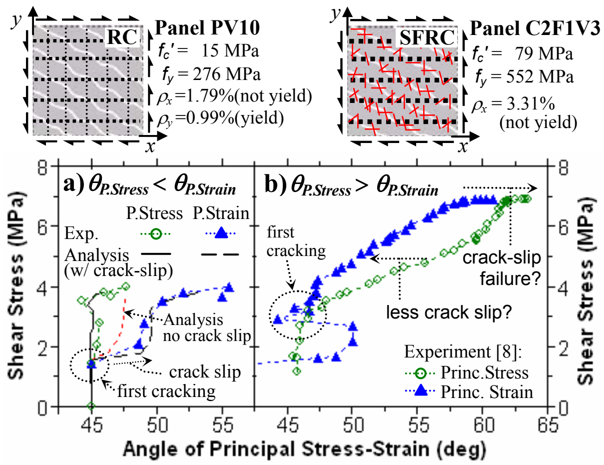

Fig.1 Comparison of Principal Angle of: (a) RC Panel PV10 [6],(b) SFRC Panel C2F1V3 [7]

reinforced concrete [8]. Of particular interest of the study was to investigate the effectiveness of steel fibers as a shrinkage crack control [7], and no attempt was made to study shear transfer across cracks. Here, the authors recognize that the evaluation on the response of the panels will provide an opportunity to characterize crack-shear transfer of the new material.

In response to this opportunity, consider the plot shown in Fig. 1(b). The plot relates to the inclination of principal stress-strain fields of SFRC Panel C2F1V3, which contains 3.31% of longitudinal reinforcement and 1.5% of steel fibers with fiber-aspect-to-length ratio of 80/50. Comparison of the two plots shown in Fig. 1 reveals one interesting phenomenon. The inclinations of the stress field in the SFRC panel, after first cracking, surprisingly exceed those of the strain field, rather than always lag behind as the RC panel shows. Implicitly, this phenomenon indicates that the crack slip in the SFRC panel is much limited. Near the

*1 Post Doctoral Fellow, Dept. of Civil Engineering, The University of Tokyo, Ph. D., JCI Member
*2 Lecturer, Dept. of Civil Engineering, The University of Tokyo, Dr. E., JCI Member
*3 Professor, Dept. of Civil Engineering, The University of Tokyo, Dr. E., JCI Member

ultimate stage, the lag angles of the two fields quickly diminish and ultimately almost coincide to each other, indicating either significant crack slips of the existing cracks or formation of new cracks at a different angle.

To evaluate these indications in more details, numerical analyses were carried out. The analysis framework for reinforced concrete in the context of smeared, fixed crack approach was used [9]. This analysis framework was chosen as separate tensile and shear models are employed to represent average tensile stress and shear stress transfers in the concrete at cracks, providing a flexible way to study stress transfer at cracks. This paper presents the results to date and indicates directions of further possible development.

2. EXPERIMENTAL PROGRAM

The experimental program referred involved the testing of two *high-strength* RC panels and eight *high-strength* steel fiber-reinforced concrete (SFRC) panels, carried out extensively by Susetyo [7]. The RC panels were orthogonally reinforced, while the SFRC panels were uniaxially reinforced. They were 890 mm square and 70 mm thick; all were tested using the shear rig test facility by applying uniform shear stresses to the shear keys at the perimeter edges of the panels.

Of the ten panels tested, only the results of the five panels are discussed in this paper, namely Panels C1C, C1F1V1, C1F1V2, C1F1V3, and C2F1V3. For clarity, the details of the panels are illustrated in Fig. 2, while the property of the materials incorporated is listed in Table 1. The test parameters of the five panels include compressive strength f_c' and fiber volume fraction V_f (0.5, 1.0, 1.5%). The fiber-aspect-to-length ratio $L_f/d_f\,/\,L_f$ is constant and approximately 80/50.

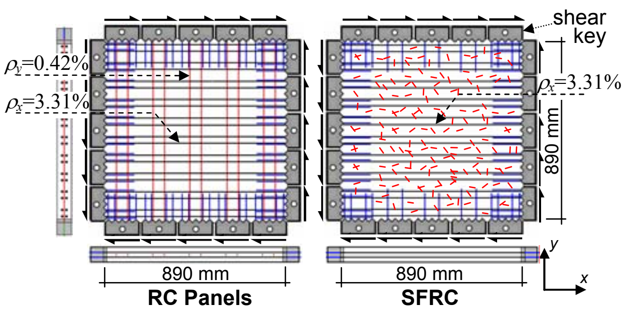

Fig.2 Typical Reinforcement Details [7]

Table 1 Material Properties of the Test Panels [7]

Panel	ρ_x[#] (%)	$f_{y\text{-}x}$[&] (MPa)	ρ_y[$] (%)	$f_{y\text{-}y}$[&] (MPa)	f_c' (MPa)	V_f[@] (%)	L_f/d_f / L_f
C1C-R	3.31	552	0.42	442	65.7	–	–
C1F1V1	3.31	552	–	–	51.4	0.5	80/50
C1F1V2	3.31	552	–	–	53.4	1.0	80/50
C1F1V3	3.31	552	–	–	49.7	1.5	80/50
C2C	3.31	552	0.42	442	90.5	–	–
C1F1V3	3.31	552	–	–	78.8	1.5	80/50

[#]2-layers of D8 (21 mm apart); [&]taken as the stress at the limit proportionality; [$]1-layer of D4;
[@]hooked-end type, diameter 0.62 mm, length 50 mm, tensile strength 1,050 N/mm^2.

3. MATERIAL AND GEOMETRICAL MODELING

The material models presented in this paper are the two-dimensional constitutive models of reinforced concrete, which the full documentation of the models is available in Maekawa *et al* [9]. To deal with SFRC, most aspects of the models were retained as they are also applicable for SFRC. Presented herein are the modifications made to the parameters of the models.

3.1 Compression Model

The compression model used was the elasto-plastic fracture model [9]. No modifications were made to the model except for the compression softening factor ω. For *high-strength* concrete, it was presumed that the degree of compression softening is less. This may happen as concrete strips/struts between two cracks are not much disturbed due to smooth crack surface. Unfortunately, the test panels in this series, as will be shown, exhibited a low compressive stress, which made the factor ω difficult to be quantified. At this current development, no compression softening factor was assumed (ω=1.0).

3.2 Tension Model

Observations on the response of the panel revealed that the tensile strain-hardening response of the material was not significant [7]. For this reason, the tension model of concrete proposed by Okamura and Maekawa [9] was referred with a slight modification to the value of the c factor. The formulation is given by:

$$f_t = f_{tu}\left(\frac{\varepsilon_{tu}}{\varepsilon}\right)^c \qquad (1)$$

where ε_{tu} is the strain that the tensile softening/stiffening starts ($2\varepsilon_{cr}$, e.g.: 0.02%), f_{tu} is the tensile strength, and c is the parameter that controls the degree of tensile stiffening/softening. In RC, it was shown that a value of c=0.4 correlated well with a wide range of tests [9]. In SFRC, it is expected that the steel-fibers can provide an extra average tensile stress due to fiber bridging at cracks. Adopting Eq. 1, this can be accommodated by lowering the c value.

It is understood that the model adopted is physically incorrect as the observed tensile stress, after first cracking, slightly increases or relatively constant. Since the observed strain corresponding to the last hardening is insignificant, the adoption results in negligible error. Examples will be shown herein to confirm this argument. Another simplifying assumption made herein is that the average SFRC tensile response with embedded rebar is the same as that without rebar.

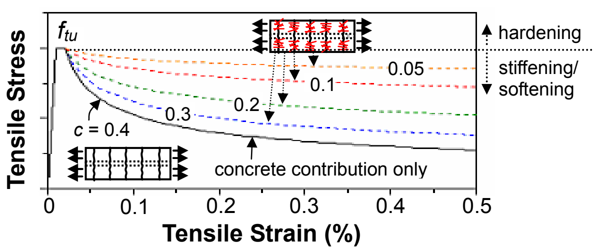

Fig. 3 Tensile Stress-Strain for Various c Values

3.3 Shear Transfer Model

The proposed shear transfer model includes the contribution of crack-interlocking by smooth crack surface and crack-bridging by stiff steel fibers. For the basic formulation, the shear transfer model based on the contact density theory [9] was adopted. To deal with SFRC, two parameters A and B were introduced to the model as given by:

$$v_{cr} = A.f_{st}\frac{(B.\beta)^2}{1+(B.\beta)^2} \tag{2}$$

where v_{cr} is the shear stress transferred along the crack surface, f_{st} is the intrinsic shear strength and is given by $f_{st} = 3.8 f_c^{'1/3}$ ($f_c^{'}$ in MPa), and β is the ratio of shear strain due to crack slip (γ_{cr}) to tensile strain due to crack opening (ε_t). The parameter A relates to the shear transfer capacity, while the parameter B relates to the shear stiffness for a given crack opening ε_t. It is expected that the value of A relies more on the crack roughness and less on the fiber volume, while B solely on the fiber volume. For clarity, Fig. 4 shows the role of each parameter on shear stress-strain response obtained for f_c'= 50 MPa, A=0.5, and various B values.

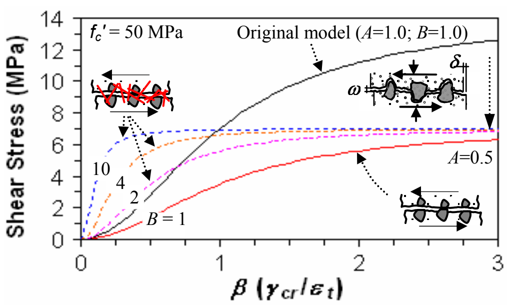

Fig.4 Illustration of Shear Transfer Response for A=0.5 and Various B Values

3.4 Modeling of the Test Panels

A three-dimensional 8-node plate element was used to model the test plate, but only its in-plane degree-of-freedom was used. The analysis was performed with the nonlinear FE program COM3 [9] and under a load controlled condition. The analysis employed the computational algorithm of the four-way fixed, rotating-mixed crack model. This special crack algorithm was selected as the arrangement of the reinforcement in the test panels was highly anisotropic. New cracks can form at a certain angle, if the stress dictates. If not, a slight orientation of the former crack is allowed. More detailed descriptions of the algorithm can be found in Maekawa *et al* [9].

Material properties of the concrete, SFRC, and reinforcement were as reported as previously listed in Table 1, except for the tensile properties of both concrete and SFRC, which were determined close to those obtained from the panel tests. This was because the tensile response of the SFRC observed from the panel tests differed remarkably from those obtained from the tensile tests. This was likely due to various secondary effects such as fiber orientation and shrinkage, and will be addressed in a further study.

3. COMPARISON OF EXPERIMENT AND ANALYSIS RESULTS

Fig. 5 shows the comparison of the observed and predicted response of the selected five panels in terms of overall load-deformation, inclination of principal stress-strain direction, and principal stress-strain relations. All parameters that result in predictions with the best correlation are listed in Table 2. The analyses were performed until the shear strain at the crack coordinate reached 0.5% unless failure occurs earlier.

Table 2 Tensile and Shear Properties in Analysis

Panel	*Tensile model*		*Shear model*	
	$f_t^{\$}$ MPa	*C* –	*A* –	*B* –
C1C	3.00	0.4, 0.2#	0.75	1
C1F1V1	3.20	0.25	0.35	1
C1F1V2	3.10	0.15	0.35	4
C1F1V3	3.25	0.10	0.35	5
C2C	3.00	0.4, 0.2#	0.75	1
C1F1V3	3.50	0.125	0.40	5

$ assumed value; # c_y, assumed as $\rho_{cr\text{-}y} \cong f_t / f_{y\text{-}y}$

Panel C1C (RC, f_c'=65.7 MPa)

Panel C1C contained no steel fibers, and therefore suitable to evaluate crack-shear transfer in *high-strength* concrete. As can be seen, the analysis with A=0.75 shows a good agreement to the observed response, in terms of the overall shear stress-strain and the angle deviation between the inclination of principal stress and principal strain. While a smaller reduction factor (A=0.35) was considered, the correlation between the predicted and observed response is not as good. The overall response becomes softer and the deviation between the principal stress-strain directions becomes much larger, indicating an over-estimation of crack slip.

The close agreement of prediction with A=0.75 to the observed response indicates that the difference between shear transfer of *high-* and *normal-strength* concrete in this test series is marginal. This is likely due to the fact that the concrete matrix was not so much strong (as indicated by the f_c'), allowing some cracks in the concrete propagated around aggregates, rather than always passed through them.

Panel C1F1V1 (SFRC, V_f=0.5%, f_c'=51.4 MPa)

Panel C1F1V1 was uniaxially reinforced and contained relatively low fibers content, a content that usually considered for shrinkage control in many practical situations. The response of this panel is thus useful to verify whether a few amounts of fibers can contribute to the structural enhancement or not.

The use of A=0.75 (similar to the previous case) overestimates somewhat the panel capacity. The predicted angles of principal stress-strain also remain coincide to each other, indicating an overestimation of crack-shear transfer, and obviously against the observed response. While the panel was re-analyzed with A=0.35, a better agreement can be observed. Although the peak load is still slightly underestimated, the analysis now correctly predicts the significant lag between the inclinations of principal stress and strain.

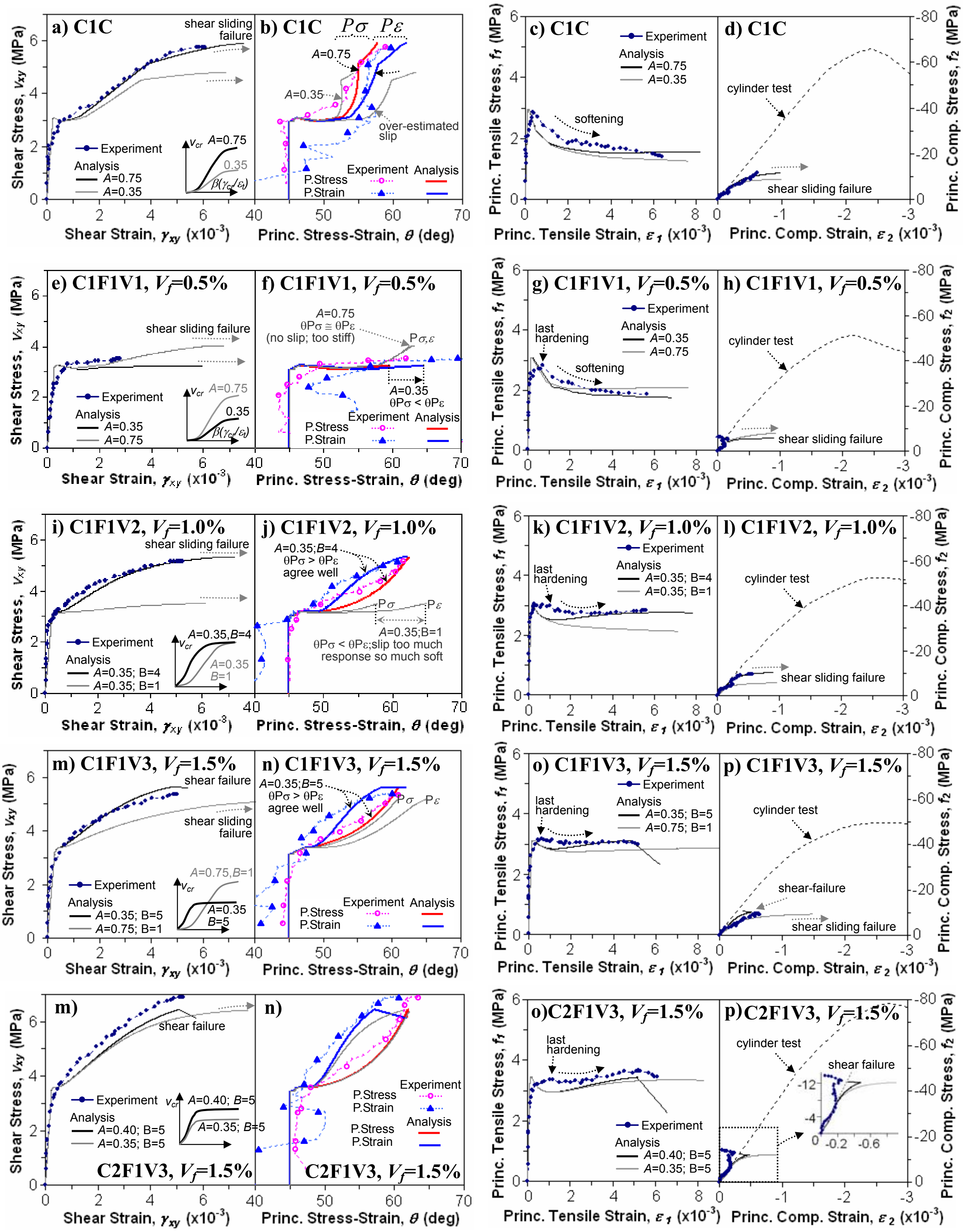

Fig.5 Comparison of the Observed [7] and Predicted Response for Selected Test Panels

In regards to the reduced shear transfer capacity (from A=0.75 to A=0.35), there are two probable factors. *First*, the SFRC mixtures contained a fewer amount of coarse aggregate (792 kg/m^3 and 595 kg/m^3, about 70% and 55% of that of the control panels, for C1- and C2-series, respectively). *Second*, the addition of steel fibers may introduce additional damage that reduces the degree of aggregate interlock. At this stage, it is not known, however, which factor is more influencing.

What is known is that the ability of cracks to transmit tensile and shear stresses is equally important. Nevertheless, Susetyo reported that it was difficult to use the same aggregates amount [7]. Thus, the reduction in shear resistance tends to be unavoidable.

Panel C1F1V2 (SFRC, $\underline{V_f=1.0\%}$, f_c'=53.4 MPa)

The parametric analysis shows that while the crack-shear transfer capacity remains approximately the same (A=0.35), the addition of fibers significantly improves the crack-shear slip resistance. Ignoring the contribution of steel fibers in limiting slip (B=1) results in a considerable underestimation of the plate capacity. The inclination angle of the principal stress-strain is also incorrectly predicted. In contrast, accounting its contribution (B=4) results in a much better agreement. It correctly predicts the lag between the two fields at intermediate load stages and the gradual coincidence of the two at loads near the ultimate stage [see Fig. 5(j)]. A better correlation of the internal principal tensile stress-strain response is also seen [see Fig. 5(k)].

Panel C1F1V3 (SFRC, $\underline{V_f=1.5\%}$, f_c'=49.7 MPa)

Compared to the response of the previous panel, the parametric analysis shows that by increasing the fiber volume by 0.5%, the resistance of the crack to slip (B factor) increases from 4 to 5. It is worth to note that the predicted response with A=0.75, B=1 (Panel C1C) incorrectly predicts both load-deformation and principal stress-strain angles [see Fig. 5(m, n)].

Once again, the analysis can simulate not only the load-deformation response, but also the lag between the stress-strain fields. At intermediate loading, the lag is significant, indicating a limited slip, and quickly diminishes near the ultimate stage. The results support that adding a sufficient amount of steel hooked-end fibers are effective in limiting crack-shear slip.

Panel C2F1V3 (SFRC, V_f=1.5%, $\underline{f_c'=78.8\text{ MPa}}$)

Susetyo reported that the fibers at cracks mostly pulled out, rather than being ruptured [7]. It was therefore expected that increasing compressive strength would improve the bond between fibers and matrix, and thereby improving the fiber bridging mechanism.

The parametric analysis presented shows that the increase of compressive strength slightly improves the crack-shear transfer capacity (from 0.35 to 0.4), while the slip resistance tends to be the same (B=5). The improvement of shear transfer capacity is likely attributed to the improved fibers bridging as the contribution of crack interlock should somewhat decrease (the higher the f_c', the stronger the bond between fiber and matrix, but the higher the probability of the aggregates to break). Confirming the experiment, the analysis predicts that the panel failed in shear-compression failure [see Fig. 5(p)].

4. DISCUSSION ON THE INFLUENCING FACTORS

4.1 Influence of Fiber Volume in the Parameters of the Tension and Shear Models

Fig. 6 shows the relationship between the fiber volume and the parameters in the tension and shear models. It can be seen that the tension stiffening coefficient c decreases as the fiber volume increases. This indicates clearly the advantage of adding fibers to improve the ability of the concrete to resist tension. Nevertheless, it is expected that the improvement is effective until a certain fiber volume only.

For the parameters in the shear model, it is evident that the B parameter is strongly affected by fiber volume, while the parameter A is not. Since the parameter A is related to crack-shear resistance, the trend shown affirms the judgment by Susetyo [8], which stated that the fibers pulled out from the concrete and hence limits the shear transfer capacity. The trend shown by the B parameter strengthens the usefulness of stiff fibers in limiting crack slip.

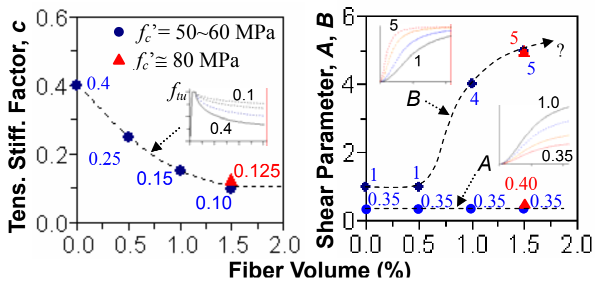

Fig.6 Influence of Fiber Volume on Coefficients c in the Tension model, and A - B in the Shear Model

4.2 Influence of the Coefficients Formulated in the Shear Transfer Model

To gain deeper insight regarding the contribution of the coefficients proposed in the formulation of shear transfer model, Panel C1F1V3 was re-analyzed with various A and B factors in which the results are plotted in Fig. 7 and Fig. 8, respectively.

(1) Crack-Shear Transfer Capacity (A factor)

Assuming the steel fibers only affect the shear transfer capacity (A=1.5…0.35) and ignoring the contribution of the steel fibers in preventing crack slip (B = 1), the parametric analyses indicate that none of the predictions can represent the observed response.

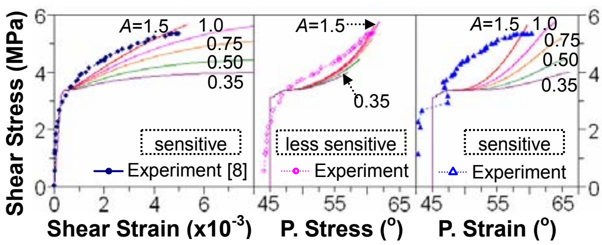

Fig.7 Effects of Shear Strength Reduction Factor A on the Response of Panel C1F1V3

The analysis with strong crack shear transfer resistance (A=1.5, stronger than aggregate interlock) fairly represents the observed load-deformation response, yet incorrectly predicts the inclination of principal strain. This reminds the role of the principal strain field data to capture the true shear transfer mechanism and to avoid false conclusion.

(2) Crack-Shear Slip Resistance (B factor)

Assuming no crack-shear transfer capacity as that of C1C (A=0.75) and considering the steel fibers only affect to prevent crack shear slip (B=1…5), the parametric analyses indicate that the predicted response is extremely sensitive to the shear transfer model adopted. The predicted responses are in a wide range, from a ductile shear failure (B=1) to a sudden shear failure (B=5). The best estimate response is seen with B=2, yet again, it cannot represent the observed principal strain [compare to Fig. 5(m) and (n)].

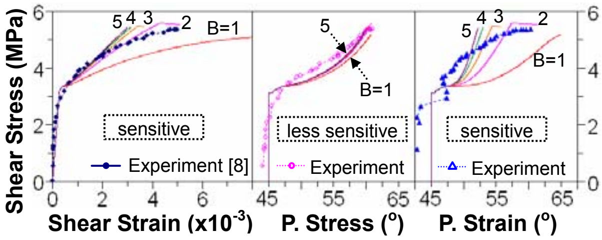

Fig.8 Effects of Shear Slip Resistance *B* on the Response of Panel C1F1V3

5. CONCLUSIONS

This paper describes an approach that can rationally evaluate the mechanisms of shear transfer across cracks in *high-strength* SFRC. The following conclusions can be drawn:

(1) The inclination of principal stress-strain fields is a good indicator to understand the mechanism of crack-shear transfer and the resulting crack slip in RC and SFRC panels with little or no transverse reinforcement.

(2) The less inclined of principal strain-to-stress field is confirmed and is not merely an experimental scatter. This supports the perception by Vecchio [5] and against the argument by Hsu [10].

(3) The stiff load-deformation response and the less inclined of principal stress-to-strain field in the SFRC panels can be rationally explained by incorporating a shear transfer model that is high in stiffness, but low in capacity. High stiffness reflects the significant contribution of the steel hooked-end fibers in limiting shear slip, while low capacity represents a crack plane with moderate roughness and substantial damage.

(4) When the shear model is incorporated in a nonlinear FE program, it provides a response that correlated well with the response of five SFRC panels in shear. The load-deformation, inclination angle of principal-stress-strain, and internal principal stress-strain responses are successfully simulated.

(5) The proposed shear model is deemed appropriate for representing shear transfer behavior of *high-strength* SFRC under mixed mode deformation, where the cracks in the concrete simultaneously open and slip.

(6) Ignoring either the shear capacity reduction factor or the shear stiffening factor results in predictions with either significant difference in response or gross underestimation in load capacity.

(7) The analysis confirms the experimental finding that at least 1.0% steel fibers is necessary to limit crack-shear slip effectively. However, the use of 1.5% steel fibers results in only a marginal improvement.

(8) The analysis shows that the inclination of the stress direction is much less sensitive than that of the strain field. This means that if the information on the stress fields are not available (which are difficult to be measured in most experiments), it is still possible to capture the mechanisms of crack-shear transfer by observing the strain fields.

ACKNOWLEDGEMENT

The authors deeply acknowledge the Vector Analysis Group at the University of Toronto, especially to Prof. F. J. Vecchio and to Dr. J. Susetyo, for sharing the experimental data to researchers worldwide. The first author also thanks the Japanese Scholarship Promoting Science (JSPS) for the postdoctoral fellowship.

REFERENCES

[1] Vecchio, F. J., and Collins, M. P., "Response of Reinforced Concrete to In-Plane Shear and Normal Stresses," Report No. 82-03, Dept. of Civil Eng., University of Toronto, Toronto, 1982

[2] Bhide, and Collins, M. P., "Influence of Axial Tension on the Shear Capacity of Reinforced Concrete Members," ACI Structural J., Vol.86, No.5, May-June, 1988, pp.570-581

[3] Vecchio, F. J., and Nieto, M., "Shear Friction Tests on Reinforced Concrete Panels," ACI Structural J., Vol.88, No.3, May-June, 1991, pp.371-379

[4] Vecchio, F. J., Collins, M. P., and Aspiotis, J., "High-Strength Concrete Elements Subjected to Shear," ACI Structural J., Vol.91, No.4, July-Aug., 1994, pp.423-433

[5] Vecchio, F. J., "Disturbed Stress Field Model for Reinforced Concrete: Formulation," J. of Structural Eng., ASCE, Vol.126, No.9, Sept., 2000, pp.1070-1077

[6] Vecchio, F. J., "Disturbed Stress Field Model for Reinforced Concrete: Implementation," J. of Structural Eng., ASCE, Vol.127, No.1, Jan., 2001, pp.12-20

[7] Susetyo, J., "Fibre Reinforcement for Shrinkage Crack Control in Prestressed, Precast Segmental Bridges," Ph.D. Thesis, Dept. of Civil Eng., Univ. of Toronto, 2009

[8] Naaman, A.E., Liao, W.C., Chao, S.H., and Park, S.Y., "Self-Consolidating High Performance Fiber Reinforced Concrete (SCHPFRC) - Preliminary Investigation," Research Report UMCEE 06-02, Univ. of Michigan, Ann Arbor, Dec., 2006

[9] Maekawa, K., Pimanmas, A., and Okamura, H., "Nonlinear Mechanics of Reinforced Concrete, Spoon Press, London, 2003

[10] Hsu, T. T. C., "Discussion of Disturbed Stress Field Model for Reinforced Concrete: Formulation by F. J. Vecchio," J. Structural Eng., Vol.128, No.11, Nov., 2002, pp.1487-1488