Synopsis: The mechanical properties of the cement-aggregate bond are reviewed, with particular reference to the inherent difficulties in determining these properties. The properties that are determined experimentally appear to be largely artifacts of the specimen preparation and the test procedures. In particular, bleeding effects, the roughness of the rock surface, and the heterogeneity of the interfacial region make it very difficult to compare experimental results amongst the different investigations that are found in the literature. It is concluded that we are still far from being able to make useful measurements of the properties of the cement-aggregate interfacial zone. We cannot, therefore, yet try to control the properties of concrete by systematically altering the nature of the interfacial region.

Keywords: Aggregates; bleeding (concrete); bonding; cements; fracture properties; hardness; interface; mechanical properties; specimens; strength; toughness
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INTRODUCTION

It is now commonplace to consider concrete as a material consisting of three phases: the hardened cement paste (hcp), the aggregate, and the interfacial transition zone (ITZ) between the hcp and the aggregate particles. In fact, of course, there are a number of other interfaces within modern concrete, including those between

- i) the various phases in the hcp
- ii) the hcp and anhydrous cement grains
- iii) the hcp and pozzolanic additions or mineral fillers.

All of these interfaces are important in determining the mechanical properties of the concrete. This is particularly the case as we move from normal strength concrete, in which the properties of the hcp largely control the concrete properties, to high strength concrete, in which the composite behaviour of the concrete must be considered.

Clearly, the strength of concrete must depend upon the intrinsic strength of the hcp (and possibly of the aggregate), and upon the strength of the bond between the hcp and the aggregate. Unfortunately, it has so far not been possible to determine, in a meaningful way, the bond strengths between the various phases in concrete, and consequently it has not been possible to quantify the effect of the properties of the ITZ on the properties of concrete.

It is generally assumed that the ITZ is the "weak link" in normal strength concrete. Thus, the focus of this review is on the ITZ, and in particular, on the difficulties that arise when trying to measure its mechanical properties. Unfortunately, there are no test methods specified in any national standards dealing with the properties (or even the extent) of the ITZ, and so one is left with trying to draw conclusions from the disparate
collection of experimental studies that have been reported in the literature. Since there has been virtually no work at all on the other interfaces mentioned above, they will not be discussed in this review.

**HOW DO THE PROPERTIES OF THE ITZ AFFECT THE MECHANICAL PROPERTIES OF CONCRETE?**

For the purposes of this review, the precise nature of the composition and morphology of the ITZ is not very important; what we know of this zone has, in any event, been reviewed extensively in recent years (e.g. 1-9). It is sufficient to note that the thickness of the ITZ is generally taken to be about 50 μm, with the major differences from the bulk hcp occurring within about the first 20 μm from the physical interface. We must also recognize the extent of the ITZ. While it is only 50 μm thick, Diamond et al. (10) have shown that, in normal concrete, the average minimum spacing between adjacent aggregate particles is only about 75-100 μm. Thus, a relatively large proportion of the hcp lies within the ITZ.

While it is generally accepted that increasing the cement-aggregate bond strength will lead to an increase in the concrete strength, the experimental evidence for this is not consistent. A number of studies have shown that, in going from "no bond" to "perfect bond" (insofar as either case can be simulated experimentally) strength increases have been in the range of 15% to 40%, with increases in tensile or flexural strength being higher than the increases in compressive strength. For instance, Alexander and Toplin (11,12), based on a regression analysis of the data then available to them, developed a relationship of the form

\[ \sigma = b_0 + b_1 m_1 + b_2 m_2 \]

where

- \( \sigma \) = concrete strength (compression or flexure)
- \( b_0, b_1, b_2 \) = linear regression coefficients
- \( m_1, m_2 \) = flexural strength of the paste and of the cement-aggregate bond, respectively,

From these regression coefficients, it may be seen that a change in the strength of the paste has about twice as much effect in the concrete strength as does a change in the bond strength. Similar results were reported by Darwin and Slate (13).
Other investigations, however, have yielded different results. For instance, Chen and Wang (14) showed that an improvement in cement-aggregate bond strength would significantly increase the tensile strength, and to a lesser extent the compressive strength, of concrete. Similar results were obtained by Wu et al. (15) and Wu and Zhan (16). One can only assume that the differences between these results and those referred to above (11-13) are due to differences in test techniques.

Results of numerical simulations (17) have also shown that changing the bond strength should increase the tensile and flexural strengths, but should have little effect on the compressive strength. On the other hand, a numerical study by Schlangen and van Mier (18) showed that "the specimen strength is mainly governed by the matrix strength". Thus, "numerical" concrete appears to be at least as inconsistent with regard to bond strength as does "real" concrete!

**RELATIONSHIP BETWEEN SPECIMEN PREPARATION AND MEASUREMENT OF INTERFACIAL MECHANICAL PROPERTIES**

The inconsistencies in the effects of bond strength on the mechanical properties of concrete appear to be due to inconsistencies in the measurements of the properties of the ITZ or of the cement-aggregate bond. There are a number of reasons for this:

1. It is not possible to make direct tests of the strength of the material in the ITZ, because it is not possible to isolate this material, which is only about 50 μm thick. Moreover, the ITZ is itself variable in its composition, and thus its mechanical properties must also vary from point to point, as shown by the microhardness data that have been reported (19-20). These data indicate that the weakest part of the ITZ lies about 20 μm from the aggregate surface.

2. Because of bleeding effects, water tends to be trapped at the undersides of large aggregate particles. Thus, the ITZ will vary in the general case, from the top to the bottom of a coarse aggregate particle.

3. The bond between the aggregate and the cement must depend upon the roughness of the aggregate surface. Different investigations have prepared the aggregate surface in various ways, ranging from a "mirror-like" finish (21) to the rather rough fracture surface of rock (22). These different surface finishes will clearly lead to very
different measurements of bond, and perhaps to different morphologies of the ITZ.

4. Most studies of the ITZ involve specimens specially prepared by casting cement against a rock surface. However, such interfaces are quite different from those which occur in conventional concrete, where there is relative movement between the cement grains and the aggregate particles during mixing.

5. As has been shown clearly by Odler and Zurz (21), different cement/rock systems exhibit different failure mechanisms, with failure occurring either right at the interface, or at various distances from the interface, depending on the aggregate.

DISCUSSION AND CONCLUSIONS

From these observations, it is clear that we are still unable to measure the properties of the ITZ, or of the cement-aggregate bond, in a reproducible and unambiguous manner. There is no standard test specimen or test method. Indeed, to the author's knowledge virtually no two studies of the transition zone have been truly comparable, because of differences in the test procedure.

Moreover, it is also still not clear whether it is the actual cement-aggregate bond, or the strength and density of the entire 50 μm thick ITZ, which must be modified in order to improve concrete strength. Since it is apparently not possible to alter one without altering the other, this question is perhaps only of philosophical interest, but it does illustrate the complexity of the problem.

It is, of course, well known that densification of the ITZ, generally by the incorporation of fine mineral fillers such as silica fume, can lead to significant increases in concrete strength. However, it would appear that we are still very far from being able to alter the properties of concrete in a predictable way by modifying the properties of the ITZ.

In order for us to determine, in a systematic way, the effects of the interface on concrete properties, we need to develop a standardized test procedure, which would address the following questions:

1. What sort of specimen geometry and type of test do we use?
2. How is the aggregate surface prepared?
3. How are bleeding effects controlled?
4. How are the experimental results analyzed?

As pointed out above, there is still no agreement on answers to these questions. New specimen geometries and test methods are still being developed; of these, the push-out test recently described by Mitsui et al. (23) looks quite promising, but much further work with this and other test procedures is required before a standard can be developed.

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REFERENCES


Softening Slip and Size Effect in Bond Fracture

by Z. P. Bažant and R. Desmorat

**Synopsis:** The size effect caused by post-peak softening in the relation of interface shear stress and slip displacement between a fiber or reinforcing bar and the surrounding matrix, is analyzed. The problem is simplified as one-dimensional. It is shown that the post-peak softening leads to localization of slip. The larger the bar or fiber size, the stronger the localization. The size effect in geometrically similar pullout tests of different sizes is found to represent a transition from the case of no size effect for small sizes to the case of a size effect of the same type as in linear elastic fracture mechanics, in which the difference of the pullout stress in the fiber and the residual pullout stress corresponding to the residual interface shear stress is proportional to the inverse square root of bar or fiber size. An analytical expression for the transitional size effect is obtained and is found to approximately agree with the generalized form of the size effect law proposed earlier by Bažant. Measurements of the size effect can be used for identifying the interface properties.

**Keywords:** Bonding; composite materials; damage; fiber reinforced concretes; fibers (discrete fibers); fracture properties; size effect
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INTRODUCTION

The problem of pullout of fibers or bars from the surrounding matrix has received considerable attention in recent years. Many important results have been achieved; see e.g. Lawrence (1972), Freund (1992), Fuller et al. (1990), Gao et al. (1988), Leung and Li (1990 a,b), Li et al. (1991), Shah and Ouyang (1991), Stang et al. (1990), Steif and Hoysan (1986), Wang et al. (1988), Beaumond and Alezka (1978); Bowling and Groves (1979); and Gray (1984 a,b). Further light on the interface slip has been shed by studies of slip at interfaces of various types, including slip on cracks (e.g. Bažant and Gambarova, 1980; Divakar et al., 1987; Feenstra et al., 1991). An excellent review of the pullout test analysis has recently been presented by Shah and Ouyang (1991). To make analytical solutions feasible, many previous authors have simplified the complex three-dimensional behavior at interface as one-dimensional (e.g. Gao et al., 1988; or Freund, 1992). In the one-dimensional solution, the normal pressure across the interface and the interface dilatancy cannot be taken into account. A more general formulation or some adjustments become necessary; see e.g. Lawrence et
Fig. 1—a) Geometry of fiber pullout tests analyzed; and b) Interface shear stress versus relative displacement

al. (1972), Bowling and Groves (1979), and Hutchinson and Jensen (1990). Stang et al. (1990) considered the stress-slip relation to consist of an elastic part followed by a sudden stress drop and a residual constant friction part. However, it is no doubt more realistic to consider a gradual softening. As for the post-peak softening, it will be considered to be linear (Fig. 1b), in order to make a simple analytical solution feasible.

The size effect in the problem of fiber pullout has not yet been theoretically studied, although its existence has already been demonstrated for the case of bar pullout from concrete (Bažant and Sener, 1988). In the present brief conference presentation, the focus of the analysis will be on the size effect, using an approach with a two-way debonding similar to that of Leung and Li (1990). It will be shown that the post-peak softening in the interface stress-slip inevitably causes a size effect and that this size effect can be exploited for determining the interfacial material properties solely from measurements of maximum pull-out forces. The detailed analysis, ramifications and extensions to other cases will appear in a forthcoming journal article (Bažant and Desmorat, 1994).
SOLUTION OF PULL-OUT FORCE

A cylindrical fiber or bar of diameter $d$ is assumed to be embedded in an outer cylinder of diameter $D$ representing the matrix of a composite material (Fig. 1a). The bar is supported at the points of reaction forces acting on the cylinder, shown in Fig. 1a. The interfacial debonding is characterized by the relationship of interface (bond) shear stress $\tau$ versus relative stress displacement $v$, as shown in Fig. 1b, where $\tau_s =$ initial bond strength (initial cohesion), $\tau_d =$ residual bond stress at sliding interface and $v_0 =$ critical slip, determining the slope of the $\tau(v)$ diagram, which is assumed to be linear. The fiber and matrix are elastic, characterized by elastic moduli $E_f$ and $E_m$. The interface shear stress at the softening portion is

$$\tau = \tau_s \left(1 - \frac{v}{v_0}\right) \quad (1)$$

The cross-hatched area in Fig. 1b represents the bond fracture energy, which is expressed as:

$$G_f = \frac{1}{2} \tau_s v_0 \left(1 - \frac{\tau_d}{\tau_s}\right)^2 \quad (2)$$

Let $z$ be the longitudinal coordinate. The fiber has a free end at $z = -l$.

For the case of softening slip, the following differential equation for the fiber or bar stress can be derived:

$$\frac{d^2\sigma}{dz^2} + \omega^2 \sigma = \frac{\phi}{1 + \phi} \omega^2 \sigma_a \quad (3)$$

in which $\phi = A_f E_f/A_m E_m$, $A_f = \pi d^2$, $A_m = \pi (D^2 - d^2)$ and $\omega^2 = 4(1 + \phi) \tau_s/(E_f v_0 d)$.

At the cross sections with no interface crack, the strains in the fiber and the matrix cylinder are equal, $\sigma/E_f = \sigma_m/E_m$; this yields $\sigma = \sigma_a \phi/(1 + \phi)$; where $\sigma_a =$ applied pull-out stress ($\sigma_a = P/A_f$ where $P$ is the pull-out load).

In the pull-pull case, two interface cracks grow from both ends of the fiber until they join. At that moment, the maximum applied stress $\sigma_a = \sigma_N$, representing the nominal strength, is reached. If the load is controlled, failure occurs at that moment. The solution is simpler when $\phi = 1$, and therefore attention is restricted to this case, although the general conclusions and implications for the size effect are the same for any $\phi$.

Several stages of loading have to be distinguished: (1) The initial stage, in which there are two separate cracks emanating from the ends of the fiber and the shear stress is everywhere larger than the residual strength $\tau_d$. 
(2) The final stage, in which the two cracks have joined into one and the residual strength $\tau_d$ has been reached at both ends. (3) The intermediate stage, in which one must distinguish two cases: (a) The two cracks join before $\tau$ reaches $\tau_d$ at the ends, (b) the shear stress $\tau_d$ is reached before the cracks join.

The solution of the maximum value of $\sigma_a$ yields the following result:

For $\omega L < 2 \arccos \frac{\tau_d}{\tau_s}$:

$$
\sigma_a = \text{Min} \left( \frac{8\tau_s}{\omega d} \sin \frac{\omega L}{2}, \frac{8\tau_s}{\omega d} \sqrt{1 - \left( \frac{\tau_d}{\tau_s} \right)^2} \right)
$$

(4)

Otherwise:

$$
\sigma_a = \frac{8\tau_s}{\omega d} \sqrt{1 - \left( \frac{\tau_d}{\tau_s} \right)^2} + \frac{4\tau_d}{\omega d} \left( \omega L - 2 \arccos \frac{\tau_d}{\tau_s} \right)
$$

(5)

In the view of recent studies of localization, one might question whether symmetric curves can occur at both ends simultaneously. However, localization into one crack cannot occur here because the problem is one dimensional and the load increases during fracture growth (since we are interested only in the solution up to the peak load).

**SIZE EFFECT**

In the mechanics of damage and nonlinear fracture, the problem of size effect has received major attention in recent years. It has been shown that generally the scaling law represents a transition from elasticity (or plasticity), for which the size effect is absent, to linear elastic fracture mechanics, for which the nominal strength is inversely proportional to the square root of the structure size (see Bažant, 1984; Bažant and Cedolin, 1991, Bažant (1993) with further reference and discussion of competing theories). The size effect can be defined only for structures with similar geometries and similar cracks. Therefore, the ratios $D/d$ and $L/d$ are considered as constant and the fiber-diameter $d$ to plays the role of characteristic dimension of the structure (note that, in this case, $\omega d$ and $\omega L$ are both proportional to the $\sqrt{d}$). The applied pullout stress $\sigma_a$ at maximum load may be employed as the nominal strength, $\sigma_N = \sigma_a$.

For the numerical examples, consider material properties $\tau_s = 31$ MPa, $\tau_d = 3$ MPa, $v_0 = 0.021$ mm and $E_f = 200$ GPa. The calculations are made for the sizes $d = 1, 2.9, 6.4$ and $12.7$ mm at constant ratio $L/d = 4$. The results of the calculations are plotted in Fig. 2. It is apparent that the maximum pullout stress decreases with increasing size. Furthermore, the type of the load-displacement diagram changes. For the smallest size,
there is a gradual post-peak softening, for the next size there is a nearly vertical stress drop, and for the largest two sizes, snap back instability right after the peak occurs. This behavior is typical of the size effect in all materials exhibiting damage localization or nonlinear fracture. The size effect is caused by increasing localization of the softening regions along the fiber length as $d$ increases. The softening region at maximum load, which represents the fracture process zone and is characterized by stress values between $\tau_s$ and $\tau_d$ (Fig. 1), extends in small specimens over a large portion of the fiber length and in large specimens over a small portion of the fiber length. This behavior is similar to other failures due to damage growth or nonlinear fracture.

The size effect obtained for our example is shown in Fig. 3 by the diagram of $\log(\sigma_N - \sigma_0)$ vs. $\log d$, where $\sigma_0$ is the residual fiber strength corresponding to the residual interface bond stress $\tau_d$.

Now the asymptotic behavior will be examined. In the limit of small sizes, $d \to 0$,  
$$\sigma_N = 4\tau_s \frac{L}{d} = \text{constant}$$  
(6) 
In the limit of the large sizes, $d \to \infty$,  
$$\sigma_N = \sigma_0 + \frac{8\tau_s E_f v_0}{\sqrt{d}} \left(1 - \frac{\tau_d^2}{\tau_s^2} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s}\right)$$  
(7) 
Here $\sigma_0$ is the residual pullout stress of the fiber when the interface is completely debonded and softened to $\tau_d$,  
$$\sigma_0 = 4\tau_d \frac{L}{d}$$  
(8) 
According to (7), the basic form of the size effect for the large sizes is  
$$\sigma_N - \sigma_0 \propto \frac{1}{\sqrt{d}}$$  
(9)
fracture mechanics. For \( d \ll d_0 \), the solution reduces to \( \sigma_N = \text{constant} \) (no size effect), which is characteristic of elasticity or plasticity. For the intermediate values of size \( d \), the solution describes a gradual transition between these two asymptotic cases. Matching the asymptotes to those calculated for fiber pullout, the simple size effect law in Eq. (11) gives in Fig. 3 the plot shown by the dashed curve \((Bf'_t = 500 \text{ MPa}, d_0 = 4.2 \text{ mm})\).

The presently calculated size effect law may be rewritten for \( \sigma_0 = \tau_d = 0 \) as follows:

\[
\sigma_N = \frac{Bf'_t}{\sqrt{\beta}} \sin \sqrt{\beta} \quad \text{if} \quad \beta \leq \frac{\tau^2}{4}; \quad \sigma_N = \frac{Bf'_t}{\sqrt{\beta}} \quad \text{if} \quad \beta > \frac{\tau^2}{4} \tag{12}
\]

Comparing with the size effect law (10), one can evaluate the additional parameter \( m \) by equating (10) and (12) for \( d = d_0 \) or \( \beta = 1 \). This yields \( m = -(\ln 2)/2\ln(\sin 1) \simeq 2 \). For the value \( m = 2 \) the agreement of (12) with the solution of (10) becomes virtually perfect.

It is well-known that the value of exponent \( m \) is related to the shape of the strain-softening diagram (Bazant, 1985). Striving for the simplest analytical solution possible, one may assume this diagram to be linear (Fig. 1b). It was shown before for tensile fracture that a softening of progressively decreasing slope, with a long tail, yields a more gradual transition in the size effect plot. It may be expected that if Fig. 1b were replaced by such a softening diagram, the calculated size effect could be made to match the dashed curve in Fig. 3, corresponding to the simple size effect law in Eq. (11). It remains to be seen whether the actual behavior of interfaces corresponds to the simple case (as it approximately does for tensile fracture of concrete) or an \( m \)-value different from 1 needs to be used.

**IDENTIFICATION OF INTERFACE FRACTURE CHARACTERISTICS FROM MEASURED SIZE EFFECT**

In the mechanics of tensile fracture, one can use the measured size effect to determine the material fracture characteristics (Bazant, 1987; Bazant and Pfeiffer, 1987; Bazant and Kazemi, 1990). The same must be possible for fiber pullout.

Indeed, after calculating the asymptotes of the size effect plot, the size effect parameters can be identified by matching these asymptote with equations (6) and (7). This yields

\[
Bf'_t = 4(\tau_s - \tau_d) \frac{L}{d}, \quad d_0 = \frac{8\tau_s E_f \rho_0}{(Bf'_t)^2} \left( \sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s} \right)^2 \tag{13}
\]
Except for the presence of $\sigma_0$, this represents the size effect characteristic of linear elastic fracture mechanics. In the plot of Fig. 3, it corresponds to the inclined straight-line asymptote of slope $-1/2$.

The size effect obtained by the present analysis and shown in Fig. 3 agrees with the general size effect of damage mechanics or nonlinear fracture. Introducing the hypothesis that the energy dissipated at failure is a smooth function of both the specimen (or structure) size and the fracture process zone size, with the latter being a material property, it was shown (Bažant, 1985) by dimensional analysis and similitude arguments that, in

$$\sigma_N = B f'_t \left[ \xi (1 + \xi^{-1} + A_1 \xi^{-2} + A_2 \xi^{-3} + \ldots) \right]^{-1/2m} \quad \xi = (d/d_0)^m \quad (10)$$

in which $f'_t$ is the tensile strength of the material, introduced strictly for convenience, and $m, B, d_0, A_1, A_2, \ldots$ are positive empirical coefficients.

For materials with residual strength, represented here by $\tau_d, \sigma_N$ must be replaced with $\sigma_N - \sigma_0$ where $\sigma_0$ is the residual nominal strength. Also, for a smaller size range, the asymptotic series in (10) may be truncated after the linear term. This yields:

$$\sigma_N - \sigma_0 = B f'_t (1 + \beta^m)^{-1/2m} \quad (11)$$

where $\beta = d/d_0 = $ relative size. For $m = 1$ this reduces to the simple approximate size effect law originally proposed in Bažant (1984).

It is obvious that, for $d \gg d_0$, the solution reduces to $\sigma_N - \sigma_0 \propto d^{-1/2}$, which is the form of size effect exhibited by all formulas of linear elastic
For the purpose of analyzing these data, the solution for the pull-push test has been derived;

\[
\text{for } \omega L \leq \arccos \frac{T_d}{\tau_s} : \quad \sigma_N = \frac{4\tau_s}{\omega d} \sin \omega L
\]

(14)

\[
\text{for } \omega L > \arccos \frac{T_d}{\tau_s} : \quad \sigma_N = \sigma_0 + \frac{4\tau_s}{\omega d} \left( \sqrt{1 - \left(\frac{T_d}{\tau_s^2}\right)^2 - \frac{T_d}{\tau_s} \arccos \frac{T_d}{\tau_s}} \right)
\]

(15)
in which \( \omega \) is given by \( \omega^2 = 4(1 + \phi)\tau_s/(E_f v_0 d) \) and \( \sigma_0 = 4\tau_d L/d, \phi = A_f E_f/A_m E_m \).

Knowing the exponent \( m \), which is here taken as \( m = 1 \) (same as Bažant and Sener, 1988), one may use the aforementioned linear regression plot \( Y = AX + C \) to determine the size effect law parameters \( B f_f' \) and \( d_0 \). Then, matching the asymptotes yields the following expressions for the interface properties:

\[
\tau_s = \frac{d}{4L} B f_f' + \tau_d
\]

(16)

\[
v_0 = \frac{1 + \phi (B f_f')^2}{4 \tau_s E_f} d_0 \left( \sqrt{1 - \left(\frac{T_d}{\tau_s^2}\right)^2 - \frac{T_d}{\tau_s} \arccos \frac{T_d}{\tau_s}} \right)^{-2}
\]

(17)

Using the size effect law parameters obtained by Bažant and Sener (1988), one obtains from (16) and (17) the following interface properties: \( \tau_s = 31 \) MPa, \( v_0 = 2.1 \cdot 10^{-2} \) mm, \( G_f = 325 \) J/m\(^2\). The value of \( \tau_d \) has been neglected in these calculations. The optimum fit by the size effect law given by Eq. (11) is shown by the dashed curve in Fig. 4, and the fit based on (14) and (15) is given by the solid curve. Assuming progressively increasing values \( \tau_d = 0, 1, 2, 3 \) MPa, one obtains from (14) and (15) the solid curves shown in Fig. 4 b, c, d. Unfortunately, the scatter of the data is insufficient to decide which of these curves is more correct.

The foregoing data analysis is merely an example, used because more relevant test data pertinent to failure by slip alone (without radial cracks) are lacking.

**CONCLUSIONS**

1. One-dimensional simplification of the fiber (or bar) pullout problems allows a simple analytical solution yielding closed form expressions for the stress-displacement diagram as well as the size effect. The solution shows that, for geometrically similar situations: (1) the maximum pullout stress decreases with increasing size (characterized for example by the fiber diameter), (2) the post-peak slope of the load-deflection diagram becomes steeper as the size increases, and (3) for a sufficiently large size, snapback failure is obtained.
When the size effect law is to be matched to experimental data on $\sigma_N$, parameters of the Eq. (11) can be easily identified by linear regression $Y = AX + C$ where $X = d$, $Y = 1/(\sigma_N - \sigma_0)^2$, and for $m = 1$: $bf' = 1/\sqrt{C}$ and $d_0 = C/A$.

To demonstrate the procedure, we will use the test data of Bažant and Sener (1988) (circles in Fig. 4) for pullout of reinforcing bar from concrete cubes are used, even though the failure mode observed in these tests does not fit the assumptions of the present analysis (the cubes failed by radial splitting cracks emanating from the bar, which were caused by lugs on reinforcing bars and cannot be described by a one-dimensional model). Deformed reinforcing bars of yield strength 414 MPa and diameters 2.9, 6.4 and 12.7 mm were used. In each cube, there was one bar parallel to one edge of the cube and sticking out at the center of one face. The embedment length of the bar was $L = 4d$. The size effect law parameters, identified previously (Bažant and Sener, 1988) were $bf' = 500$ MPa, $d_0 = 2.1$ mm and $\sigma_0 = 0$. 
2. An inevitable consequence of softening in the relation of interfacial shear stress versus slip displacement is localization of the fracture process zone along the interface, with a gradual approach to a mode II-type interface shear fracture. Due to localization, the distribution of the interface shear stress along the fiber or bar becomes strongly nonuniform, and the nonuniformity gets stronger as the size increases. The localization is the cause of size effect.

3. The size effect is transitional between the case of elasticity or plasticity, for which there is no size effect, and the case of linear elastic fracture mechanics, for which the difference of the interface strength and the residual stress is inversely proportional to the square root of the size. This transitional size effect can be described by the approximate size effect law proposed by Bazant (1984) or its subsequent generalization with parameter \( m \) controlling the shape of the size effect curve.

4. Measurements of the size effect in fiber pullout can be exploited for determining the interface properties.

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Fracture and Internal Cracking of High-Strength Concrete under Axial Compression

by L. R. Taerwe

Synopsis: The fracture process of high strength concrete (HSC), submitted to uniaxial compression, is analysed by means of loading tests on cylinders under special closed loop control. Conclusions are drawn from axial and circumferential strain curves, cross-sections of specimens impregnated with a fluorescent dye and visual observations. The evolution of the internal crack extension is revealed and it is shown that under stable progressive fracture, predominantly aggregate bond failure and crack branching occur with the cracks passing around the coarse aggregates. The onset of damage is explained in terms of elementary force transfer concepts. The influence of maximum aggregate size and fiber addition are also discussed.

Keywords: Compression; crack propagation; fibers (discreet fibers); fracture properties; high-strength concretes; polypropylene fibers; steels; strains; stress-strain diagram; tests
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INTRODUCTION

As concrete strength increases, the descending branch of the stress-strain curve gradually becomes steeper and for HSC an almost vertical softening curve is obtained. When no particular precautions are taken during the compression test, a very explosive failure occurs. Typically, the fracture surface crosses the coarse aggregates and a fairly smooth fracture surface results. According to (1), this phenomenon can be attributed to the smaller number of continuous crack paths for high strength concrete, which results in a decrease in the redundancy of the nearly homogeneous material. In this way, the absence of a stable descending branch in the stress-strain curve could be explained.

From the previous observations it follows that HSC is generally considered as a brittle material. This brittleness causes some concern with respect to applications in structural elements. However, HSC beams and slabs show a higher flexural ductility than corresponding normal strength concrete (NSC) specimens. This apparent contradiction between material brittleness and member ductility was discussed in detail by Taerwe (2).

Hence, it is of great practical importance to gain fundamental insights into the fracture process of HSC and the factors that influence it. This can result in appropriate measures to improve material toughness or in rational ways to account for it in design procedures.

TEST SPECIMENS

The loading tests on cylinders discussed in this paper are part of a comprehensive research program that includes normal, medium and high strength concrete, the latter with and without fiber addition. A general survey of the characteristics of the plain high strength concrete cylinders (designated as HS followed by a number) is given in table 1. This series of specimens serves as the basis for the analysis of the fracture process and internal crack propagation. The characteristics of the concretes containing larger aggrega-
Concrete composition is as follows: river gravel 4/14 (1300 kg), river sand 0/5 (550 kg), Portland cement (400 kg), water (120 kg), high-range water-reducer (10 \text{ t}). The water-cement ratio equals 0.30. The concrete shows a mean slump of 200 mm and a mean compressive strength of 93 MPa at an age of 4 months. The specimens were cured at 20°C in a humid environment with relative humidity of at least 90 %. Eight specimens have a diameter of 155 mm, while the other three are 192 mm in diameter. This corresponds to a height/diameter ratio of 2.45 and 2.52 respectively. The end faces of the cylinders were polished prior to testing.

**EXPERIMENTAL PROCEDURE**

At an age of 4 months, the specimens were submitted to axial compression in a stiff closed loop testing machine with a capacity of 6000 kN (axial stiffness about 2500 kN/mm). The loading platens were able to rotate during the test. No steel columns or bars were loaded in parallel with the specimens, as described by Glavind, Stang (4) and others. No special precautions were taken to prevent friction between the polished end faces of the specimens and the loading platens. It is deemed that due to the height/diameter ratio of 2.5 the extent of the disturbed end zones is sufficiently limited as shown in fig. 1. It should be taken into account that this longitudinal section does not represent the volumetric ratio which is smaller than the ratio of the indicated areas. Experimental evidence of this aspect can be found in (5) and (6).

When an inappropriate control signal is used to command the testing machine, a sudden, explosive failure occurs when a high-strength specimen reaches the peak stress. The occurrence of explosive failures is to some extent related to the stiffness of the testing machine relative to the stiffness of the specimen. For HSC this problem is of minor importance as it can generally be overcome by a closed loop system with high speed response. The most important factor that influences the nature of specimen failure is the feedback signal that is used by the closed loop system.

Directly controlling the cross-head movement or the rate of axial deformation does not avoid explosive failure because these quantities show snap-back behaviour as will be discussed further on. Hence, there is a need for a signal that shows a continuous increase in course of time. Shah, Gokoz and Ansari (7) put forward the idea of using the circumferential strain as control signal. Due to longitudinal splitting, the rate of increase of circumferential strain in the descending part is greater than that of axial strain and thus provides a more sensitive control signal.
To measure the average expansion of the specimen, Shah, Gokoz and Ansari (7) developed a special measuring device, consisting of a piano wire wrapped around the cylindrical specimen. The wire was fixed at one end and was attached through a spring to a linear variable differential transformer (LVDT). During loading, the expansion of the specimen caused the free end of the wire to move, producing a proportionate change in the voltage of the LVDT signal. In this way, a feedback signal that steadily increased in course of the loading process was obtained as input for the regulation of the hydraulic system.

In this research program, the same testing principle was adopted. However, instead of a piano wire, a thin strip of spring steel was wrapped around the specimen, its length being somewhat shorter than the cylinder's circumference (fig. 2). Two small steel blocks, fixed at both ends of the strip, were fitted in between the extremities of a "strain-stirrup". This device, in fact a small portal frame, consisted of two legs and a connecting beam element. The section of this small beam was reduced over the central zone to increase its flexibility. In this zone, four electrical strain gauges were applied in full bridge connection. The measured strain signal is proportional to the change in distance between the legs' extremities, which in turn represents the integral of the circumferential strain. The strain-stirrup was kept in place by means of a spring connecting both legs (fig. 2). The device yielded a reliable control signal as it is almost insensitive to friction effects. Moreover, it proved to be extremely robust. The control technique was further improved by taking a combination of axial and circumferential strain as feedback signal. This aspect is discussed in more detail by Glavind and Stang (4).

**STRESS-STRAIN CURVES**

The stress-strain curves of four HSC specimens are shown in fig. 3. The axial strain is defined as $\Delta h/h$ with $h$ the specimen's height and $\Delta h$ the displacement (shortening) measured between the loading platens. Axial compressive stress is normalized with respect to peak stress $f_c$. The specimens were unloaded deliberately at different stress levels along the descending branch to allow investigation of the progression of the fracture process in more detail by means of sections obtained after resin impregnation.

In fact, $\Delta h$ should be plotted as such because no uniform state of deformation exists over the height of the specimen. However, it was deemed reasonable to plot $\Delta h/h$ because in this way numerical values can be interpreted more easily. The axial deformation was also measured by means of three vertical "strain-stirrups" with a gage length of 200 mm, placed in the central part of the height. It was observed that in this way slightly smaller axial deformations were measured compared with the values of $\Delta h/h$, at
least in the ascending branch of the stress-strain curve. After the peak stress, the three measuring devices yield a different signal because of the non-homogeneous distribution of the surface cracks. Thus, the mean of the three signals does not necessarily reflect the overall response of the specimen. It also often happens that the measuring dots fall off the specimen due to extension of cracks, in which case the measuring signal is lost. Hence it was decided to measure the overall response by means of $\Delta h$.

The stress-strain curve of specimen HS11 (fig. 3) exhibits some characteristics typical for the HSC specimens. After the peak stress, the curve shows a fairly steep part followed by a general snap-back. At a stress equal to about $0.75 f_c$, a spontaneous unloading in the $\sigma$-$e$ path occurs, which can only be obtained with the type of control signal used. Axial strain control would not allow this phenomenon, and explosive failure would result. At a stress of about $0.6 f_c$, a kind of intermediate plateau occurs. Subsequently, before an axial strain of about 0.003 is reached, a steep part with slightly positive slope and local snap-backs results again. Below $0.2 f_c$, a more gradual decrease can be seen. At a strain of about 0.0035, the specimen was unloaded deliberately.

In fig. 4 the circumferential strain $\varepsilon_i = \Delta d/d$, with d the specimen’s diameter, is shown as a function of the axial stress. This signal shows a monotonic increase in time and no snap-back behaviour is observed. Hence it yields a suitable control signal. The typical features of the curve will be discussed further on.

**INTERNAL CRACK PATTERN**

After unloading, a selected number of specimens was impregnated with a low viscosity epoxy resin to which a fluorescent dye was added. After hardening of the resin, the specimens were sawn in two halves along a meridian plane. The crack distribution in this plane can clearly be observed under ultraviolet light. The impregnation technique reveals the structure of the continuous internal crack pattern that is connected to outer surface cracks. Internal localized microcracking cannot be detected by this technique.

Specimen HS9 was unloaded in the descending branch after the first lobe with snap-back behaviour (fig. 3). On the outer surface, only slight scaling over about half of the perimeter and at variable height was visible (fig. 5). The impregnated section revealed a small crack in the neighbourhood of a coarse aggregate, extending to a depth of about 15 mm from the surface. The picture with the internal crack pattern cannot be reproduced with sufficient quality and contrast to reveal the details of interest.
Specimen HS10 was unloaded at the small plateau mentioned before, that occurs between 0.5 and 0.6\(f_c\). At each side of the specimen, an inclined branching crack can be observed (fig. 6). This, and the following photographs are reproduced in inversed mode with the cracks showing up as black lines. From the more detailed picture in fig. 7, it can be seen that successive branching of cracks occurs, mainly at coarse aggregates, resulting in a rather extensive finely distributed crack pattern. Further, these saw cuts indicate that fracture lines predominantly pass around coarse aggregates as in normal strength concrete (except for flat shaped and weaker particles), and rarely run through the aggregates, as is observed in the case of explosive failure. Hence, the smoothness of the fracture surface is mainly related to the speed of crack propagation. Van Mier (8) arrived at similar findings for the case of uniaxial tension. He also concluded that stable failure of HSC is mainly caused by debonding of aggregates and matrix.

Specimen HS11 (fig. 8) additionally shows a transverse crack, connecting both damage zones that occur at diagonally opposite locations. This crack points to a kind of shear band which can be observed at the cylinder’s surface.

**FRACTURE PROCESS AND DAMAGE DISTRIBUTION**

In this section, a basic description of the fracture process and the damage distribution is presented. First, attention is drawn to fig. 4, where three different stages can be distinguished in the descending branch. Stage A, with fairly steep descent, corresponds to the snap-back lobe in the axial strain curve. Subsequently, a fairly extensive plateau occurs (stage B), corresponding to the smaller one in the axial strain curves. Finally stage C shows a gradual decrease of the \(\sigma-\varepsilon\) curve.

From the previous observations the following fracture sequence is derived (9)

- The fracture process is initiated by the occurrence of very localized damage at the specimen surface, gradually extending around the major part of the perimeter (stage A).
- Extension of the cracks, mainly in height (longitudinal direction) but also in depth (radial direction), which yields an important expansion of the specimen and causes a small increase in axial shortening due to the more or less intact core (stage B).
- Formation of a shear band, crossing the core of the specimen, connecting opposite damage zones and yielding a finely distributed crack pattern with the coarse aggregates acting as crack side bridges (stage C).
A similar approach is followed in the theoretical model developed by Markeset (10). The fact that in fig. 4, the slope of the curve in stage C is steeper than in stage B, is partly attributed to the fact that the shear band mainly affects the core of the specimen, which has a smaller cross-section. It is suggested that the inflexion point in the softening branch of the stress-strain curve of normal to medium strength concrete can be associated with the transition between two fracture processes i.e. on the one hand the gradual extension of surface cracks over the height and to the core of the specimen and on the other hand shear of the more or less intact core.

ONSET OF DAMAGE PROCESS

Extent of Damage Zone

As mentioned already for specimen HS9, the onset of damage occurs in a very small zone at the outer surface. There is reasonable experimental evidence that this occurs around a coarse aggregate. This is confirmed by specimen HSR5 that is shown in fig. 9 and that belongs to another test series than the one mentioned in table 1. This specimen shows pop-outs at coarse aggregates in the vicinity of the side face.

The fact that the initial damage zone is small, not only holds with respect to the height but also with respect to the cross-section. Due to the small volume of the initial damage zone, snap-back behaviour at the onset of the softening branch can be explained by the elastic extension of the large non-damaged zone that dominates the axial shortening in the damage zone. It is deemed that a one-dimensional series model with softening band, as recently considered for compression by way of extension of the more obvious representation in tension, has to incorporate the extension of damaged zones both in height and cross-section. Hence, it is proposed to consider a volumetric damage parameter defined as

\[ \omega = \frac{V_d}{V} = \frac{(V - V_n)}{V} \]

where \( V \) is the nominal volume, \( V_d \) the volume of the damage zone, \( V_n \) the volume of the non-damaged zone. Formula (1) is an extension of the similar concept used for damaged areas in a cross-section (see e.g. 11).

From observations of crack patterns in cross-sections of test specimens, the following hypothesis is proposed. The height of the cumulated damage zone is roughly equal to the transverse dimension i.e. the diameter in case of a cylinder. Hence, the final volumetric damage parameter \( \omega \) is the same for specimens with the same h/d-ratio. If the hypothesis can be extended to the evolution of \( \omega \) also, then it can be concluded that for the specimens shown in fig. 12, the magnitude of the volumetric damage as defined by (1)
decreases in the following order: C, A (= D) C and B. The approach also allows extension to real structural members (columns, compression zone of beams, ...). It is not necessary that all damage zones are closely connected, although this is mostly the case.

**Damage Initiation**

As pointed out in the previous sub-section, it appears reasonable to state that damage is initiated by the heterogeneity caused by one (or more) of the coarse aggregates. It is often claimed that HSC is a more homogeneous material than NSC and hence is prone to a more brittle behaviour. This is argued by the fact that the mechanical properties of the aggregates and the mortar matrix are less different than is the case for NSC. However, for the type of coarse aggregate used in the research program considered so far, a mean compressive strength of 260 MPa and a modulus of elasticity of 69 GPa were found (12). The difference with respect to the mortar properties still remains reasonably large. Hence we may conclude that the coarse aggregates still act as rigid inclusions in HSC, at least for the strength level considered.

Under this hypothesis it can be shown that damage and cracking initiate at the edge of a specimen by the following simplified model. Consider a regular array of circular inclusions, and a load transfer between the coarse aggregates as indicated in fig. 10. An aggregate at the edge needs a horizontal tensile force for equilibrium whereas for an inward aggregate both tensile forces cancel each other. When, due to bond or matrix failure, a vertical crack occurs, the horizontal equilibrium of the edge aggregate can only be assured by a compression exerted by the outer concrete layer. When this fails, pop-out occurs and a local damage zone initiates.

However, this two-dimensional model is not sufficient for explaining the occurrence of vertical radial cracks. For this purpose, a spatial load transfer mechanism is considered (fig. 11). Consider a cylindrical specimen and three aggregates, two of them being located along the edge of the specimen and in the same horizontal plane. Due to the divergency of the compressive forces that are transmitted from A to B and C, a hoop tension results that causes vertical cracks at the cylinder's curved side face. It follows that consideration of a 3D-model instead of a 2D-model results in an additional contribution to the volumetric damage. This aspect should also be considered in numerical simulations. The previous conclusion is in accordance with Vonk (13), who suggests, on the basis of a two-dimensional simulation: "It is not unlikely that a part of the necessary increase in heterogeneity can be found in extending the model to the third dimension (p. 190)".
Further to the more homogeneous nature of HSC, it was found that less micro-cracks were observed in HSC before peak stress than in similar NSC specimens. This results in a more linear ascending branch of the stress-strain curve. The fact that less bond cracks were found was attributed to a higher bond strength between the aggregates and the mortar matrix. However, when it is assumed that bond strength is directly proportional to tensile strength and as the latter approximately follows a $f_c^{1/2}$-law, it follows that bond strength increases less than proportional with peak stress. Hence, it is suggested that the phenomena that cause micro-cracks do not have the same magnitude in HSC, so that bond strength is not exhausted too early. In the case of silica fume addition, the aggregate-matrix interface is undoubtedly of better quality.

**INFLUENCE OF STEEL FIBER ADDITION**

In fig. 13 the stress-strain curves for specimens HS-SF-3 and HS-SF-4 are compared with the curve of the plain concrete specimen HS-11. Specimen HS-SF-3 has a diameter of 150 mm and a height of 300 mm ($h/d = 2$) whereas HS-SF-4 has a diameter of 192 mm and a height of 483 mm ($h/d = 2.52$). Both these specimens contain hooked, smooth, drawn-wire steel fibers with a length of 60 mm and a diameter of 0.80 mm at a dosage of 40 kg per m$^3$ of concrete.

Comparison with the stress-strain curve of specimen HS-11 reveals that a remarkable increase in toughness can be obtained by adding a moderate amount of steel fibers. A comprehensive series of tests on high strength concrete cylinders with steel fiber addition is discussed by Taerwe (3). He calculates the toughness as the area under the stress-strain curve for the different specimens and the specific toughness as the ratio of the toughness to the compressive strength. It was found that the specific toughness of the fiber reinforced matrix approaches about twice the value of the corresponding plain high-strength control specimen and approaches that of normal strength concrete (cylinder strength of 35 MPa). Analysis of the impregnated specimens with steel fiber addition reveals a more finely distributed internal crack pattern, especially in the shear band.

**INFLUENCE OF POLYPROPYLENE FIBER ADDITION**

Also two series consisting of HSC specimens with polypropylene fiber addition were tested. Two different types of fibers were used. The first type has a cross-section varying between 35 x 250 $\mu$m and 35 x 660 $\mu$m and a length of 12 mm. The second type has a mean diameter of 18 $\mu$m and also a length of 12 mm. In both cases 5 kg fibers were added per m$^3$ of concrete. The two series are designated respectively as HSPF1 and HSPF3. The stress-strain curve of two typical specimens is shown in fig. 14. Comparison
with the curve of the reference specimen HS-11 shows that the first snap-back lobe is slightly reduced and that the toughness of the HSC can be slightly improved by polypropylene fiber addition.

**INFLUENCE OF MAXIMUM AGGREGATE SIZE**

Another series of HSC cylinders was manufactured, this time with river gravel having a maximum size of 28 mm instead of 14 mm. The content of the different constituents remained the same as mentioned in the section on "Test Specimens". The analysis of the test results performed so far, did not reveal a significant influence of the maximum aggregate size on the shape of the strain-softening branch and the internal crack pattern. However, the compressive strength of the concrete with the larger aggregates was reduced by 10%. This trend is in agreement with results presented by other researchers as e.g. Hughes and Chapman (14), who were dealing with normal strength concrete.

**CONCLUSIONS**

- Stable testing of the softening branch of HSC is possible by making use of the circumferential strain as part of the command signal for the closed loop testing. For the purpose of this research program a robust and reliable device was developed.
- The stress-axial strain curve of HSC shows snap-back behaviour.
- The circumferential strain shows a continuous increase as a function of time.
- In the case of stable progressive fracture, the internal cracks mainly pass around the coarse aggregates and debonding and branching are the main phenomena that determine crack extension. For concrete containing river gravel as coarse aggregate, the fracture aspect is very similar to NSC.
- The first stage of the fracture process is characterized by an extension of cracks both in height and in cross-section of the specimen. In the final stage a shear band type of failure occurs.
- The onset of damage occurs at coarse aggregates, located in the vicinity of the specimen's outer surface, that act as rigid inclusions. Both a two dimensional and a three-dimensional force transfer model explain certain damage aspects that are observed in experiments.
- Damage should be characterized by a volumetric damage parameter that tends to a final value which is closely related to the ratio $d/h$, as it is proposed to take the height of the damage zone equal to the transverse dimension of the specimen.
- The addition of moderate amounts of steel fibers (40-60 kg/m$^3$) significantly improves the toughness of HSC. Also the addition of polypropylene fibers (5 kg/m$^3$) results in a slightly improved toughness.


<table>
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<th>Designation</th>
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<th>Height, mm</th>
<th>Compressive strength, MPa</th>
<th>ε₁* per thousand</th>
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*ε₁ is the axial strain at peak stress
ACKNOWLEDGMENT

This research project was partly financed by the Belgian National Fund for Basic Research (grant 1992/1993) whose support is greatly acknowledged.

REFERENCES


Fig. 1—Disturbed zones in specimens with $h/d = 2.5$

Fig. 2—Strain stirrup for measurement of circumferential strain
Fig. 3—Stress-axial strain curves of four specimens

Fig. 4—Circumferential strain for specimen HS11
Fig. 5—Specimen HS9 after loading
Fig. 6—Cross-section of specimen HS10

Fig. 7—Detail of upper left corner of Fig. 6
Fig. 8—Specimen HS11

Fig. 9—Specimen HSR5
Fig. 10—(a) Force transfer in regular array of granular inclusions; (b) Aggregate at edge of specimen before interface or matrix crack; and (c) Aggregate at edge of specimen after interface or matrix crack.

Fig. 11—(a) Side view; (b) Vertical cross-section; and (c) Horizontal cross-section.
Fig. 12—Characterization of size effect by $h/d$ ratio

Fig. 13—Influence of steel fiber addition on stress-strain curve of HSC
Fig. 14—Influence of polypropylene fiber addition on stress-strain curve of HSC
Experimental Studies and Modeling of the Concrete/Rock Interface

by J. Wang and A. K. Maji

Synopsis

This study of the concrete/rock interface addresses primarily the interface of limestone and mortar (since no coarse aggregate was used in the mix), and to a lesser extent, mortar and rock-salt. Uniaxial tensile tests with closed-loop-control were used to determine the stress-crack opening displacement relationship in the softening regime. This relationship is being proposed as the constitutive property in an interface cohesive zone model developed for interface fracture. The validity of such a model was investigated through testing and Finite Element (FEM) analysis of Compact Tension (CT) specimens.

A theoretical investigation of the effect of the complex singularity attributed to an interface crack was performed within the framework of the interface cohesive zone model. Although the theoretical analyses included only a semi-infinite geometry, and was therefore limited in scope, it was found capable of addressing many of the characteristics of quasi-brittle fracture.

Experimental tools used involved a Scanning Electron Microscope (SEM) to observe microscopic features of the interface that are responsible for strength and toughness. The Electronic Speckle Pattern Interferometry (ESPI) technique was used to evaluate pre-peak crack growth. Results indicate that the mechanisms responsible for strength and toughness at the interface are different, and that the characteristics of the fracture at the interface is qualitatively similar to that of any other quasi-brittle material.

Keywords: Finite element method; fracture mechanics; interface; microstructure; models; scanning electron microscopy; stresses; tension
Biographical Sketch

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INTRODUCTION

Interface fracture was first investigated by Williams (1959). Since then, interface crack problems, in view of their importance in numerous applications, have received considerable attention in the literature devoted to fracture mechanics (Rice and Sih, 1965; Shih and Asaro, 1988; and He and Hutchinson, 1989). Linear elastic interface fracture mechanics provides a practical approach to the characterization of interface fracture. This theory has been used to study fracture resistance of interfaces such as metal/ceramic interface (Rice et al., 1989) and epoxy/glass interface (Liechti and Chai, 1991). This theory has only recently been used to study the fracture properties of mortar/rock interface (Buyukozturk and Lee, 1992). It is well known that the interface is critical to the strength development in concrete. In recent years, the mortar/rock-salt interface has also been of interest, for the burial of nuclear wastes at the WIPP (Waste Isolation Pilot Plant) site, near Carlsbad, New Mexico.

The interfaces between mortar and rocks exhibit quasi-brittle behavior, similar to the parent materials, mortar, and rocks. Microcracking and bridging (in the so called fracture process zone) are major mechanisms in the fracture of quasi-brittle materials. The bridging zone can be modeled by the extended part of a macrocrack across which bridging stresses are transmitted and its behavior is characterized by the stress-crack opening displacement relation. This can be obtained from the post-peak tension softening curve of a uniaxial tension test. In the fracture of a concrete/rock interface, the interface itself is much weaker and less tough than the parent materials; debonding and cracking at the interface occur before any microcrack zone is formed in the parent materials (Shah and Chandra, 1968). This phenomenon has been verified with SEM observations of interface Compact Tension (CT) specimens (Maji and Wang, 1992a). Therefore, bridging is the governing mechanism in the interface fracture process zone, and the concept of an interface cohesive zone model seems appropriate and sufficient as a nonlinear fracture model for characterizing the mortar/rock interface.

The concept of a bridging stress model (or cohesive zone model) was originally proposed by Dugdale (1960) and Barenblatt (1962), and was further developed into a fictitious crack model by Hillerborg (1976). Now this model is one of the most commonly used nonlinear fracture models in the study of concrete fracture (Bazant 1992). In this research we combined the experimentally obtained stress-crack opening displacement relationship with the interface cohesive zone model, and used it to analyze the fracture of a CT specimen.

Later, in this paper we report a theoretical analysis of a semi-infinite interface crack with the interface cohesive zone model. This analysis is to illustrate the qualitative similarities between interface fracture, and fracture mechanics of a single material.
Interface Fracture Mechanics

Unlike a crack in a single material, the nature of the stresses at the tip of an interface crack is considerably more complicated (Williams 1959):

\[
\sigma = r^{-1/2} \left( \frac{\sin \alpha}{\cos \alpha} \right) \lambda \log r
\]

where \( \sigma \) is the near-tip stress, \( \lambda \) is a constant, and \( r \) is the distance from the crack-tip. While the stresses remain inverse square root singular, a sharp oscillatory behavior is also present very close to the crack-tip due to the sine or cosine term involving \( \log r \). It was also predicted in Williams' paper, that the mode I and mode II Stress Intensity Factors (SIF) are coupled, i.e., pure tensile loading would result in shear or mode II type stresses at the crack tip, due to the material mismatch across the interface.

Another complexity of the oscillatory stresses is the resulting overlap of the crack faces. Although this is not of concern for an open crack (such as a notch), it is pertinent to a real crack, where the two crack surfaces are initially in contact. Comninou (1977) addressed this anomaly, and demonstrated that when the interpenetration of the crack faces is avoided, \( K_I = 0 \), and only \( K_{II} \) is present at the crack-tip. Therefore, the propagation of an interface crack in tension is intimately connected to failure in shear. Such strong dependence of interfacial fracture toughness on the mode mixity has also been observed experimentally (Cao and Evans, 1989). For materials with similar elastic properties, however, these effects are negligible (Rice, 1988).

Research Significance

Maji and Wang (1992a) have also shown the interface 'contact zone' effects to be negligible through quantitative measurements pertinent to the limestone/mortar interface. The purpose of this research was to probe the applicability of fracture mechanics based on a cohesive zone model rather than focusing on the anomalies of interface fracture mechanics at the crack-tip. In addition, the analysis of a semi-infinite interface crack has been addressed later, to further illustrate the qualitative similarities with the conventional cohesive zone model used for either mortar or rocks.

EXPERIMENTAL PROGRAM

**Direct Tensile Test**

Tensile softening is a characteristic of quasi-brittle materials. The tensile softening behavior of plain concrete under uniaxial tension has been investigated by Gopalaratnam and Shah (1985). In this study, an experiment was designed to study the tensile softening behavior of the mortar/limestone interface. One advantage of having an interface specimen is that no prefabricated notch is needed, and hence, the stresses are truly uniform. Since the interface is the weakest link, controlling the CMOD at the interface is sufficient for a stable test.
Specimen Preparation

The mortar/limestone interface specimens of dimensions 76.2 x 25.4 x 304.8 mm (3 x 1 x 12 in.) were selected for uniaxial tension test. One half of the interface prism specimens was made of Indiana limestone. Limestone prisms of dimensions 76.2 x 25.4 x 152.4 mm (3 x 1 x 6 in.) were cut from a large block of Indiana limestone. The other half of the specimen was cast-in-place cement mortar. The mix-ratio of the cement mortar by weight was 2:1:0.46 (sand:cement:water). Type III cement was used. The batched mortar ingredients were put into a mixer drum, and were mixed thoroughly until it was uniform in appearance, with all ingredients evenly distributed. The limestone prism was cleaned and moistened by clear water, and put into a greased plexiglass mold against one end. The mixed fresh mortar paste was placed into the mold in three layers, each layer with proper consolidation on a shaking table. The cast specimen was removed from the mold after 24 hours, and placed into a lime-water tank for 48 hours. They were tested 3 days after casting. The effect of the curing process on the limestone was not separately investigated, however, no physical changes were noticable.

Test Procedure

Special frictional loading grips were designed for the uniaxial tension test (Figure 1), based on the experiments of Gopalaratnam and Shah (1985). The loading grips consist of 4 pieces of steel plates of dimension 152.4 x 152.4 x 19 mm (6 x 6 x 0.75 in.). In order to uniformly transfer load through friction from the loading grips to the specimen, a 3.2 mm (1/8 in.) thick layer of rubber was glued on the inner face of the steel plates. To minimize the effect of eccentricity, the test specimens were properly aligned with the steel plates and the loading grips. The steel plates were held together by screw bolts which applied a moderate amount of pressure (determined through trial and error) on the specimen surface.

The specimens were tested on an INSTRON testing machine (Model 1323). In order to obtain stable post-peak response, the opening of the interface crack on one side of the specimen was measured by a MTS extensometer (Model 632.03B-30). This elongation measured using a gage length of 15 mm (0.6 in.) was used as a feedback signal to the closed-loop-controller. The output signals from the load-cell and the extensometer were acquired and stored in a PC computer, with an ISAAC 2000 data acquisition system (Figure 2). Three such tests were conducted.

Uniaxial Test Results

The tensile strength of the mortar/limestone interface obtained from the tests was about 1.52 MPa (220 psi). A typical test result is shown in Figure 3. The elastic modulus of a 15mm (extensometer gage length) wide zone near the interface was measured from the tangent of the load vs. extensometer data. The measured average value of this initial tangent modulus near the interface was about 20.7 GPa (= 3000 ksi), which was less than the mean value of the Young's moduli of the two parent materials, limestone and mortar (31.0 GPa and 22.1 GPa). The stress versus interface crack opening curve (Figure 4) was obtained from the unloading lines of the stress-deformation curves. The fracture energy of the tested mortar/limestone interface, obtained from the area under the average stress versus interface crack opening curve, was calculated at a interface crack opening displacement of 0.013 mm (0.51 x 10^-3 in.). As indicated in Fig. 4, the bridging stress of the interface is about 5% of the tensile strength when the interface crack opening reached the value of 0.013 mm (0.51 x 10^-3 in.). Contribution from the area beyond that limit is small (<10%). The average value of fracture energy of the interface was approximately calculated to be 0.00975 N/mm (0.055 lb/in.).
CT SPECIMEN AND ESPI OBSERVATIONS

Specimen Preparation and Testing Procedure

To verify the interface cohesive zone model on an interface specimen with finite geometry, Compact Tension (CT) specimens were tested. The configuration of the tested CT specimen is shown in Figure 5. One half of the interface specimens constituted of limestone, which was half of a fractured limestone CT specimen of the same size. The other half was cast-in-place cement mortar as reported earlier. The 51 mm (2 in.) long notch was cast along the interface with a 1 mm thick steel plate. The steel plate was removed 6 hours after casting, once the mortar had undergone initial set. Two 12.7 mm (0.5 in.) diameter loading holes were drilled into the specimen with diamond core drills. The 3-day curing process for these interface CT specimens was identical to that of the direct tension specimens discussed earlier. They were subsequently tested on a manual loading machine, which was installed on an optical table, to facilitate application of the ESPI technique. A load-cell was installed, and an extensometer was used to measure the Crack Mouth Opening Displacement (CMOD).

Electronic Speckle Pattern Interferometry (ESPI) technique was used to monitor the development of the fracture process zone, the initiation and propagation of the crack at the interface. ESPI is a laser speckle interferometry technique, which uses the interference of the two laser beams to create fringe patterns corresponding to the displacements on the specimen surface. The technique is sensitive to the fraction of a micrometer change in the displacement field. Specimen deformation and propagation of the crack are recorded as changes in the fringe patterns, which can be observed on a video screen in real time. The crack formation can be identified as the discontinuities in the fringe pattern. Details of the ESPI technique has been reported elsewhere (Maji and Wang 1992b).

CT Test Results

The relation between load and Crack Mouth Opening Displacement (CMOD) for an interface CT specimen is shown in Figure 6. The specimens always failed along the interface. The average failure load of the interface CT specimens was about 670 N (150 lbs). Based on the ultimate load and the initial crack length, the critical strain energy release rate for the interface was computed to be 0.00455 N/mm (0.026 lb/in), compared to the energy release rates of 0.0158 N/mm (0.09 lb/in) and 0.0191 N/mm (0.109 lb/in) for mortar and Indiana limestone respectively (Maji and Wang 1992). Hence, this estimate can not be compared to the area under the bridging stress-crack opening displacement curve.

The pre-peak crack propagation along the interface could be readily observed by the ESPI technique as the initiation and development of the discontinued fringe pattern (Figure 7). The pre-peak crack propagation was obtained to be about 50 mm.
FEM ANALYSIS OF THE CT SPECIMEN

The tested interface CT specimen has been analyzed by a nonlinear finite element (FEM) code developed by Gerstle and Xie (1992). The crack propagation at the mortar/limestone interface was characterized by the interface cohesive zone model. The nonlinear behavior of the interface fracture process zone was modeled by a special type of interface element. The constitutive relation of the interface element is characterized by the normal bridging stress versus interface crack opening displacement curve, which is considered as a material property. The normal bridging stress versus interface crack opening curve were obtained through the uniaxial tension test described earlier (Figure 4). To simplify our analyses, the experimental strain-softening curve was simplified into a linear softening model as shown in Figure 4, with tensile strength $f_L = 1.5$ MPa (220 psi), and the critical interface crack opening displacement $\Delta_{COD_c} = 0.013$ mm (0.0005 in.). This $\Delta_{COD_c}$ was measured at the crack 'mouth', and can be also designated as the $\Delta_{CMOD_c}$. The Young's moduli of mortar and Indiana limestone were measured to be 22.1 GPa (3200 ksi) and 31.0 GPa (4500 ksi) respectively. The Poisson's ratio of 0.2 has been used for both mortar and limestone.

FEM Analysis Results

The deformed FEM mesh and interface crack opening at the peak load ($P=698$ N, or 157 lbs) is shown in Figure 8. A comparison of the measured load-CMOD curve with that predicted by the nonlinear FEM model is given in Figure 6. It can be seen that the result from the FEM- interface cohesive zone model is in reasonable agreement with that obtained from the experiment, with regards to pre-peak nonlinearity, and post-peak strain softening.

The progressive extension of the crack and the crack extension at the peak load (Figure 7) were continuously observed by ESPI. These were also in good agreement with the predictions of crack extension as shown in Figure 9. The distribution of the bridging stress along the interface at the different loading stages is shown in this Figure.

MICROSTRUCTURE OF THE INTERFACE

In order to understand the physical nature of the bonding mechanisms in the interfaces, Scanning Electron Microscope (SEM) studies were conducted on two types of interface specimens, mortar/limestone interface specimens and mortar/rock-salt interface specimens.

Specimen Preparation and Test Procedure

A piece of limestone of dimensions 25.4 x 25.4 x 12.7 mm (1 x 1 x 0.5 in.) was cleaned, moistened and put into a greased plexiglas mold of dimensions 50.8 x 25.4 x 25.4 mm (2 x 1 x 1 in.) against one side. The rest of the mold was filled with fresh mortar, using the same mix-ratio reported earlier. The hardened interface prism was cut into SEM specimens of dimensions 25.4 x 25.4 x 6 mm (1 x 1 x 0.25 in.) by a diamond saw. After cutting, the surfaces of the interface specimens were cleaned by an ultrasonic cleaner to wash out the dust on the surface.

The same type of mortar was used in the mortar/rock-salt interface specimens. The preparation procedure was similar to that described above, except that the hardened interface prism was cut into SEM specimens by a chop saw without the presence of water, to prevent the salt from dissolving. The prepared interface specimens were then examined with the SEM.
Microstructure Test Results

The penetration of the cement paste into limestone was found close to the interface. Cement hydration products can lie as deep as 30 μm in the crevices of the limestone (Figure 10). This creates a mechanical interlock between mortar and limestone, and is an important contributor to the bond strength at the interface.

Other microstructure studies have shown that chemical interaction between cement paste and limestone can occur at the interface. Mehta and Monteiro (1987) have discussed the chemistry of this interface in great details. New hydration products which formed only at the interface were observed, but not definitely identified in our study. Highly oriented well arranged crystals of calcium hydroxide were found along the interfaces. The bonding between these calcium hydroxide crystals is weaker and susceptible to cracking (Figure 11).

Shrinkage cracks with width of 0.1-2 μm were often found in some regions of the interface (Figure 12). Loose structures, due to ettringite and large crystals of calcium hydroxide, were evident. Open morphologies, such as porosity and microcracking, were found densely distributed in the region of the interface (Figure 13). This loose structure and open morphology region near the interface constitutes the interface transition zone. This also explains the reduced elastic modulus of the interface as compared to the parent materials. The width of the transition zone was observed to be 20-50 μm. This transition zone is not always obvious; the microstructure in some interface regions looks the same as that far away from the interface. Similar observations were reported by Shah et al. (1992), although the interface zone reported by them was somewhat larger (60-100 μm).

Similarly studies on microstructure of the mortar/rock-salt interface were also conducted. It was found that the inter-penetration and mechanical interlock also exist for this interface (Figure 14). Chemical reactions between mortar and rock-salt were not evident. Loose connections between hardened mortar and rock-salt crystals can be observed in Fig. 15, leading to preferential cracking at the interface.

INTERFACE COHESIVE ZONE MODEL

Modeling of the Interfaces of Strain Softening Materials

The following analysis was done to illustrate the dissimilarities in the theory, and the similarities in the results, between an interface crack, and a regular crack in a homogeneous material. A semi-infinite interface crack has been loaded in tension, and studied with the interface cohesive zone model.

A material with elastic properties $E_1$ and $v_1$ occupies the upper half-plane, $y > 0$, and a material with elastic properties $E_2$ and $v_2$ occupies the lower half plane, $y < 0$ (Figure 16). The two different materials are jointed along the semi-infinite plane along the positive x-axis. A line crack is situated along the negative x-axis extending from $x=0$ to $x=-\infty$, as shown in Figure 16. In the following analysis, all the quantities such as elastic constants, stresses, displacements etc., pertaining to the region $y > 0$ and $y < 0$ will be marked with subscripts 1 and 2, respectively.

In this mortar-rock interface model, the following assumptions are used:

1. The interface is the weaker than the parent materials, and a crack bridging zone exists along the interface after crack propagation (Figure 17). The two bonded materials are purely elastic, and the effect of microcracking is nonexistent.
2. The normal crack opening displacement and the bridging normal stress in the interface fracture process zone satisfies the tension softening relation:
\[
\frac{\sigma}{\sigma_{cr}} = 1 - \frac{w}{w_{cr}} \tag{2}
\]
as shown in Figure 4, where \(\sigma_{cr}\) and \(w_{cr}\) are the critical tensile stress and the critical crack opening, respectively. The shear crack opening displacement in the process zone is assumed to be zero.

3. As the interface is loaded by the applied complex stress intensity factor and the bridging stress, the stresses at the end of the bridging zone (i.e. at the developed crack tip) is bounded. This requires

\[
K_c = K_{ac} + K_{bc} = 0 \tag{3}
\]
where \(K_{ac}\) is the applied complex stress intensity factor, and \(K_{bc}\) is the complex stress intensity factor produced by the bridging stress.

Basic Equations

Let us consider the problem of an interface crack with a bridging zone as shown in Figure 18a. A solution which satisfies the conditions of Equation (2) and Equation (3) must be obtained for a given value of the applied complex stress intensity factor or for a given value of the bridging zone length.

The problem described above can be divided into two parts, as shown in Figure 18b and Figure 18c. The first part constitutes the plate with the same configuration of interface crack as stated above, and loaded only by the applied complex stress intensity factor, \(K_{ac}\) (Figure 18c). The near tip interface crack opening displacement field is given by Rice (1988) as:

\[
\Delta u_a = (\Delta v + i \Delta u)_a = \frac{c_1 + c_2}{2\sqrt{2\pi}} \frac{K_{ac} r^{1/2+i\varepsilon}}{(1+2i\varepsilon) \cosh(\pi\varepsilon)} \tag{4}
\]
where \(\Delta v\) and \(\Delta u\) are the normal and shear interface crack opening displacements respectively. The subscript 'a' indicates that the displacement is generated by the applied complex stress intensity factor, \(c_j = (1 + k_j)/\mu_j\), with \(k_j = (3 - 4v_j)\) for plane strain \(k_j = (3 - v_j)/(1 + v_j)\) and \(\mu_j\) is the shear modulus \((j=1,2)\), \(r\) is the distance from the crack tip, and \(\varepsilon\) is the bimaterial elastic constant defined as:

\[
\varepsilon = \frac{1}{2\pi} \ln\left\{ \frac{k_1/\mu_1 + 1/\mu_2}{k_2/\mu_2 + 1/\mu_1} \right\} \tag{5}
\]

In the second part, the interface crack is loaded only by the bridging stress transmitted between the upper and lower cracked surfaces (Figure 18c). In this case, the complex SIFs and the interface crack opening displacement are written as:

\[
K_{bc} = (K_1 + i K_2)_b = \int_0^{l_b} K(a) T(a) \, da \tag{6}
\]
\[ \Delta u_b = (\Delta v + i \Delta u)_b = \int_0^{l_b} g(a,r) T(a) \, da \]  \hspace{1cm} (7)

where \( K(a) \) and \( g(a,r) \) are, respectively, the complex SIF and the interface crack opening displacement at a distance \( r \) from the crack tip, due to two equal and opposite concentrated forces \( R = P + i Q = 1 \), at \( z = -a \). \( K_1 \) and \( K_2 \) are components of mode 1 and mode 2 stress intensity factors defined by Hutchinson et al. \( T(a) = \sigma_b + iT_b \) is bridging traction along the interface. \( l_b \) is the length of the bridging zone.

\( K(a) \) is given by Rice and Sih (1965) as:

\[ K(a) = \sqrt{\frac{2}{\pi a}} \cos(h \pi \varepsilon) a^{-i \varepsilon} \] \hspace{1cm} (8)

The derivation of \( g(a,r) \) is given in the Appendix, and can be expressed as

\[ g(a,r) = \frac{c_1 + c_2}{4 \pi} \left( \frac{r}{a - x} \right)^{1/2 - i \varepsilon} \int_0^r \frac{\sinh(\pi x)}{\sinh(\pi x/2)} \, dx \] \hspace{1cm} (9)

The superimposition of the two subproblems should satisfy the conditions given by Equations (2) and (3), i.e.

\[ \int_0^{l_b} \sqrt{\frac{2}{\pi a}} \cos(h \pi \varepsilon) a^{-i \varepsilon} (\sigma_b + iT_b) \, da = K_{ac} \] \hspace{1cm} (10)

and

\[ \frac{c_1 + c_2}{2 \sqrt{2 \pi (1 + 2i \varepsilon) \cos(h \pi \varepsilon)}} K_{ac} \left( \frac{r}{a} \right)^{1/2 + i \varepsilon} - \int_0^{l_b} g(a,r) (\sigma_b + iT_b) \, da = w_{cr} (1 - \frac{\sigma(r)}{\sigma_{cr}}) + i0 \] \hspace{1cm} (11)

Equations (10) and (11) give the basic equations for this interface bridging model. By solving the equations, the distribution of the bridging stress in the bridging zone and the bridging zone length can be obtained for a given complex stress intensity factor. With assumed properties of the bridging zone, the critical bridging length and the critical complex stress intensity factor can be determined. On the other hand, for a given bridging zone length the applied complex stress intensity factor can be obtained. The corresponding interface crack resistance can be computed by

\[ R = (c_1 + c_2) \frac{K \bar{K}}{16 \cosh(h \pi \varepsilon)} \] \hspace{1cm} (12)

where \( K \) is the complex SIF described earlier, and \( \bar{K} \) is its complex conjugate.

The algorithm described above was implemented in a Fortran program on a personal computer.
Results and Analysis

As an application example of the developed Interface Crack Bridging Model, the mortar/limestone interface crack was analyzed. The strain softening parameters of mortar/limestone interface from the direct tension test were used. The critical tensile stress value of 1.52 MPa (220 psi) and the critical crack opening value of 0.013 mm were used in the calculation. The critical complex stress intensity factor for the interface was determined as $K = K_1 + iK_\Pi = 0.588 + i 0.0091 \text{ MPa} \cdot \text{m}^{\frac{1}{2}} = (535.16 + i 8.34 \text{ psi} \cdot \text{in}^{\frac{1}{2}})$. The critical bridging zone length was about 106.7 mm (4.2”). Figure 19 shows the crack opening profiles as the interface crack propagates. This result is qualitatively similar to crack profiles that we obtained experimentally for CT specimens of limestone (Maji and Wang 1992b). Figure 20 depicts the R-curve behavior predicted for an interface crack growth. Although these results pertain to a semi-infinite crack, and therefore could not be validated quantitatively with an experiment, they illustrate the similarities between interface fracture and fracture in a homogeneous material.

CONCLUSIONS

Uniaxial tension tests were conducted on interface specimens to obtain the bridging stress versus crack opening curve. The microstructure of the mortar/limestone interface and the mortar/rock-salt interface were studied using a SEM. Results show that both mechanical interlocks and chemical interaction contribute to the interface bonding between mortar and limestone, while only mechanical interlocking is likely responsible for the bond between mortar and rock-salt.

Fracture tests of mortar/limestone interface CT specimens were conducted, and the energy release rates were estimated, as reported earlier. The interface cohesive zone model was used to analyze the CT specimen with a FEM program incorporating special interface elements; results were in good agreement with the experiments.

A theoretical analysis demonstrated the similarities between interface fracture mechanics and conventional fracture mechanics.

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REFERENCES


Fig. 1—Test specimen and loading grips

Load frame assembly

Load cell

Clip gage

Function generation

Load signal

Crack opening signal feedback

Crack opening signal

Command

Closed-loop-controller

Hydraulic actuator

Command

Hydraulic & pump system controller

Data acquisition system

IBM PC

Data transfer

ISAAC 2000

Fig. 2—Schematic of experimental set-up
Fig. 3—Stress-deformation curves obtained from direct tension tests

Fig. 4—Relation between stress and interface crack opening after peak load

Fig. 5—Compact-tension specimen
Fig. 6—Comparison of measured load-CMOD curve with FEM predictions

Fig. 7—ESPI fringe patterns showing crack extension at peak load
Fig. 8—Deformed FEM mesh

Fig. 9—Distribution of bridging stress at interface
Fig. 10—SEM image of penetration of cement paste into crevices of limestone

Fig. 11—Highly oriented crystals observed in some interface region
Fig. 12—Shrinkage cracks along interface

Fig. 13—Interface transition zone observed in some interface region
Fig. 14—Inter-penetration and mechanical interlock in mortar/rock salt interface

Fig. 15—Loose connections and cracking along mortar/rock salt interface
Fig. 16—Geometry of interface crack

Fig. 17—Bridging zone near interface crack tip
Fig. 18—Decomposition of interface fracture problem with bridging zone
\[ l = l_{cr}/l_{cr} \]

\[ w_0 = w/w_c \]

Critical length of bridging zone

Critical crack opening

Fig. 19—Interface crack profiles for different bridging zone length

Fig. 20—R-curve of interface predicted by cohesive zone model

Fig. 21—Semi-infinite interface crack loaded by two equal and opposite concentrated forces at crack faces
APPENDIX

Derivation of full field interface crack opening displacement

In the problem of two semi-infinite planes bonded along the positive x-axis, a line crack is situated along the negative x-axis extending from x=0 to x=-\infty and having two equal and opposite forces \( R = P + iQ \) applied at \( z = -a \), as shown in Figure 21.

The basic complex functions \( \Phi \) and \( \Psi \) for 2D isotropic elasticity has been given by Muskhelishvili (1953). In the case of two different materials, the elastic properties are discontinuous across the interface and the complex functions must be different for the two materials across the interface. A complete solution to the problem requires four complex functions \( \Phi_j \) and \( \Psi_j \), \( j = 1,2 \), corresponding to material 1 and material 2, respectively. The displacement and stress fields can be expressed by the complex function in the form:

\[
(\sigma_x)_j + (\sigma_y)_j = 4 \text{Re}\{\Phi_j(z)\} \tag{13}
\]
\[
(\sigma_y)_j - (\sigma_x)_j + 2i (\tau_{xy})_j = 2[(\bar{z}) \Phi_j(z) + \Psi_j(z)] \tag{14}
\]
\[
2\mu_j (u_j + i v_j) = k_j \int \Phi_j(z) dz - z \Phi_j(z) - \int \Psi_j(z) dz \tag{15}
\]

where \( z = x + iy \), \( u_j \) and \( v_j \) are components of displacement; \( (\sigma_x)_j \) and \( (\sigma_y)_j \) are components of stress; \( \mu_j \) and \( k_j \) are the same as defined before.

The complex functions, \( \Phi_j \) and \( \Psi_j \) in Equation (13) to Equation (15) have to satisfy the boundary conditions of

\[
(\sigma_y + \tau_{xy})_1 = (\sigma_y + \tau_{xy})_2 = 0 \quad \text{for } y = 0 \quad \text{and} \quad x < 0, \tag{16}
\]
\[
(\sigma_y + \tau_{xy})_1 = (\sigma_y + \tau_{xy})_2 = 0 \quad \text{for } y = 0 \quad \text{and} \quad x > 0, \tag{17}
\]
\[
(u + i v)_1 = (u + i v)_2 = 0 \quad \text{for } y = 0 \quad \text{and} \quad x = 0. \tag{18}
\]

The complex function \( \Phi_j \) and \( \Psi_j \) in Equations (13) to (16) which satisfy the boundary conditions of Equations (16) to (18) have been found out by Rice and Sih (1965) as

\[
\Phi_1(z) = z^{-1/2} e^{i \pi \varepsilon} f(z) \tag{19}
\]
\[
\Psi_1(z) = e^{2i \pi \varepsilon} z^{-1/2} \bar{f}(z) - z^{-1/2} e^{i \pi \varepsilon} \left[ (\frac{1}{2} - i \varepsilon) f(z) + z f'(z) \right]
\]

for the region \( y > 0 \) and

\[
\Phi_2(z) = e^{2i \pi \varepsilon} z^{-1/2} e^{i \pi \varepsilon} f(z) \tag{20}
\]
\[
\Psi_2(z) = e^{2i \pi \varepsilon} z^{-1/2} \bar{f}(z) - e^{2i \pi \varepsilon} z^{-1/2} e^{i \pi \varepsilon} \left[ (\frac{1}{2} - i \varepsilon) f(z) + z f'(z) \right]
\]

for the region \( y < 0 \), with

\[
f(z) = \frac{P - iQ}{2\pi e^{2\pi \varepsilon}} \frac{a^{1/2 + i\varepsilon}}{z + a} \tag{21}
\]
The displacement vector of the upper crack surface at \( z = -x \) (\( x > 0 \)) can be found by substituting Equation (19) into Equation (15). Notice that, \( z = -x + i0 = x e^{i\pi} \). Therefore,

\[
[\Phi_{1}(z)]_{z=-x+i0} = -i e^{\pi x} x^{1/2+i\epsilon} f(-x)
\]  

(22)

\[
\left[ z\Phi_{1}(z) \right]_{z=-x+i0} = -i e^{\pi x} x^{1/2+i\epsilon} f(-x)
\]

\[
= i e^{\pi x} \int x^{-1/2+i\epsilon} \left[ \frac{1}{2} + i \epsilon \right] \overline{f}(-x) - x \overline{f}'(-x) \right] dx
\]

(23)

\[
[\overline{\Phi_{1}(z)}]_{z=-x+i0} = i e^{\pi x} \left[ x^{-1/2+i\epsilon} f(-x) - x^{-1/2+i\epsilon} \left[ \frac{1}{2} + i \epsilon \right] \overline{f}(-x) - x \overline{f}'(-x) \right]
\]

(24)

Substituting Equations (22) to (24) into Equation (15), the displacement vector of the upper crack surface at a distance \( x \) from the crack tip can be obtained as:

\[
2\mu_{1}(u_{1} + i v_{1}) = i (1 + k_{1}) e^{\pi x} \int x^{-1/2+i\epsilon} f(-x) \right] dx
\]

(25)

Similarly, notice that, \( z = -x - i0 = x e^{-i\pi} \); the displacement vector of the lower crack surface at \( z = -x \) can be found by substituting Equation (20) into Equation (15) as:

\[
2\mu_{2}(u_{2} + i v_{2}) = i (1 + k_{2}) e^{\pi x} \int x^{-1/2-i\epsilon} f(-x) \right] dx
\]

(26)

The interface crack opening displacement vector at \( z = -x \) is given as:

\[
\Delta u + i \Delta v = (u_{1} - u_{2}) + i (v_{1} - v_{2}) = i \frac{c_{1} + c_{2}}{2} e^{\pi x} \int x^{-1/2-i\epsilon} f(-x) \right] dx
\]

(27)

with \( c_{j} = (1 + k_{j})/\mu_{j}, j = 1,2 \). Substituting equation (21) into the above Equation, we obtain

\[
\Delta v + i \Delta u = \frac{c_{1} + c_{2}}{4\pi} (P + iQ) a^{1/2-i\epsilon} \int_{-x}^{x} \frac{x^{-1/2+i\epsilon}}{a - x} \right] dx
\]

(28)

In order to determine the arbitrary integral constant in the above equation, we notice that when \( x = 0 \) the crack opening displacement is 0, and Equation (28) can be written as

\[
\Delta v(r) + i \Delta u(r) = \frac{c_{1} + c_{2}}{4\pi} (P + iQ) a^{1/2-i\epsilon} \int_{0}^{r} \frac{x^{-1/2+i\epsilon}}{a - x} \right] dx
\]

(29)

By the definition of the function \( g(a,r) \), which is the interface crack opening displacement generated by \( (P + iQ) = 1 \). Then,
Now let us apply equation (30) to two simplified cases in order to check its validity. First, it is expected that the near tip displacement field given by Equation (30) should coincide with that given for the interface fracture mechanics (Rice, 1988). As $r$ is sufficiently small compared to a, i.e., $r \ll a$, Equation (30) gives

$$\Delta v(r) + i \Delta u(r) = (P + iQ) g(a, r) = \frac{c_1 + c_2}{2\pi} \frac{P + iQ}{1 + 2iE} \frac{r^{1/2 + ie}}{a}$$  \hspace{1cm} (31)$$

In this case the complex stress intensity factor is given as

$$K_c = (P + iQ) \sqrt{\frac{2}{\pi}} \cosh(\pi e) a^{-1/2 - ie}$$  \hspace{1cm} (32)$$

By using Equation (32), Equation (31) can be rewritten as:

$$\Delta v(r) + i \Delta u(r) = \frac{c_1 + c_2}{2\sqrt{2\pi}} \frac{K_c r^{1/2 + ie}}{(1 + 2iE) \cosh(\pi e)}$$  \hspace{1cm} (33)$$

this is identical to that given by Rice (1988).

Secondly, when the properties of material 1 and material 2 involved above are the same, Equation 33 should give the same solution as that of homogeneous materials. By letting $e$ equal 0 and $c_1 = c_2 = (1 + k)/\mu = 8/E'$, Equation (30) gives

$$g(a, r) = \frac{4}{\pi E'} \log \left[ \frac{\sqrt{a + r}}{\sqrt{a - r}} \right]$$  \hspace{1cm} (34)$$

where $E' = E$ (plane stress) or $E/(1 - v^2)$ (plane strain). It can be found that Equation (34) is the same as that given by Nirmalendran and Horii (1992) for homogeneous materials.
The Role of Interfacial Fracture Toughness in Cracking Behavior of High-Strength Concrete

by K. M. Lee, O. Buyukozturk, and Y. Kitsutaka

Synopsis: The global behavior of concrete is influenced by various scenarios of crack initiation and crack propagation. Recently, the study of the interface fracture and cracking in interfacial regions has emerged as an important research field, especially, in the context of the development of high performance concrete composites. For a rigorous study, the use and further development of fracture mechanics based concepts are needed. The crack path criterion for elastically homogeneous materials is not valid when the crack advances at an interface because, in this case, the consideration of the relative magnitudes of the fracture toughnesses between the constituent materials and the interface are involved. In this paper, criteria based on energy release rate concepts are considered for the prediction of crack growth at the interfaces and an experimental/numerical study is presented on two-phase composite models of concrete to investigate the cracking scenarios in interfacial regions. From the testing and numerical analysis on physical models the interface fracture and the crack propagation in concrete composites is studied and the role of interface fracture toughness is discussed.

Keywords: Composite materials; cracking (fracturing); fracture mechanics; fracture properties; high-strength concretes; interface; models; toughness
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INTRODUCTION
Concrete composites consist of various elements such as cement paste, sand, coarse aggregates, and fibers. Significant considerations in understanding the mechanical behavior of the concrete composites include the nature of the deformation and failure at interfaces between these constituent elements and their interaction. Development of advanced concrete composite materials with improved toughness and durability requires a fundamental understanding of the characteristics of the interfaces. For example, one outstanding question in the development of high strength concrete is the brittleness of the material and the role of matrix-aggregate interfaces in that respect. Although a strong bond at the interface between the matrix and the aggregate may enhance the overall strength and stiffness of the concrete system, the increase in the interfacial bond strength would cause a brittle deformation and failure behavior. In that respect, various scenarios of crack initiation and crack propagation need to be studied in order to engineer the material for an optimum behavior. This paper presents the analysis of interface fracture and crack propagation in the interface regions of two-phase concrete composite models using a fracture mechanics based methodology.

INTERFACE FRACTURE PARAMETERS

Interface Fracture Mechanics Concepts
Consider a semi-infinite free crack lying along an interface between two homogeneous isotropic half planes, with material 1 above and material 2 below (see Fig. 1). The elastic moduli mismatch parameters that govern interface crack fields in plane strain are
\[ \alpha = \frac{\bar{E}_1 - \bar{E}_2}{\bar{E}_1 + \bar{E}_2}, \quad \beta = \frac{1}{2} \frac{\mu_1(1 - 2v_2) - \mu_2(1 - 2v_1)}{\mu_1(1 - v_2) + \mu_2(1 - v_1)} \]  

where \( \bar{E} = E/(1-v^2) = 2\mu/(1-\nu) \), and \( E, \mu, \) and \( \nu \) are Young's modulus, shear modulus, and Poisson's ratio, respectively; the subscripts 1 and 2 refer to the two materials. The parameter \( \alpha \) measures the relative stiffness of the two materials. The parameter \( \beta \) causes the linear crack tip stress and displacement fields to oscillate.

For the plane problems, the normal and shear stresses of the singular field acting on the interface a distance \( r \) ahead of the crack tip can be written by

\[ \sigma_{22} + i\sigma_{12} = \frac{K}{\sqrt{2\pi r}} r^\varepsilon \]  

where \( K = K_1 + iK_2 \) is the stress intensity factor at the interface, \( i = \sqrt{-1} \), and the oscillation index \( \varepsilon \) is defined as

\[ \varepsilon = \frac{1}{2\pi} \ln \left( \frac{1 - \beta}{1 + \beta} \right) \]  

It is noted that \( K_1 \) and \( K_2 \) do not strictly measure the normal and shear traction singularities on the interface ahead of the crack tip due to the term \( r^\varepsilon \) in Eq. (2).

Fracture at a bimaterial interface can be expressed in terms of two parameters (1,2). The first parameter is the energy release rate, \( G \), per unit length of extension of the crack along an interface as

\[ G = \frac{1}{2\cosh^2 \pi \varepsilon} |K|^2 \]  

where \( |K|^2 = K_1^2 + K_2^2 \), and \( \cosh^2(\pi \varepsilon) = 1/(1-\beta^2) \). The second parameter is the loading phase angle which is a measure of the contribution of shear to opening experienced by the interface crack surface. The phase angle \( \Psi \) is defined as

\[ \Psi = \tan^{-1} \left( \frac{\text{Im}(K\hat{L}^{\varepsilon})}{\text{Re}(K\hat{L}^{\varepsilon})} \right) \]  

where \( \hat{L} \) is a fixed length and somewhat arbitrary. When \( \beta = 0 \), the phase angle denoted by \( \psi \) is defined as \( \tan^{-1}(K_2/K_1) \). Interface fracture occurs when the interfacial energy release rate \( G \) reaches the interfacial fracture energy \( \Gamma_i \), that is characterized as a function of \( \Psi \).

**Measurement of Interface Fracture Toughness**

In general, the fracture at the interfaces often appears in mixed mode by tensile loading and shear loading. Differences between the material properties of matrix
and aggregates would also result in the mixed mode fracture failure even when the
game and loading are symmetric with respect to crack plane. Recently, a
methodology involving a combined fracture mechanics based analysis and
experimentation using interface fracture test specimens were developed to establish
the interface fracture toughness curves in the full range of mixed mode effects
(3,4). The test specimens include the sandwiched beams and the sandwiched
Brazilian disk specimens, from which mixed mode stress states ranging from pure
mode 1 to pure mode 2 can be achieved. As an example, the interface fracture
energy curve for a mortar-granite interface are given in Fig. 2. It is observed from
Fig. 2 that the interfacial fracture energy markedly increases as the loading phase
increases. This is attributed to the shielding effects at the interfaces with increased
shear loading.

CRITERIA FOR THE PREDICTION OF CRACK GROWTH IN
INTERFACIAL REGIONS OF CONCRETE COMPOSITES

In elastically homogeneous brittle solids, cracks propagate such that mode I
conditions are maintained at the crack tip. This crack path criterion is not valid
when a crack advances in the region of an interface because in this case the relative
magnitudes of the fracture toughnesses between the interface and the constituent
materials are also involved. Cracking scenarios such as the crack penetration vs.
and deflection at an interface and crack kinking out of an interface need to be
studied. For this, criteria based on energy release rate concepts may be
considered.

Crack Penetration vs. Deflection at an Interface

In a concrete composite system a mortar crack impinging an interface may advance
by either penetrating into the aggregate inclusions (material 1) or deflecting along
the interface (see Fig. 3). The impinging crack is likely to be deflected if

$$\frac{\Gamma_i}{\Gamma_i} < \frac{G_d}{G_p^{\text{max}}}$$

where $\Gamma_i$ is the toughness of the interface as a function of the loading phase angle,
$\Gamma_i$ is the mode I toughness of material 1, $G_p^{\text{max}}$ is the maximum energy release
rate of the penetrated crack (Fig. 3a), and $G_d$ is the energy release rate of the
deflected crack (Fig. 3b). The ratio $G_d/G_p^{\text{max}}$ on the right hand side of Eq. (6)
can be computed by solving the crack problem for a semi-infinite body (5). For an
interface crack $\psi$ and $G_d$ can be evaluated by a finite element based numerical
method (6). When $\alpha = 0$ and $\gamma_2 = 90^\circ$ the critical ratio of $G_d/G_p^{\text{max}}$ is
approximately 0.25, meaning that the crack will deflect if the interface toughness is
less than a quarter of the toughness of the material ahead of the crack (5).

Kinking of a Crack out of an Interface

A crack staying in an interface may propagate along the interface or kink into the
mortar matrix or aggregate. The kinking of a crack out of an interface between
two brittle solids has been analyzed (7,8). As shown in Fig. 4, a semi-infinite
crack lies along the interface with its tip at the origin. Prior to kinking ($\alpha = 0$), the parent crack is loaded with a complex interface stress intensity factor $K$ with the specific phase angle. The interface crack will kink if

$$\frac{G}{G_{\text{max}}} < \frac{\Gamma_1}{\Gamma_2}$$

where $\Gamma_1$ and $\Gamma_2$ denote the interface fracture toughness and the mode I toughness of material 2, respectively, $G$ is the energy release rate for straight-ahead advance in the interface, and $G_{\text{max}}$ is defined as the maximum of $G_i$ with respect to kink angle $\Omega$ for a given $\psi$. It is found that the preferred crack path is influenced by the magnitude of the phase angle, such that crack kinking out of the interface is most likely when $\psi$ is $65^\circ$ with $\alpha = 0$ (8).

EXPERIMENTAL/NUMERICAL ANALYSIS OF COMPOSITE BEAM MODELS WITH A SLAB INCLUSION

Model Description and Test Procedure

A composite model shown in Fig. 5 was developed as an application to study the crack penetration versus crack deflection in interface regions. This model consists of a beam of mortar and a slab inclusion of rock. One can achieve different crack hitting angles in the beam model by appropriately inclining the embedded inclusion. The main purpose of this composite model study is to verify the application of a criterion given in Eq. (6) to cracking problems that may occur in concrete composites.

The dimensions of the beam specimen were 228.6 mm (span length) x 76.2 mm (height) x 25.4 mm (width) and the dimensions of the inclusion were 50.8 mm (length) x 12.7 mm (height) x 25.4 mm (width). The inclination angle of rock slab, $\gamma$ ($\gamma_2$ in Fig. 3), was adjusted to two different angles, $90^\circ$ and $60^\circ$, with respect to the impinging crack. The pre-crack introduced using a diamond saw was close enough to the slab inclusion so that the crack propagated, in general, vertically and that the crack hitting angle onto the inclusion was nearly the same as the slab inclination angle. The crack length, $a$, was 25.4 mm and thus, the ratio of the crack length to the beam height was 0.33.

For manufacturing of beam specimens three mortar mixes (M1, M2, and M3) for matrices and one rock (granite) for inclusions were used. Thus, in the present experiment three mortar/rock combinations were considered. M1 is normal strength mortar with no silica fume, and M2 and M3 are high strength mortars with silica fume. Table 1 shows the mix proportions for the three mortar mixes. Average mechanical properties of the tested materials are listed in Table 2.

After aggregate inclusions were placed in the beam molds made of plexiglass, fresh mortar mixture was poured. The specimens were removed from the molds 24 hours later, and cured in water for 28 days. Before testing of beams, a pre-crack was introduced in the middle of beam using a diamond saw. A rock inclusion was very close to the pre-crack such that a distance between the crack tip and the bottom surface of the rock inclusion was less than 1.0 mm. The beam specimen was placed in a loading fixture for bending test and then, four-point bending test was performed using a compression machine with a stroke control at a loading rate of 0.0762 mm/min. A computerized data acquisition system was
used to record the applied load and deflection at the loading point and the crack path was monitored.

**Cracking Loads and Cracking Mode**

The cracking loads and cracking modes of tested specimens are listed in Table 3. When the hitting angle was 90°, the cracking loads of three M1/G specimens ranged from 2.22 kN to 2.63 kN; the failure mode being crack deflection. On the other hand, for both M2/G and M3/G system with \( \gamma = 90^\circ \) the failure mode was crack penetration with higher cracking loads. This is attributed to the effects of silica fume combined with low water/cement ratio, leading to the strong interface. It is also observed from Table 3 that when the hitting angle is 60°, the cracking load decreases and the cracking mode tends to be the crack deflection. This is caused by the fact that the energy release rate of the interface direction increases but the corresponding interfacial fracture toughness decreases.

**Finite Element Analysis**

For the composite model tested, the energy release rate and the loading phase angle for the deflected crack were calculated by the finite element analysis scheme. This numerical method is based on the virtual crack extension method for the evaluation of the energy release rate and the crack surface displacement method for the evaluation of the phase angle (6). Finite element analysis was performed on the models with \( \gamma = 90^\circ \). Materials constituting the beam model were assumed to be linear and elastic. The finite element mesh shown in Fig. 6 consists of 2D eight-node isoparametric plane strain elements. In the crack tip region the mesh was refined using 9 element rings, and 24 triangular elements with quarter mid-point nodes at the crack tip. The total number of elements was 1488. For the analysis of the deflected crack, a small crack \( \Delta a \) of 2.286 mm was introduced along the interface. Table 4 presents the Poisson’s ratios and the mismatch parameters \( \alpha \) and \( \beta \) for the material combination considered in the finite element analysis. Table 5 shows the values of the energy release rate and the phase angle for the deflected crack. It is observed from Table 5 that as the parameter \( \alpha \) decreases the phase angle increases. This trend agrees well with the results by He and Hutchinson (5).

**Numerical Simulation**

Comparison of cracking modes from the composite model tests and those from the numerical results is given in Table 6. This numerical simulation was limited to models with \( \gamma = 90^\circ \) only. As the first example, the results of M1/G system will be discussed. For M1/G composite beam, the phase angle for the deflected crack is 37° (Table 5). From the tests the mode I fracture energy of granite, \( \Gamma_g \), was 17.5 J/m² and \( \Gamma_i(37^\circ) \) for M1 was approximately 5.0 J/m² (3). Therefore, the ratio \( \Gamma_i(37^\circ)/\Gamma_g \) which is 0.28 is found to be less than the ratio \( G_d/G_p^{\text{max}} \) which is 0.37 (6). Thus, according to Eq. (6) the impinging crack can be predicted to be deflected. This agrees with the test results showing the interface cracking. With the measured cracking loads ranging from 2.22 kN to 2.63 kN, the average value of the interface fracture energy, \( \Gamma_i(37^\circ) \), is predicted to be 5.9 J/m² from the numerical calculation. This predicted value is somewhat higher than 5.0 J/m².
which is the average value measured directly from the sandwiched specimen testing. This difference may be attributed to either the linear elastic material assumption in the numerical analysis or the difference of specimen geometries. However, the agreement is reasonable.

Secondly, for M3/G beam composite model, the phase angle for the deflected crack was calculated to be 41° and thus, the ratio $\Gamma_1(41°)/\Gamma_g = 0.46$ is found to be larger than the ratio $G_d/G_{pl,max} = 0.30$. Thus, Eq. (6) predicts the crack penetration. This agrees with the test results showing the transgranular cracking. For this case, the fracture toughness of granite can be obtained. From the cracking loads, the average fracture toughness of granite is predicted to be 25.0 J/m². The predicted fracture toughness value of granite are 34 % higher than the average measured value of 17.5 J/m², as obtained from the fracture toughness testing of granite. As shown in Table 6, all specimens exhibited an agreement between the cracking modes from composite model testing and those from the criteria. It is seen that the criterion given by Eq. (6) provides a reasonable prediction for all cases, and may be used for studying the crack penetration vs. deflection scenarios in the interfacial regions of concrete composites.

**TESTING OF COMPOSITE BEAM MODELS WITH A CIRCULAR INCLUSION**

**Experimental Procedure**

To study the effects of crack patterns on the fracture properties of high strength concrete composites, beam composite models shown in Fig. 7 were developed and tested. Specimen size was 228.6 mm (span length) x 76.2 mm (height) x 25.4 mm (thickness), and notch length was 25.4 mm. For the two-dimensional continuous inclusion case, two aggregate disks of 12.7 mm diameter were arranged (Type 1), and for the three-dimensional discontinuous inclusion case, four small aggregate disks were arranged (Type 2). In manufacturing these composite beams the same materials as those for the composite beam with a slab inclusion were used. Three-point bending tests were conducted to establish load versus load line displacement curves and crack paths were monitored.

**Results and Discussion**

The results including cracking mode and crack hitting angle ($\gamma$) onto the lower rock inclusion, are shown in Table 7. It is observed that crack path of all M1 composite beams is crack deflection, i.e., interface cracking. M3/G specimen with type 2 (T2) cross section failed by crack penetration into the aggregate, while M3/G specimen with type 1 cross section (T1) failed by interface cracking. When $\gamma$ is close to the 90° the crack penetration is likely to occur. The observed crack patterns generally agree with the predictions by the criterion given in Eq. (6). Figure 8 shows the typical load versus load line displacement (LLD) curves obtained from the tests. The load versus LLD curves for the M3/G specimens were much affected by the granite inclusion. This is because with these specimens there appears to be an increased effect of the crack arrest mechanism by the strong aggregate and/or interface.

Fracture energy values computed from the area under the load-deflection curve are listed in Table 7. It is also noted that M1/G and M3/G composite beams
with Type 2 cross-section have higher fracture energy than that of the mortar specimens, showing a trend similar to that observed in the real concrete testing. This may be due to the significant toughening effects by the crack trapping mechanism in the three-dimensional system. Thus, it may be stated that composite beam models with discontinuous inclusions (3-D model) may simulate well the behavior of real concrete in which aggregates are randomly distributed.

CONCLUSION

For designing concrete composite materials with optimum mechanical properties, the relative properties of mortar, aggregate, and the interface between them should be considered. In particular, the interface fracture toughness plays a major role in determining the concrete properties. Quantitative studies on the role of the interface fracture toughness on global behavior of concrete, both experimental and analytical, should be performed.

Development of advanced concrete composite materials with improved toughness and durability requires a fundamental understanding of cracking scenarios in the interfacial regions. For this purpose analysis of interface fracture and crack propagation is an essential tool. In this paper, criteria based on energy release rate concepts are considered for the prediction of crack growth at the interfaces and an experimental/numerical study is presented on the two-phase composite models of concrete to investigate the cracking scenarios in interfacial regions. The numerical predictions are compared with the results from an experimental study, and, generally, good agreement is found.

ACKNOWLEDGEMENT

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REFERENCES


**Table 1 — Mix Proportions for Mortar Mixes (by weight)**

<table>
<thead>
<tr>
<th></th>
<th>W/(C+SF)</th>
<th>Sand/C</th>
<th>SF/C (%)</th>
<th>HRWR/C (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar 1</td>
<td>0.50</td>
<td>2.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Mortar 2</td>
<td>0.35</td>
<td>2.0</td>
<td>5.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Mortar 3</td>
<td>0.28</td>
<td>2.0</td>
<td>10.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

W: water, C: cement, SF: silica fume, HRWR: high range water reducer

**Table 2 — Mechanical Properties for Three Mortars and Granite**

<table>
<thead>
<tr>
<th>Material</th>
<th>(\sigma_c) (MPa)</th>
<th>E (GPa)</th>
<th>(G_{IC}) (J/m²)</th>
<th>(v)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar 1</td>
<td>42.5</td>
<td>27.8</td>
<td>10.3</td>
<td>0.22</td>
</tr>
<tr>
<td>Mortar 2</td>
<td>58.5</td>
<td>33.8</td>
<td>14.8</td>
<td>0.22</td>
</tr>
<tr>
<td>Mortar 3</td>
<td>80.5</td>
<td>39.2</td>
<td>16.7</td>
<td>0.20</td>
</tr>
<tr>
<td>Granite</td>
<td>140.1</td>
<td>55.3</td>
<td>17.5</td>
<td>0.16</td>
</tr>
</tbody>
</table>

**Table 3 — Cracking Load and Cracking Mode of Beam Composite Specimens with a Slab Inclusion**

<table>
<thead>
<tr>
<th>Material Combination</th>
<th>Hitting Angle, (\gamma) (degree)</th>
<th>Cracking Load (kN)</th>
<th>Cracking Mode¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1/G-1</td>
<td>90</td>
<td>2.22</td>
<td>D</td>
</tr>
<tr>
<td>M1/G-2</td>
<td>90</td>
<td>2.47</td>
<td>D</td>
</tr>
<tr>
<td>M1/G-3</td>
<td>90</td>
<td>2.63</td>
<td>D</td>
</tr>
<tr>
<td>M1/G-4</td>
<td>60</td>
<td>1.48</td>
<td>D</td>
</tr>
<tr>
<td>M2/G-1</td>
<td>90</td>
<td>2.76</td>
<td>P</td>
</tr>
<tr>
<td>M2/G-2</td>
<td>90</td>
<td>3.00</td>
<td>P</td>
</tr>
<tr>
<td>M2/G-3</td>
<td>60</td>
<td>2.01</td>
<td>D</td>
</tr>
<tr>
<td>M2/G-4</td>
<td>60</td>
<td>3.03</td>
<td>P</td>
</tr>
<tr>
<td>M3/G-1</td>
<td>90</td>
<td>2.92</td>
<td>P</td>
</tr>
<tr>
<td>M3/G-2</td>
<td>90</td>
<td>3.17</td>
<td>P</td>
</tr>
<tr>
<td>M3/G-3</td>
<td>60</td>
<td>1.84</td>
<td>D</td>
</tr>
</tbody>
</table>

¹ D: crack deflection and P: crack penetration
### TABLE 4 — DUNDURS' PARAMETERS FOR THREE MORTAR-AGGREGATE COMBINATIONS

<table>
<thead>
<tr>
<th>Combinations (#2/#1)</th>
<th>$E_1$ (GPa)</th>
<th>$v_1$</th>
<th>$E_2$ (GPa)</th>
<th>$v_2$</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1/G</td>
<td>55.3</td>
<td>0.16</td>
<td>27.8</td>
<td>0.22</td>
<td>0.320</td>
<td>0.099</td>
</tr>
<tr>
<td>M2/G</td>
<td>55.3</td>
<td>0.16</td>
<td>34.0</td>
<td>0.22</td>
<td>0.227</td>
<td>0.064</td>
</tr>
<tr>
<td>M3/G</td>
<td>55.3</td>
<td>0.16</td>
<td>39.2</td>
<td>0.20</td>
<td>0.163</td>
<td>0.049</td>
</tr>
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</table>

### TABLE 5 — NUMERICAL RESULTS OF ENERGY RELEASE RATE AND CORRESPONDING PHASE ANGLE

<table>
<thead>
<tr>
<th>Combination</th>
<th>$G_d^{1}$</th>
<th>$\psi^{2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1/G</td>
<td>0.4460</td>
<td>37.4</td>
</tr>
<tr>
<td>M2/G</td>
<td>0.3970</td>
<td>39.2</td>
</tr>
<tr>
<td>M3/G</td>
<td>0.3672</td>
<td>40.6</td>
</tr>
</tbody>
</table>

1 nondimensionalized by $P^2 l_2^2 (1-v_1^2) a/E_1 d^4 b^2$
2 with the fixed length = 2.54 mm

### TABLE 6 — COMPARISON OF CRACKING MODES FROM TEST RESULTS AND PREDICTIONS BY CRITERIA IN BEAM COMPOSITE MODELS WITH SLAB INCLUSION

<table>
<thead>
<tr>
<th>Material Combination</th>
<th>Cracking Mode from Testing</th>
<th>$\Gamma_2/\Gamma_a$</th>
<th>$G_d/G_p^{*}$</th>
<th>Cracking Mode by Criteria</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1/G-1</td>
<td>D</td>
<td>0.28</td>
<td>0.37</td>
<td>D</td>
<td>O.K.</td>
</tr>
<tr>
<td>M1/G-2</td>
<td>D</td>
<td>0.28</td>
<td>0.37</td>
<td>D</td>
<td>O.K.</td>
</tr>
<tr>
<td>M1/G-3</td>
<td>D</td>
<td>0.28</td>
<td>0.37</td>
<td>D</td>
<td>O.K.</td>
</tr>
<tr>
<td>M2/G-1</td>
<td>P</td>
<td>0.40</td>
<td>0.33</td>
<td>P</td>
<td>O.K.</td>
</tr>
<tr>
<td>M2/G-2</td>
<td>P</td>
<td>0.40</td>
<td>0.33</td>
<td>P</td>
<td>O.K.</td>
</tr>
<tr>
<td>M3/G-1</td>
<td>P</td>
<td>0.46</td>
<td>0.30</td>
<td>P</td>
<td>O.K.</td>
</tr>
<tr>
<td>M3/G-2</td>
<td>P</td>
<td>0.46</td>
<td>0.30</td>
<td>P</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

* from He and Hutchinson (6)
TABLE 7 — CRACKING MODE AND FRACTURE ENERGY OF BEAM COMPOSITE SPECIMENS WITH CIRCULAR INCLUSIONS

<table>
<thead>
<tr>
<th>Material Combination</th>
<th>Hitting Angle, $\gamma$ (degree)</th>
<th>Fracture Energy ($J/m^2$)</th>
<th>Cracking Mode$^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1/G-T1-1</td>
<td>63</td>
<td>44.0</td>
<td>D</td>
</tr>
<tr>
<td>M1/G-T1-2</td>
<td>66</td>
<td>24.3</td>
<td>D</td>
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<tr>
<td>M1/G-T2-1</td>
<td>60</td>
<td>56.7</td>
<td>D</td>
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<tr>
<td>M1/G-T2-2</td>
<td>90</td>
<td>45.9</td>
<td>D</td>
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$^1$ D: crack deflection and P: crack penetration
Fig. 1—Geometry of interface crack

Fig. 2—Fracture toughness curve for M3/G interface [from (4)]
a) Crack penetration  b) Crack deflection

Fig. 3—Crack geometry

Fig. 4—Conventions for a crack kinking out of an interface
Fig. 5—Geometry of beam model subjected to four-point bending

Fig. 6—Finite element mesh
Fig. 7—Two-phase beam model with circular inclusions

Fig. 8—Load versus LLD curves of M3 specimens with type 2 cross-section
A Lattice Approach for Analyzing Steel-Concrete Bond-Slip-Layer Fracture

by A. Vervuurt and J. G. M. Van Mier

Synopsis: Crack Propagation in composite materials is a very complex process. Of utmost importance seems the behaviour of the interfaces between the constituting phases. In reinforced concrete, interfaces not only appear in the concrete itself, but also between the concrete and the steel reinforcement. Fracture of the steel-concrete interface can be seen as a combination of adhesion, mechanical interlock and frictional stress transfer. In this paper steel-concrete interface fracture is modelled at the meso level. At this level a simple linear elastic fracture law seems to suffice to explain global fracture mechanisms of composite materials. Interfaces between aggregate and matrix and between matrix and reinforcing bars are simulated using a lattice model. In the model the material is discretized as a lattice of brittle breaking beam elements. Disorder of the material is implemented by assigning different strength and stiffness properties to the beam elements. Cracking is simulated by removing in each load step the element with the highest stress over strength ratio. The model is applied to uniaxial tensile fracture of plain concrete specimens and to bond of steel to concrete. Comparison from the simulations presented in this paper with experimental data shows that crack mechanisms are simulated quite accurately. However the bond-displacement behaviour is still too brittle. This point can be improved when more detail is included in the material structure that is incorporated in the analysis. The macroscopic bond-slip behaviour of a reinforcing bar depends strongly of the micro cracking near the interfacial zone between concrete and rebar. The analyses clearly show the influence of adhesion between steel and concrete on the simulated crack patterns.

Keywords: Bonding; composite materials; cracking (fracturing); failure; fracture properties; interface; models; particle size distribution; reinforced concrete; steels
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INTRODUCTION

Interfaces play an important role in concrete technology and reinforced concrete structural design. In the material itself, interfaces are encountered at the meso-level between the matrix and the aggregate particles. In reinforced concrete the interface between steel and concrete is essential for a good performance of the structure. In both cases, i.e. for the behaviour of the interface between aggregate particles at the meso-level and the steel-concrete interface, three different types of bonding must be considered. These are, physical-chemical interaction between the two interfaces (adhesion), mechanical interlock and frictional stress-transfer.

Let us first consider the aggregate-matrix interface. In general chemical bonding seems of importance only when reactive aggregates are used (1). Porosity of the aggregates seems to play a major role in the development of the interface between aggregate and cement paste (2). The interface zone between the aggregate and the cement paste is rather porous when normal dense natural aggregates (such as river gravel) are used, see for example in Mindess (3). However, when more porous aggregates are used, for example artificial products such as lytag, a more dense bond zone can develop (4), due to the suction of water from the cement paste at the interface. Mechanical interlock of aggregate particles seems more effective for irregular and flaky types of aggregates. This is mainly of importance when aggregate interlock is modelled (5). In that case the resistance of the interface (which is now a crack) to in-plane shear or tension normal to the interface plane is enhanced. Note that in aggregate interlock experiments at small crack openings often secondary cracking may develop inclined to the main crack (6). The development of these secondary cracks is in many cases triggered by irregularities in the crack plane. When such secondary cracking appears (7), friction on the interface seems of less importance, and a simple tensile fracture model suffices to describe the observed phenomena.

For the bond between steel and concrete, adhesion (chemical-physical interactions) seems of less importance. The steel can be compared to the non-porous natural aggregates such as river gravel. Mechanical interlock and friction seem
to be the major load-transfer mechanisms for steel to concrete interfaces. These latter mechanisms are often enhanced by using special ribbed reinforcing bars. For example the bond between ribbed reinforcing bars and concrete leads to a completely different stress-transfer mechanism and distinct crack patterns develop in the concrete near the ribs on the reinforcing bar as shown clearly in the classical experiments of Goto (8) and the more recent tests of Otsuka (9).

The details of the interaction between aggregates and cement matrix at the meso-level, or between a steel bar and concrete can be better understood using numerical modelling techniques. The sole use of experiments is often too limited, or even some phenomena like internal cracking are very difficult to visualise. Early work on the analysis of bond-slip of reinforcing bars was done by Ingraffea et al. (10). At that time the fracture models were still too limited to allow for a full fledged analysis of the problem. In their analyses, Ingraffea and co-workers, ignored the appearance of longitudinal cracking, and the simulations were stopped as soon as the first primary crack appeared. Recently, more detailed analyses were carried out by Rots (11). He used the smeared crack model of DIANA to simulate cracking in a tensile specimen with a central reinforcement bar, which is a very popular test to measure bond-slip relations. The analyses of Rots were a great improvement: a radial symmetric analysis was carried out, allowing for longitudinal cracking (12).

The bond zone between steel and concrete can be looked upon in different manners. The most common point of view is to consider the concrete and steel as two independent operating materials, and to lump the bond-slip behaviour in a zone with zero thickness between the steel and the concrete. In most models, spring type elements are used with a specific bond-slip behaviour. The interface springs are in the ideal case three-dimensional springs (13-15). Another point of view is to assume that most of the slip between steel and concrete originates from the cracking that takes place in a narrow zone of concrete just outside the steel bar. Earlier it was hypothesized that the extent of such a zone could be approximately equal to the diameter of the reinforcing bar. The bond problem could then also be tackled in numerical codes by assuming a zone of a single diameter size around the steel bar. Different constitutive properties for the concrete in this zone would have to be defined then. This idea was worked out by Dragosavić (13-15), and a special detailed bond-slip test was designed for determining the properties of this rather extensive bond-zone. Independent of which point of view is taken, both methods may profit from a detailed analysis of the cracking in the bond-zone. In such an approach the concrete and the steel bar must be modelled to a high degree of accuracy in order to mimic the load-transfer from steel bar to concrete as closely as possible. This means that also the ribs on the steel bar must be modelled. With the development of the lattice model such an approach has become possible. The lattice model was designed for simulating cracking in concrete at the grain level (16-18). In view of the results obtained with the model for cracking in plain concrete, it was expected that good results could also be obtained for modelling steel-concrete
interactions. The only limitation for the present model is that no frictional slip is modelled, but this seems no objection for analyzing the first stage of steel-concrete interactions where cracking is the main cause for the observed behaviour. In this paper, first the lattice model is outlined. Next the capabilities of the model in analyzing fracture of plain concrete are shown. Subsequently a detailed analysis of steel-concrete interactions is given.

**DESCRIPTION OF THE LATTICE MODEL**

In the lattice model, the continuum is discretized in a network of brittle breaking beam elements. A similar procedure was already proposed in 1941 by Hrennikoff (19), who used large trusses to solve problems of elasticity. At the time that Hrennikoff presented his model, computational capabilities were insufficient, and the approach was not followed. Recently, mainly through activities in theoretical physics (20), the approach has received new attention, and modelling fracture has become possible. The major step made by Herrmann (21) was to use a frame model rather than a truss model. Fracturing was simulated by removing beam elements from the frame as soon as the specified failure strength was reached. After removal, the complete frame was relaxed until the next beam could be removed. The model proposed by Herrmann was recently linked to a finite element code (17-19). An important change was made, namely, the square lattice originally proposed by Herrmann (figure 1a) was replaced by either a regular triangular lattice or by a lattice with random beam lengths (22), see figure 1b and 1c. This has the important implication that the macroscopic Poisson's ratio (i.e. the Poisson's ratio of the total lattice) can be adjusted quite accurately to the Poisson's ratio of concrete (or rock). Note that Hrennikoff's original truss-model would always lead to a Poisson's ratio of 0.33, which is clearly much too high for geomaterials.

**Implementing Disorder**

It is essential that heterogeneity is included in the model. Up till now, several different approaches have been followed for introducing disorder in the model. In our first model (23), heterogeneity was implemented by assigning different limit strengths to each of the beam elements in a regular triangular lattice. In the first simulations a normal distribution of beam strengths was assumed, but clearly other distributions could be implemented quite easily. In the second approach (16), first a particle structure of concrete is generated, whereafter it is projected on top of a regular triangular lattice as shown in figure 2. The beam elements falling inside the aggregates, or in the matrix, or at the interface between the aggregate and matrix are given the properties of these distinct phases (figure 2b). Thus, essentially, concrete is modelled as a three-phase
material.

The method of particle overlay as shown in figure 2 is of course not limited to the regular triangular lattice. The same method can be applied using other types of lattices such as the random lattice of figure 1c. Especially this combination is appealing, because in the regular lattice sometimes the cracks seem to follow the mesh lines (22). In the random lattice this problem is eliminated. The random lattice is constructed starting from a regular square grid as shown in figure 3. The method was developed by Moukarzel and Herrmann (24). In each box of the grid (box size s x s mm), a point is selected at random, with uniform distribution. Subsequently the random lattice is defined by connecting always the three points which are closest to each other.

The random lattice can also be used directly, i.e. an overlay with a generated particle structure is not required. When the random lattice is used without particle overlay, disorder is caused by the differences in beam length. If this disorder is strong enough to mimic the structure of concrete is debatable. Of the three methods described above, in particular the particle overlay method, in combination with either the regular triangular lattice or with the random lattice yields very good results for concrete specimens subjected to either uniaxial tensile or combined tensile and shear loadings.

Fracture Law and Model Parameters

As mentioned before, fracture is simulated in the lattice model by removing in each load-step the beam element with the highest stress over strength ratio. The effective stress in a beam is the maximum stress in the outer fibres of the beam following

$$\sigma = \frac{F}{A} + \alpha \times \left( \frac{|M_i|, |M_j|}{\text{max}} / W \right)$$

(1),

where $F$ is the normal force in the beam, $M_i$ and $M_j$ are the bending moments in the nodes $i$ and $j$ of the beam, $A = b \times h$ the cross-section of the beam, and $W = b \times h^2/6$ the section modulus. The factor $\alpha$ is introduced for regulating the amount of bending that is taken into account. In general $0 < \alpha < 1$. The coefficient $\alpha$ mainly influences the tail of the softening diagram (25) but also influences the ratio between tensile and compressive strength of a lattice (35). After a beam has been fractured, simply a new linear elastic analysis is carried out using the reduced lattice. The analyses are carried out with a standard finite element package.

The parameters that have to be specified in the model are quite limited. Contrary to macroscopic fracture models no softening function has to be introduced (11, 26), but only single valued limit strength values have to be
specified for the various constituents, viz. aggregate, matrix and bond zone. In principle the following parameters must be given:

- tensile strength \( f_{ta}, f_{tm}, f_{tb} \)
- Young’s Modulus (in tension) \( E_a, E_m, E_b \)
- beam length \( l \)
- beam size \( b \times h \)
- coefficient \( \alpha \)

where the subscript 'a' stands for aggregate, 'm' for matrix and 'b' for bond zone between aggregate and matrix.

The determination of some of these parameters is quite straightforward. The influence of beam length \( l \) on the global response was systematically investigated (18, 25). It was found that the length \( l \) of the beams in the lattice should be smaller than approximately one third of the diameter of the smallest aggregate particle in the concrete structure. If a larger length is chosen, the effect of the small particles will not be included. As recently demonstrated, the small particle effect is quite substantial in the tail of the softening diagram (27) (see also the next section).

The cross-section of the beams, and the values for the Young’s moduli are related to one another. In general we first select realistic values for the Young’s moduli, i.e. values obtained from macroscopic experiments are used. It remains of course questionable if this is allowed. In most analyses the Young’s moduli are \( E_a = 70 \) GPa and \( E_m = E_b = 25 \) GPa. Now the beam dimensions have to be adjusted to obtain a reasonable value for the stiffness of the complete lattice. The procedure is as follows. First the thickness of the beams is set to the thickness of the element that is to be analyzed. Next the height of the beam is chosen such that the stiffness of the complete lattice matches the stiffness of a real concrete element with the same composition. For the regular triangular lattice with particle overlay (aggregates between 3 and 8 mm included) this results in a beam height \( h = 0.68 \times l \). This means that relatively high beams are used.

The main problem so far has been the choice for the strength of the beam elements \( f_{tm}, f_{tb} \) and \( f_{tm} \). In fact it would be better to specify the ratio's \( f_{ta}/f_{tb} \) and \( f_{tm}/f_{tb} \) because these determine the amount of disorder in the microstructure. Note that always the strength is tuned to a uniaxial tensile test on a standard laboratory specimen of \( 50 \times 60 \times 150 \) mm. This tuning procedure is essential, and has to be repeated always when a new type of lattice is used, for example when a random lattice based on a larger grid-size \( s \) (figure 3) has been selected. Up till now the procedure has been to use strength values for mortar and rock for the matrix and aggregates respectively. For the bond strength a low value was selected based on the macroscopic bond tests of Rehm et al. (1). These values had to be multiplied by a factor \( B \) in order to arrive at the same peak
strength in the above mentioned standard tensile test. In addition, it should be mentioned here that the tensile strength of, at least mortar and rock, is size dependent (28). An unrestricted use of macroscopic strength values is therefore probably not allowed, but because of a lack of relevant data this simplification has been used throughout this paper. In all, the physical basis for the limit strength parameters is still under debate. However, the large advantage of the new approach is that only single valued parameters have to be specified, which makes the derivation of the parameters from a tuning analysis on the standard tensile test inherently more simple. For more details concerning the determination of the model parameters, see refs. (18, 22). So far we have been successful in simulating fracture under combined tension and shear using the parameters tuned on a uniaxial tensile test (7, 22, 25, 29). In particular simulated curvilinear crack patterns are quite realistic; this has never been achieved using classical smeared crack models (11).

Obviously, many lattice elements have to be included in a calculation if the fracture response of concrete is to be mimicked in great detail. Therefore, in general, only the area of the specimen where cracks are expected to grow is modelled as a lattice, the remainder of the geometry is modelled with simple plane stress continuum elements that are available in the finite element package that is used to solve the model. Up till now most problems were treated in plane stress because of computational limitations. However the same procedure can be transferred directly to three dimensions. In this paper a simulation is included in which three dimensional behaviour is schematized as two parallel lattices connected by springs.

CRACKING IN CONCRETE

In the past few years the model has been tested extensively for concrete and sandstone laboratory scale specimens subjected to tensile or combined tensile and shear loading, see (7, 16-18, 22, 25, 27, 29). The great strength of all these simulations was that correct crack patterns could be found for many different loading cases, without changing the initially determined model parameters. In this paragraph some results of simulations of uniaxial tensile tests are shown. First some experimental feedback of the model will be discussed. Recently obtained experimental crack data will illustrate the capability of the model to mimic real crack behaviour of concrete.

Experimental Observations

As mentioned before, the amount of small particles in the material is of utmost importance for the softening tail in the load-crack opening diagram of a
specimen. In the tail of the softening diagram a specimen is completely cracked but stress-transfer is still possible because crack branches and overlaps remain intact. These so-called crack face bridges appear at various scales, and neglecting the small particles in the concrete mix is not allowed. Figure 4 and 5 show some examples of crack face bridging observed by using a vacuum impregnation technique (30), and by using an optical microscopy technique (31) respectively. In figure 4 a clear correlation between the maximum size of the aggregates and the amount of stress that can be transferred in the tail of the softening diagram is shown. In figure 4c the effect of small particles can be recognised. Quite some branching and bridging is visible, especially around small sand particles. Failure of a single crack face bridge, observed with a high resolution long distance microscope (QUESTAR QM-100) is shown in figure 5. Even when the cracks have localized still stresses can be transferred due to the intact ligaments between the two overlapping crack tips. It may be obvious that this process of crack face bridging is a three dimensional process. Therefore it is not only of great importance that details of the material are implemented as much as possible in the numerical model but it also highly recommended that these details are modelled in three dimensions. However, to model concrete including small particles (up to a diameter of 2 mm) it is necessary to use a lattice with very small beam elements (s=0.75 mm). This results in an enormous amount of beam elements. Due to computational efforts three dimensional modelling is not yet considered possible. The effects of small particles and three dimensional modelling will be demonstrated separately in the next sections.

Small Particle Effect

The influence of the amount of detail in the generated grain structured is demonstrated by performing two analyses of the uniaxial tensile test. In the first simulation a generated grain structure with aggregate particles between 2 and 8 mm was projected on top of the lattice of beams. The particle distribution was according a Fuller curve. The grain structure in the second simulation was kept exactly the same, only the particles between 2 and 3 mm were excluded from the mesh. In figures 6 and 7, the load-displacement diagrams and the crack patterns obtained in both the simulations are shown. The dimensions of the prism are 60 x 150 mm, with a thickness of 50 mm. The specimen is double notched at half height; the notches are 5 mm wide and 5 mm deep saw-cuts. The nodes at the bottom are supported in vertical and horizontal direction. A uniformly increasing vertical displacement is applied at the upper edge of the specimen, thereby keeping the bottom and upper edges parallel to each other during the complete loading to fracture. Only the part where cracks are expected to grow is modelled with the random lattice with a grid size s = 1 mm. The load-displacement curves are shown in figure 6, where they are compared to an experimental result. The boundary conditions in the test were
exactly the same as those in the simulations. For each of the simulations two crack patterns are shown, viz. the stages where 200 and 300 beams are removed. The respective locations are indicated in the load-displacement diagrams of figure 6.

In the analyses, a similar softening behaviour was found as in the experiment, except that the behaviour just beyond the peak is somewhat too brittle for the simulations. The effect of including smaller particles in the mesh is clearly visible: when more small particles are included a more 'ductile' response is obtained, especially in the tail of the softening diagram. The simulations indicate that around peak micro cracking takes place around the larger aggregate particles: both figure 7a and 7c indicate that debonding occurs first. Again this seems to be in agreement with experimental observations (30, 32). In the tail of the softening diagram, larger crack branches have developed in the specimen, but the prisms are still not separated in two parts. Intact material ligaments connect the two parts.

Three-Dimensional Effects

Another reason for the observed discrepancy is the fact that the three-dimensional fracture process in the laboratory test is simulated in two dimensions. This means that a crack in the simulations always must grow through the depth of the entire specimen. Obviously this does not occur in experiments (30). To study the effects on the global softening behaviour when a specimen is schematized in two dimensions, again two simulations were performed. In both analyses a regular lattice with a beam length of \( t = \frac{5}{3} \) mm was used. The lattice was part of a two dimensionally simulated specimen and the grain structure contained particles with a diameter between 3 and 8 mm. In the first simulation an attempt was made to show the influence of three dimensional effects. However to avoid three dimensional modelling, the problem was schematized as a pair of parallel meshes (figure 8). Two similar meshes, containing different grain structures were generated parallel to one another. Only in the area where cracks were expected to grow this parallel mesh was applied. The remainder of the specimen was modelled with 4 noded plane stress elements. To accomplish three dimensional stress transfer, each pair of nodes with equal x and y-coordinates was connected by spring elements (figure 8b). One spring element was attached for vertical stress transfer \( (k_y) \) and per pair also one horizontal spring \( (k_x) \) was used. An arbitrary stiffness of \( 10^4 \) N/mm was taken for all spring elements connecting the two lattices. The procedure for simulating fracture was identical as for a single lattice. For comparison a (traditional) two dimensional simulation with a single lattice was performed. The grain structure was taken the same as the structure of the front mesh of the parallel lattice.

The stress-crack-opening diagram for the parallel simulation shows a much
more ductile behaviour as compared to the single-mesh simulation (figure 9). In
the tail of the diagram a higher load can be transferred which seems to be in
more agreement with experimental load-deflection curves. The higher load
transfer in an advanced stage of the fracture process can also be recognised in
the corresponding crack patterns (figure 10). For each load step the two parallel
meshes are plotted separately. On the left the mesh at the front face is shown
including the remainder of the specimen. The second mesh representing the
back face of the specimen is shown to the right. It can be seen that around
peak mainly micro cracking around the larger particles occurs. As soon as the
crack has localized still stresses can be transferred by the intact ligaments in­
plane but also through the out-of-plane spring elements which connect both
meshes. Even when both meshes are almost separated (step 650, figure 10b)
still half of the maximum stress (approximately 1.5 MPa) can be transferred.
Crack face bridges between the two separate meshes can be observed in the
middle of the specimen (figure 10b).

ANALYSIS OF BOND-SLIP-LAYER FRACTURE

Description of the Problem

Good bond between concrete and steel is essential when reinforced concrete is
used. Slip in the bond-zone is often seen as a combination of adhesion, fric­
tional stress transfer and mechanical interlock of the ribs (33). In terms of the
lattice model presented earlier, it can be stated that bond between steel and
matrix it basically the same as bond between aggregate and matrix. Mutual
differences in behaviour are expressed by different strength and stiffness of the
beam elements in the model. According to Goto (8) failure of ribbed rebars
occurs through the growth of internal cracks among cracks that are visible at
the surface of the concrete. Experiments (figure 11) show that internal cracks
are formed near the compressive side of a lug at angles of about 60° to the bar
axis. Vos (33) was among the first to study this local crack behaviour near steel
reinforcement bars, using axisymmetric simulations of a single rib. A single lug
of a reinforcement bar was schematized and modelled. The simulations of Vos
have recently been repeated with the lattice model (34). The results of both the
original simulations performed by Vos (33) and the simulations with the lattice
model indicate that the failure mechanism depends strongly on the boundary
conditions. However because the simulated geometry is strictly fictitious it is
very difficult to extrapolate results to any real situation. Comparison is impos­
able, not with any experiment nor with any practical situation. As mentioned in
the introduction, a detailed bond test was developed at the Institute for Building
Research (TNO) in order to define the effective properties of the (cracked)
bond-layer (13-15). In these local bond tests the assumption was made that slip
is mainly caused by micro cracking in a small zone around the rebar. An
axisymmetric concrete specimen was developed contained in a hollow aluminium tube with a centred rebar. The rebar was either ribbed or smooth with a diameter of 16 mm. The thickness of the concrete layer was taken equal to the radius of the rebar and resembles the slip layer around the rebar. The aluminium tube with a thickness of 6.3 mm represents the confinement of the surrounding concrete which is assumed to be uncracked. With the lattice model this situation has now been analyzed. However we had to confine ourselves to a plane stress simulation, because a full fledged three dimensional analysis would require a too large computational effort. The two dimensional mesh used for the numerical simulation for rebar and tube is given in figure 12a. To obtain perfect bonding between concrete and tube, a groove is created at the inner side of the aluminium tube. A cross sectional view of the tube used in the experiment is given in figure 12b.

Considerable attention is given in modelling the axisymmetric experiment. To obtain the same radial stiffness in the plane stress simulations as in the experiments springs were attached at the outer side of the tube. The stiffness of the springs was determined using the tube geometry and linear elastic properties of the tube that was used in the tests. For the simulations of the concrete part of the specimen a regular lattice was used (figure 12c). Particles with a minimum diameter of 2 mm were projected on top of the lattice. The maximum diameter was taken a quarter of the diameter of the rebar, i.e. 4 mm. To model the tests with high accuracy a small beam length was taken (0.75 mm). Especially the groove in the tube and the ribs of the rebar made it necessary to use a lattice with such a small beam size. The boundary conditions are also given in figure 12a. Loading was achieved for \( F_2 = 0.5 \times F_1 \) and \( R_A + R_B = 0.5 \times F_1 \). The stress and displacement measurements in the experiments were not taken axisymmetric in contradiction with the geometry of the specimen. To mimic the situation as good as possible deformations were measured at 30 different nodes in the finite element mesh. Five nodes at regular distance were selected at the outside of the tube and the transition between tube and concrete and between concrete and rebar. Four different situations were examined to qualify the bond slip behaviour. First simulations of a ribbed and smooth rebar were performed where no special strength was assigned to the beam elements in the interfacial zone, i.e. the transition between concrete and rebar. Thereafter also the case was examined where an extremely low interfacial (or adhesive) strength was used, i.e. \( f_I = 0.5 \) MPa. Note that the interfacial zone has a rather high porosity, and also the stiffness of the interfacial zone should be reduced. Here we only show the effect by reducing the interfacial strength.

Numerical Results

The results of the detailed tests presented by Dragosavić (13-15) are rather limited. The failure mechanism is hard to study because of the presence of the
tube which encloses the whole concrete specimen. Therefore conclusions were
extracted only from the various stress-deformation relationships. The horizontal
and vertical stress was plotted as a function of the horizontal and the vertical
displacement of the bond zone. However because the extensometers were
attached to the tube surface no direct measurements were taken at the alumin­
ium tube interface. The experimental results (14, 15) show very unstable
behaviour of all load-displacement curves. The maximum radial bond stress
measured in the experiments was 1 and 6 MPa for the smooth bar and ribbed
bar respectively when a tube of 6.3 mm was used. For a diameter of the rebar
of 16 mm and a length of the bond zone of 50 mm this results in a force (F₂-
F₁) of 5 and 30 kN respectively.

When the results of the simulations are compared with the experiments, only a
very little resemblance is found. Two explanations can be given. First the
experiments were tested under cyclic loading which is not possible with the
lattice model. The computed load-displacement curves can therefore be com­
pared only with the envelope curve of the cyclic experiment. The second point
is mentioned earlier and concerns the deficiency of the predicted relation
between the applied load and displacements. Both points make that precaution
has to be taken in drawing any definite conclusions. However mutual compari­
on between the various simulations remains very well possible. Because the
grain structure is kept equal for all simulations, differences in strength of the
interfacial zone as well as the presence of the ribs at the rebars can be studied.
Figure 13 shows the crack pattern of each simulation in an advanced stage of
the fracture process. To emphasize the crack pattern in the lattice of beam
elements no grain structure was plotted in figure 13 and 14. The rib geometry
is shown schematically in these figures. The grain structure which was used is
shown in figure 12c and was not changed during the simulations. When no
interfacial strength is assigned (figure 13a and 13b) not very much differences
can be observed in the crack patterns of the smooth and the ribbed rebar. The
adhesive strength is quite large (between 1.25 and 10 MPa) and seems to
override the effect of the lugs. These inclined cracks seem to nucleate every­
where along the bar irrespective of the location of the lugs. The angle of the
cracks with the rebar is approximately 60°.

Because the strength of the interface elements seems to play a major role, also
simulations were performed where a low adhesive strength (f₁=0.5 MPa, see
figure 13c and 13d) was assigned to the beam elements connected to the steel
rebar. The behaviour during fracturing is as follows: first the concrete is torn
loose from the rebar, compressive struts are formed near the lugs of the ribbed
reinforcing bar and continuous cracks are formed parallel to these struts (figure
14). When a smooth bar is used, the bar is separated almost immediately but
now hardly any diagonal cracks are formed. The angle of the cracks found in
the simulations corresponds with the findings of Goto (8) and Otsuka (9). It is
mentioned again that the present example is just indicative. The adhesive
strength is an rather arbitrary chosen value. A more thorough parameter study
of the interfacial properties should be carried out to arrive at more definite results. The simulations demonstrate however perfectly clear the essence of the bond-slip behaviour. The global behaviour of a reinforcing bar depends strongly of the micro cracking near the interfacial zone between concrete and rebar. Another important point concerns the simulations of one single lug (33, 34). In such simulations a slip-layer section was defined. The assumption was made that only the the outer surrounding concrete influences the failure mechanism. The fact that also the neighbouring ribs strongly influence each other was not taken into consideration.

The influence of the adhesive strength is shown in the load-displacement responses shown of figure 15. In this figure the load $F_1$ is plotted versus the average lateral deformation. The lateral deformation is calculated as the displacement of the inner side of the aluminium tube minus the displacement of the transition between concrete and rebar. Hardly no differences are observed between a smooth and ribbed rebar when no interfacial strength is specified. However when an adhesive strength of 0.5 MPa is used for the interfaces the behaviour of the smooth rebar becomes considerably more ductile (figure 15b). The fact that the curves in figure 15b are interrupted at a low lateral deformation is owing to the interfacial properties of the beams in these simulations. At this stage almost all elements at the interface of concrete and rebar are torn loose.

**CONCLUSIONS**

In this paper crack growth simulations with a recently developed lattice model are presented. It was shown that crack face bridging is one of the main mechanisms of the softening behaviour in concrete tensile specimens. The amount of detail which is included in the lattice model influences the amount of crack face bridges that can be obtained. Including more detail leads to a more ductile stress-crack-opening diagram. Furthermore it was shown that modelling the three dimensional fracture process in two dimensions will lead to major losses of detail in the load displacement response. When a two dimensional mesh is used cracks are assumed to grow through the entire depth of the specimen. By using two parallel meshes connected with springs it was shown that out-of-plane crack face bridging has a considerable influence on the ductility of the specimen. Both the small particle effect and the three dimensional effect should not be neglected in numerical simulations of fracture. If results are evaluated the consequences of these two effects should always be considered.

The model was successfully applied for the simulation of bond-slip behaviour of ribbed rebars. The cracking of the concrete layer near the lugs on the rebars is in agreement with observations by Goto (8). The inclination of the cracks that nucleate near the lugs is strongly dependent on the amount of confinement
on the interfacial zone. This confinement is either caused by the concrete surrounding the interfacial zone, or alternatively by some external load. The results of the bond-slip simulations indicate that the lattice model can be used for analyzing macroscopic bond-slip behaviour between steel and concrete. However, it should be kept in mind that boundary condition effects play a major role on bond-slip behaviour. A reliable (macroscopic) bond-slip model should therefore include these effects.

An alternative approach may (in the near (?) future) be to model the bond-slip behaviour at the meso-scale. The advantage of the 'brittle' lattice model is that only single valued parameters are needed for simulating realistic crack phenomena. It is the author's opinion that a good model for interface behaviour should describe the physics of the problem as accurately as possible. The lattice model is a first step towards such an approach.

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Fig. 1—a) Regular square lattice after Herrmann (21); b) Regular triangular lattice; and c) Triangular lattice with randomly distributed beam lengths.

Fig. 2—a) Particle overlay; and b) Assigning different properties to beam elements falling in either of three phases.

Fig. 3—Construction of lattice with random beam length.
Fig. 4—(a) Load-crack opening diagrams for three different concretes: 2 mm mortar, 12 mm lytag and 16 mm concrete; (b) Crack face bridging at 100 μm in 2 mm mortar (curve No. 1); (c) in 12 mm lytag (curve No. 2); and (d) in 16 mm concrete (curve No. 3) (31)

Fig. 5—Failure of ligament between two overlapping cracks observed with an optical microscope (17)
Fig. 6—Stress-crack opening diagrams in uniaxial tension. Comparison between numerical simulations with and without particles between 2 and 3 mm, and an experiment on normal concrete with 8 mm maximum aggregate.

Fig. 7—Crack growth for two simulations of Fig. 6: a), b) Simulation without particles smaller than 3 mm; and c), d) With particles between 2 and 3 mm included. The two initial stages a), c) are the crack patterns at 200 beams removed; b) and d) are the crack patterns when 300 beams are removed.
Fig. 8—Parallel modeling: a) Overview of two parallel meshes; and b) Connecting of a pair of nodes by spring elements

Fig. 9—Stress-crack-opening diagrams of simulations with a single mesh and a parallel mesh compared with an experiment.
Fig. 10—Crack patterns for two different loading levels using two parallel meshes. a) 450 beams removed; and b) 600 beams removed.

Fig. 11—Photograph of internal cracking around reinforcing steel, after Goto (8)

Fig. 12—a) Finite element mesh; b) Cross section; and c) Lattice model with generated grain structure used to simulate the detailed experiments (14, 15)
Fig. 13—a), b) Crack patterns for smooth and ribbed rebar where no interfacial strength is specified (i.e., it can vary between 1.25 and 10 MPa); and c), d) For an interfacial strength of 0.5 MPa.

Fig. 14—Crack history for a ribbed rebar with an adhesive strength of 0.5 MPa for beam elements in the interfacial zone. The numbers between brackets indicate number of broken beams.

Fig. 15—Load-slip diagrams of local bond simulations. a) With interface strength equal to concrete strength (1.25 ≤ $f_i \leq 10$ MPa); and b) Interface strength of 0.5 MPa.
Fracture Mechanics Analysis of Bond Behaviour under Dynamic Loading

by C. Yan and S. Mindess

Synopsis: The bond between reinforcing bars and concrete under impact loading was studied both experimentally and by the finite element method. The experiments consisted of pull-out tests and push-in tests, under three different types of loading: static, medium rate, and impact. Different concrete strengths (normal and high), types of fibres (polypropylene and steel), and fibre contents were considered. The study focused on the bond-slip relationships, and the fracture energy in bond failure. The experimental results were compared with those obtained by the finite element method, in which a special “bond-link element” that was able to transmit both shear and normal forces was adopted to model the connection between the rebar and concrete. It was found that higher loading rates, higher concrete compressive strengths, and the addition of steel fibres had significant effects on the bond resistance, the fracture energy and the bond stress-slip relationship, especially for the push-in case. Reasonably good correspondence in the results between the two methods was also found, and a bond-stress-slip relationship under high rate loading could be established analytically.

Keywords: Bond (concrete to reinforcement); fiber reinforced concretes; finite element method; fracture mechanics; fracture properties; impact; loads (forces)
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ACI Fellow Sidney Mindess is a Professor of Civil Engineering at the University of British Columbia, where he has taught since 1969. He is a member of the ACI Committees 370 and 446. He is also a member of the Co-ordinating Committee of RILEM.

INTRODUCTION

The behaviour of a reinforced concrete structure is strongly dependent upon the bond between the concrete and the reinforcing bars. The prediction of the linear or nonlinear response of reinforced concrete structures subjected to static or dynamic loads, regardless of the method of analysis, is based upon our knowledge about the local bond stress vs. slip relationship governing the behaviour at the steel-concrete interface.

While there is extensive literature on static bond tests, there is little experimental work on the bond between concrete and steel reinforcement under dynamic loading (Isenberg et al. [1]). The bond behaviour under dynamic loading is quite different from that under static loading, and involves more complex mechanisms. Most of the reported experimental results for impact loading [2-6] show that the shearing mechanism (rib bearing against the concrete) is the main mechanism for the bond between deformed bars and concrete, and that the concrete strength, the loading rate, and the presence of reinforcement (either in the form of fibres or of continuous bars) have a great effect on the bond behaviour. These effects may result from the strengthening of the material surrounding the rebar, the increasing crack resistance, the strain rate sensitivity of the materials, the non-uniform strain distribution along the reinforcing bar, or other energy absorbing mechanisms.

Previous studies regarding the application of the finite element method to the bond problem simply introduced the load bond stress-slip relationships which were obtained from tests. Theoretically, there is a unique relationship between bond stress and slip at the interface between a steel bar and concrete for which the geometric and mechanical properties are known. The problem can be solved by reasonably modelling the mechanical properties at the interface between the rebar and the concrete, as well as the
constitutive laws for both materials and appropriate cracking and crushing criteria.

The present paper describes the results of both an experimental study and a finite element technique in which the bond behaviour between rebars and concrete under impact loading was studied.

**EXPERIMENTAL PROCEDURES**

**Specimens**

The test specimens were concrete prisms 152.4x152.4x63.5 mm, containing a centrally loaded 11.3 mm diameter (No. 10) deformed reinforcing bar. Two concentric 6.35 mm steel spirals, 63.5 mm and 127.0 mm in diameter, were also cast in the prisms (Fig. 1). Their purpose was to prevent splitting of the concrete, and thus a pure pull-out or push-in bond failure could be achieved. Three types of concrete were tested: plain concrete, polypropylene fibre reinforced concrete (with fibre contents of 0.1% and 0.5% by volume) and steel fibre reinforced concrete (with fibre contents of 0.5% and 1.0% by volume). The polypropylene fibres were fibrillated fibres (40.0 mm long, 0.05 mm diameter)\(^1\). The steel fibres had hooked ends (30.0 mm long, 0.5 mm diameter)\(^2\). The basic mix designs are given in Table 1 for low compressive strength concrete (about 40 MPa) and high compressive strength concrete (about 75 MPa). The maximum aggregate size was 10 mm.

About one-quarter of the reinforcing bars were instrumented with five pairs of electric resistance strain gauges to measure the strain distribution along the rebar. These pairs of strain gauges were mounted on diametrically opposite sides of each test bar at a spacing of 15.9 mm (centre to centre) to take care of the bending effect, if any, in the test bar during loading.

**Impact Tests**

The impact tests were carried out using a large, instrumented drop weight impact machine, which was designed and constructed at the University of British Columbia. The details of this machine have been presented elsewhere [7,8]. Briefly, it is capable of dropping a mass of 345 kg\(^3\) from

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1 Produced by Fibermesh Corporation, Chattanooga, Tennessee, U.S.A.
2 Produced by Bekaert Corporation.
3 It has since been modified to provide a capacity of 505.0 kg.
heights of up to 2.5 m on the target specimen, giving a kinetic energy of up to 8450 Nm. Two accelerometers mounted on the falling mass and the rebar, respectively, monitored their accelerations; a strain-gauged bolt load cell monitored the load on the specimen. The static and medium rate tests were carried out in an Instron universal testing machine. By appropriately setting the crosshead speed of the universal testing machine, or altering the drop height of the hammer of the impact machine, three different ranges of loading rates could be achieved to induce a wide range of bond stress rates. They were: static, medium, and impact rates. The equivalent ranges of the bond stress rates are listed in Table 2.

The specimens were supported by a steel base (200 mm in diameter and 100 mm in height) with a 35 mm diameter hole in its centre. They were pushed (for static and medium rate loading) or struck (for impact loading) at the top of the rebar. In the case of pull-out tests a solid steel frame with a stiffness of 15 times that of the reinforcing bar was used to apply a pull-out force.

The load, acceleration and strain data were recorded at 200 μs intervals by a PC-based, 16-channel high speed data acquisition system. A high speed video camera (EKTRAPRO 1000 Motion Analyzer)\(^4\), which can take 1000 to 6000 frames per second, was also used to take a video of the specimen during the impact event. By analyzing the videos frame by frame, the calculated displacements could be verified. Figure 2 shows a schematic of the set-up for the experiments. The details can be found in reference [9].

**REDUCTION OF TEST DATA**

After signal processing, the applied load, the accelerations of both the hammer and the reinforcing bar, and the strains along the rebar could be calculated from the data acquired, using linear calibration curves. These data were all obtained as a function of time.

The displacements of the hammer and the rebar were found by integrating the recorded accelerations over time. The axial stresses in the rebar and in the concrete were calculated from the recorded strain data. Using the dynamic equilibrium condition, the local average bond stresses and slips along the rebar were calculated; the fracture energy in the entire bond-slip

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\(^4\) Manufactured by Eastman Kodak Company, U.S.A.
process was calculated as the work done by the bond stress during slip. Detailed equations for the above calculations can be found in reference [9].

RESULTS AND DISCUSSIONS

Bond Stress-Slip Relationship by Experiments

It is clear that for all of the bond tests, the bond stress-slip relationships kept changing with time under dynamic loading; in other words, there were different relationships between the bond stress and the slip at different stages of loading. For simplicity, all of the bond stress-slip relationships were referred to the moment at which the bond stress reached the peak value, and these relationships were then averaged over the embedment length, unless otherwise specified.

Figures 3 to 5 show several bond stress-slip relationships obtained by experiments. It can be seen from these figures that there were always a greater bond stress for the high strength concrete, especially under higher rate or push-in loading (impact), or for specimens with steel fibres). The polypropylene fibres seemed to have much less effect in this regard than the steel fibres.

The shear mechanism is the main mechanism for the bond resistance of deformed bars, under either static or dynamic loading [9,10]. The force transferred by the concrete surrounding the rebar increased with an increase in the shear strength of the concrete, which is, to some extent, proportional to the compressive strength. The stress rate sensitivity of concrete has been reported by numerous investigators (e.g. [11, 12, 13]), and explained on the basis of fracture mechanics [14]. Steel fibres increased the load carrying capacity in the area surrounding the rebar, especially in the post-cracking region [15,16].

In the case of pull-out tests, when the bond stress reached the critical value, a longitudinal tensile stress and a radial tensile stress (tending to cause separation), combined to produce the first internal cracks from the tops of the ribs, because of the stress concentrations at these locations. With a further increase in external loading, the Poisson effect in the steel resulted in a decrease in the bar diameter, and the contact area between the concrete and the ribs of the deformed bar was reduced. This would increase the bearing stress between the concrete and the ribs, and enhance crack development around the tip of each rib. Generally speaking, the combination of the above-mentioned longitudinal and radial tensile stresses
was quite large so that the high strength concrete was not much better than the normal strength concrete in inhibiting first cracking.

For the push-in tests, the stress transfer mechanism involved was quite different from that in the pull-out tests. The force in the rebar deformed the concrete inwards (in the direction of the force). This served to tighten the concrete around the bar and increased the frictional resistance between the concrete and the rebar. The slight increase in the diameter of the rebar due to the Poisson effect also improved the frictional resistance [17]. The stresses in the concrete, thus, were generally less for pull-out tests than for push-in tests. A small zone of concrete was subjected to compression-tension-tension in the radial, longitudinal and circumferential directions, respectively. However, few cracks were found after slicing the specimens. The inward deformation of the concrete provided some lateral compression in the concrete surrounding the bar, and thus reduced the radial component of the wedging force. All of this contributed to the great influence of concrete strengths, high loading rates, push-in loading, and steel fibre additions on the bond strength.

Fracture Energy

The fracture energy results for different types of specimens are presented in Table 3. It can be seen that specimens made of high strength concrete, or steel fibre reinforced concrete and specimens under high rate loadings, or push-in loading always absorbed more fracture energy (about 2.5% to 6.7%). For reinforced concrete structures it is essential that the bond between the reinforcing bar and the concrete exhibit a certain “ductility” during dynamic loading. That is, the bond resistance in the member should decrease gradually instead of suddenly failing, so that the dynamic energy can largely be transferred, absorbed and dissipated to the entire structural member over a relatively long time period. This bond ductility may be represented by the fracture energy, which is calculated as the work done by the bond stress. A larger value of fracture energy means a more “ductile” bond.

Comparison with Dynamic Finite Element Method

In the finite element analysis, using fracture mechanics concepts, the chemical adhesion, frictional resistance and the shear mechanism were all taken into account. Twenty node quadratic solid isoparametric elements were employed for both steel and concrete before cracking. After cracking the concrete elements were replaced by quadratic singularity elements capable of modelling curved crack fronts. A special “bond-link element”, which was able to transmit both shear and normal forces, was adopted to
model the connection between the rebar and concrete. A new approach was proposed for the establishment of the stiffness matrix of the "bond-link element". The dynamic constitutive laws of both steel and concrete, the criteria for crack formation and propagation in concrete, based on the energy release rate theorem, and the criterion for concrete crushing were used in the finite element process. Details can be found in reference [9].

The calculated results showed that at very low levels of the steel stress (about 30-40 MPa) the chemical adhesion between the rebar and the concrete was destroyed, and for the case of pull-out loading the frictional resistance reduced rapidly with the separation between the rebar and the concrete when the steel stress increased. At that point, the rib bearing became the main factor providing resistance in the bond process. These calculations seem to agree well with the experimental results.

It was found from the finite element analysis that at a relatively low level of applied load, the distribution of the stress in the rebar was not much different from that obtained by the experimental method. With further increases in the applied load, however, the differences in the distributions between the two methods increased. That is, the finite element analysis became increasingly sensitive to the various parameters adopted in the analysis, mainly the nonlinear dynamic constitutive relation of the concrete and the damage mechanism. The results of the experimental method were obtained directly from the strain gauge measurements and are considered to be more reliable. Using more accurate parameters in the finite element analysis would result in very good prediction for the bond behaviour at the interface between rebar and concrete.

It was also found that relatively high values of principal tensile stresses developed in the concrete in the vicinity of the tips of the ribs, especially for the pull-out case, which indicated that the secondary cracks would form first. For the plain concrete and the polypropylene fibre concrete, some crushing of the concrete also took place at the tips of the ribs. This resulted in a great decrease in the bond strength, or, from the viewpoint of energy, in the capacity of energy transfer. On the other hand, there was seldom crushing in the concrete for the steel fibre concrete. This may help to explain why the specimens made of plain concrete and polypropylene fibre concrete consumed much less fracture energy during the entire bond-slip process.

The calculated results also indicated that there were more cracking elements for the steel fibre concrete than for the plain and polypropylene fibre concrete. Because of this, the bond slips in the former case were always found to be larger than in the latter cases in the calculations, which, in turn,
made the fracture energy for the steel fibre concrete much larger than for the other types of concrete. This is also in agreement with the experimental results.

As expected, the bond strength and the fracture energy for push-in loading were found to be greater than for pull-out loading. This indicates that by adopting the 3-dimensional elastic matrix in the constitutional law, the Poisson effect was properly considered, and that the modelling of the frictional resistance at the contact surface between the rebar and the concrete by the “bond-link element” was reasonable.

Two examples of the bond stress-slip relationships determined by the finite element method are given in Figures 6 and 7, which represent the cases of polypropylene fibre reinforced concrete under medium rate loading (bond stress rate = $0.5 \times 10^{-5} - 0.5 \times 10^{4}$ MPa/s) and steel fibre reinforced concrete under impact loading (bond stress rate = $0.5 \times 10^{2}$ MPa/s), respectively. In these figures the curves from experiments are also given for comparison.

The shapes of the curves obtained by the finite element method are different from those from the experimental measurements. There is only a very small linear portion from the beginning of the loading in those curves obtained by the finite element method. This may be because for the finite element models the chemical adhesion is destroyed at a very low level of loading, and the contribution of the frictional resistance to the bond strength depends on the calculated stress state at the interface to a great extent. Both the peak and average bond stress are larger for the analytical than for the experimental results. From the viewpoint of mechanics, the models of the finite elements make the specimen more “rigid”, i.e. its stiffness becomes larger even though the modelling of the chemical adhesion and the frictional force may lessen the stiffness of the interface between the rebar and the concrete to some extent. The increase in the bond resistance and the relatively smaller local slip corresponding to the same bond stress may also attribute to this.

**CONCLUSIONS**

1. High loading rates increase the bond strength and fracture energy during bond failure, especially for the push-in loading case. The bond stress-slip relationships under impact loading is quite different from that under static loading.

2. The high strength concrete, or steel fibre reinforced concrete exhibit higher bond strength and absorbs more fracture energy in the bond
process under impact loading. Their bond stress-slip relationships are also quite different from those for normal strength concrete, or plain concrete.

3. A finite element method based on fracture mechanics with appropriate interface modelling can be used to solve the bond problem. The bond stress-slip relationship can be established analytically.

ACKNOWLEDGEMENTS

This work was funded by the Natural Sciences and Engineering Research Council of Canada, through the Network of Centres of Excellence on High Performance Concrete.

REFERENCES


### TABLE 1 — BASIC CONCRETE MIX DESIGN (PER m³)

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Normal Strength (40 MPa)</th>
<th>High Strength (75 MPa)</th>
</tr>
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<tbody>
<tr>
<td>Type 10 Portland Cement (kg)</td>
<td>343.0</td>
<td>513.0</td>
</tr>
<tr>
<td>Silica Fume (kg)</td>
<td>---</td>
<td>98.0</td>
</tr>
<tr>
<td>Sand (&lt;4.75 mm) (kg)</td>
<td>686.0</td>
<td>958.0</td>
</tr>
<tr>
<td>Aggregate (4.75−10.0 mm) (kg)</td>
<td>1200.00</td>
<td>635.00</td>
</tr>
<tr>
<td>Water (kg)</td>
<td>171.5</td>
<td>201.0</td>
</tr>
<tr>
<td>Air Entraining Admixture (ml)</td>
<td>70.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Superplasticizer* (ml)</td>
<td>0 ~ 1715</td>
<td>1715 ~ 6110</td>
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</table>

**Fibre Additions**

<table>
<thead>
<tr>
<th>Types of Fibres</th>
<th>By Volume</th>
<th>By Weight (kg)</th>
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<tr>
<td>Polypropylene</td>
<td>0.1%</td>
<td>0.9</td>
</tr>
<tr>
<td>Fibres</td>
<td>9.5%</td>
<td>4.5</td>
</tr>
<tr>
<td>Steel</td>
<td>0.5%</td>
<td>39.0</td>
</tr>
<tr>
<td>Fibres</td>
<td>1.0%</td>
<td>78.0</td>
</tr>
</tbody>
</table>

*The amount varied from 0.0~10.0 ml per kg of cement.*
<table>
<thead>
<tr>
<th>Loading Type</th>
<th>Bond Stress Rate (MPa/s)</th>
</tr>
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<tbody>
<tr>
<td>Static</td>
<td>0.5\times10^{-8} \sim 0.5\times10^{-6}</td>
</tr>
<tr>
<td>Medium</td>
<td></td>
</tr>
<tr>
<td>I (Low)</td>
<td>0.5\times10^{-6} \sim 0.5\times10^{-5}</td>
</tr>
<tr>
<td>II (High)</td>
<td>0.5\times10^{-5} \sim 0.5\times10^{-4}</td>
</tr>
<tr>
<td>Impact</td>
<td></td>
</tr>
<tr>
<td>I (Low)</td>
<td>0.5\times10^{4}</td>
</tr>
<tr>
<td>II (Medium)</td>
<td>0.5\times10^{-3}</td>
</tr>
<tr>
<td>III (High)</td>
<td>0.5\times10^{-2}</td>
</tr>
</tbody>
</table>
### TABLE 3 — FRACTURE ENERGY IN BOND FAILURE

<table>
<thead>
<tr>
<th>Type of Concrete</th>
<th>Pull-out Tests (Nm)</th>
<th>Push-in Tests (Nm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal</td>
<td>High</td>
</tr>
<tr>
<td>Plain Concrete</td>
<td></td>
<td></td>
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<tr>
<td>Polypropylene</td>
<td>40.2</td>
<td>42.1</td>
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<td>Fibre Concrete</td>
<td>40.6</td>
<td>42.4</td>
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<tr>
<td>Steel</td>
<td>40.7</td>
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<tr>
<td>Fibre Concrete</td>
<td>54.6</td>
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<td>1.0% Polypropylene</td>
<td>71.5</td>
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<td>0.1% Polypropylene</td>
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<tr>
<td>0.5% Fibre Concrete</td>
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<td>44.1</td>
</tr>
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<td>58.2</td>
<td>63.5</td>
</tr>
<tr>
<td>0.5% Steel</td>
<td>60.3</td>
<td>64.2</td>
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<td>1.0% Fibre Concrete</td>
<td>75.9</td>
<td>80.7</td>
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<tr>
<td>0.5% Polypropylene</td>
<td>45.2</td>
<td>47.4</td>
</tr>
<tr>
<td>0.1% Polypropylene</td>
<td>45.4</td>
<td>47.7</td>
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<tr>
<td>0.5% Fibre Concrete</td>
<td>45.5</td>
<td>47.8</td>
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<tr>
<td>1.0% Fibre Concrete</td>
<td>63.9</td>
<td>68.5</td>
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<tr>
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<td>79.5</td>
<td>85.4</td>
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<tr>
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<td>47.4</td>
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<td>0.5% Polypropylene</td>
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<tr>
<td>0.1% Polypropylene</td>
<td>48.0</td>
<td>50.5</td>
</tr>
<tr>
<td>0.5% Steel</td>
<td>65.8</td>
<td>71.4</td>
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<tr>
<td>1.0% Fibre Concrete</td>
<td>87.9</td>
<td>93.4</td>
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<td>50.2</td>
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<tr>
<td>0.1% Polypropylene</td>
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<td>52.5</td>
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<tr>
<td>0.5% Fibre Concrete</td>
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<tr>
<td>0.5% Steel</td>
<td>70.9</td>
<td>76.5</td>
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<tr>
<td>1.0% Fibre Concrete</td>
<td>94.6</td>
<td>100.5</td>
</tr>
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</table>

*S - Static  M - Medium  I - Impact*
Fig. 1—Specimens for bond tests
Fig. 2—Schematic of set-up for impact test
Fig. 3—Bond stress-slip relationship — Different concrete strengths and loading rates

Fig. 4—Bond stress-slip relationship — Different fiber additions and loading rates

Fig. 5—Bond stress-slip relationship — pullout and push-in
Fig. 6—Bond stress-slip relationship by finite element method (polypropylene fiber reinforced concrete, medium rate loading — $0.5 \times 10^{-5}$ to $0.5 \times 10^{-4}$ MPa/s)

Fig. 7—Bond stress-slip relationship by finite element method (steel fiber reinforced concrete, impact loading — $0.5 \times 10^{2}$ MPa/s)
Prediction and Verification of Interface Debonding for Fiber Reinforced Cementitious Material

by S. H. Li, S. P. Shah, Z. Li, and T. Mura

Synopsis:
A new method to predict the debonding behavior of fiber-matrix interface has been proposed by applying the principles of the micromechanics of inclusion and fracture mechanics. The validity of the mathematical model is further verified by uniaxial tension tests carried out on steel fiber reinforced cementitious composite specimens by employing a digitally controlled closed-loop MTS testing machine. It is demonstrated that the debonding occurs before the bend over point and the debonded lengths are largely influenced by the sequence of the occurrence of transverse matrix cracks and the loading stage. A stable growth of debonding has been observed in the investigation. The measured debonded lengths are compared with the theoretical prediction of the proposed model. A reasonable agreement is observed.

Keywords: Cracking (fracturing); debonding; fiber reinforced concretes; fluorescence; fracture mechanics; interface; mathematical models; microscopy; tension
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**INTRODUCTION**

The mechanical properties of monolithic-brittle materials can be improved by high strength ductile fibers. Enhanced stress capacity and multiple matrix cracking have been observed from the stress-strain curve of the uniaxial tensile test for steel, glass and synthetic fiber-reinforced cementitious composites[1]. The toughness enhancement of the matrix can be largely attributed to the interfacial debonding, frictional shear resistance along the fiber-matrix region and fiber bridging effect in the cracks.

To better understand the toughening mechanism of the composites, a significant research activity has been aimed at characterizing fiber debonding. Most of these activities are concentrated on the fiber debonding and pull-out problem since it is generally agreed that this problem can be used to represent the behavior of the fiber in the composites. There are two theoretical approaches
which are being used to interpret material parameters for the fiber debonding and pull-out type of experiments. Based on a maximum shear strength criterion debonding takes place when the maximum shear stress at interface reaches a critical value. This approach is typified by the work of Lawrence [2], Gopalaratnam and Shah[3], Naaman et. al[4], Greszczuk [5], Takaku and Arridge[6] and Marshall and Oliver[7].

An alternative approach is a formulation based on the fracture mechanics principles using an energy release rate criterion. Such formulation is based on the assumption that the propagation of the debonded zone requires a certain energy and that debonding will occur only when the energy flowing into the interface exceeds the value of the specific resistance energy. Fracture mechanics approaches include those of Gurney and Hunt[8], Outwater and Murphy[9], Bowling and Groves[10], Atkinson et al.[11], Stang and Shah [12][13], Gao et al. [14] and Wells and Beaumont[15], Hutchinson and Jenson[16] and Nair[17].

Wells and Beaumont [15] analyzed the fracture of fibrous composites and proposed a model enabling the debonded lengths and pull-out to be calculated under non-uniform stress. They concluded that the energy absorption in composites was dependent on the length of debonding and pull-out. Marshall and Oliver[7] used the experimental measurement (fiber push-out test) to analyze interfacial debonding and frictional sliding. Hutchinson and Jensen [16] used approximate closed-form solutions to model the debonding and pull-out with two idealizations of friction: constant friction and Coulomb friction. Campbell et al.[18] used transmission electron microscopy to observe the interfacial failure for the SiC-whisker toughened Al₂O₃ and Si₃N₄ materials. The role of interface in ceramic matrix composites was recently studied by Evans et al.[19]. Fracture surfaces were examined in a scanning electron microscope, and numerical and analytical crack growth simulations were compared with experimental results by Zok et al.[20].

In this paper, focusing on the continuous longitudinal fibers, the debonding properties of fiber reinforced cementitious composites were analyzed. The principles of the micromechanics of inclusion and the concept of energy balance were employed. Multiple cracking of the matrix was considered and formulae for calculating the debonding length associated with each matrix transverse crack occurring at different loading stages were derived.

In the present study, an attempt is also made to measure the interfacial debonding directly from the specimen under the uniaxial tension test. To observe the internal debonding phenomena, the specimens were first loaded to a predetermined displacement magnitude and then the deformation in the specimens was "frozen" by gluing rigid steel blocks onto the specimen. After unloading the specimen, the technique of optical fluorescence microscopy was applied to acquire the extent of debonding length from a thin sectioned samples obtained
by cutting the "frozen" specimen. After that, the debonding phenomena were examined using the technique of optical fluorescence microscopy. Finally, the experimental results are compared with the present theory and other published predictions.

**MATHEMATICAL MODEL**

A stage of the composite after BOP is depicted in Fig. 1. A second crack of length 2a has formed at a distance of \(l_c + l_d^{(o)}\) from the first complete matrix crack. Note that the length \(l_c\) is determined by subtracting the extent of debonding \(l_d^{(o)}\) of the first crack from the crack spacing. It is assumed that a constant frictional stress \(\tau_f\) is acting over the debonded length \(l_d^{(o)}\). The solution of the problem is simplified by shifting the origin from the crack surface to the place just above the debonded crack tip[21]. It is assumed that a second crack is in a semi-infinite domain defined by \(x_3 \geq 0\), subjected to a uniform tensile stress \(\sigma_\infty\) on the far field \((x_3 \rightarrow \infty)\), and a periodic stress over the surface \(x_3 = 0\) (see Fig. 2). The shape of the second crack is modeled by a thin spheroid as:

\[
\Omega_0 : \frac{x_1^2 + x_2^2}{a^2} + \frac{(x_3 - l_c)^2}{c^2} \leq 1, \quad \frac{c}{a} < 1
\]

where "a" corresponds to the radius of the crack and 2c is the maximum width of the crack, and length \(l_c\) is the distance from position \(x_3 = 0\) to the crack center. If the number of fibers intersected by the matrix crack \(\Omega_0\) is N, the total bridged domain \(\Omega\) for the fiber in the matrix crack is equal to the sum of the inclusions. The cross section of a typical ith fiber with a radius of \(r\) is approximated as

\[
\Omega_i : \frac{x_1^2 + x_2^2}{r^2} + \frac{(x_3 - l_c)^2}{c^2} \leq 1, \quad \frac{c}{r} < 1
\]

The fibers are aligned parallel to the \(x_3\)-axis with an average spacing \(\lambda\).

According to the traction free condition in a unbridged crack, the bridging stress of the fiber, \(\sigma_r\), in the second crack has been derived[22] as

\[
\sigma_r = \frac{(1 - \alpha) \frac{a}{r}}{1 + \beta \left(\frac{a}{l_c}\right) + (1 - \alpha) \left(\frac{a}{r}\right)} \xi \sigma_\infty
\]

In which, the \(\alpha\) is called bridging factor and is defined as
\[
\alpha = \frac{1}{2r} - \frac{\alpha}{r} \left[ 2K_f + \beta \left( \frac{r}{l_c} \right) \right] \\
- \frac{1}{r} \left[ 1 + \frac{\alpha}{r} \left[ 2K_f + \beta \left( \frac{r}{l_c} \right) \right]^2 - 4K_f (f + \kappa \alpha^2) \right]^{1/2}
\]

where

\[
\xi = \left[ 1 + \frac{(1-f)E_m}{fE_f} \exp \left( -\sqrt{\frac{\eta}{r}} \right) \right]^{-1/2}
\]

and

\[
\kappa = \frac{\pi \bar{\mu} (1-f)E_m}{8\tau_f (1-\nu)} E_f E \xi \sigma_A
\]

and \( \beta \) is a constant factor. The detailed definitions of the terms used in above equations are explained in reference [22]. Note that \( \alpha = 1 \) corresponds to unreinforced matrix where as \( \alpha = 0 \) refers to the perfectly bonded composites.

**DETERMINATION OF DEBONDING LENGTH IN A CRACK**

The debonding length is determined from energy considerations and using concepts of fracture mechanics[22]. It is assumed that when a debonding crack propagates, the length of the matrix crack \( 2a \) remains unchanged. It is also assumed that all of fibers associated with the debonding inside the matrix crack have identical debonding lengths.

In order to use the energy approach, the debonding region can be treated as an extension of a kind of Mode II crack. A typical cylindrical element [14] has been chosen to calculate the energy change when the debonding crack grows as shown in Fig. 3. If the stresses above the debonding front remain unchanged [21], then energy changes of this isolated cell when the debonding crack grows by an incremental amount \( 2\pi rd(\ell_0) \) come from: (i) potential energy release \( dW_p \), which includes the increment in the elastic strain energy of fiber/matrix \( dW_e \) and work done by external load (bridging stress) \( dW_L \); and (ii) frictional work \( dW_f \) along the debonding zone. The surface energy of the debonding crack \( (dW_r) \) must be balanced by the increment of released energy deducting the amount of frictional energy dissipation for the system (cylindrical cell), i.e.
\[
\begin{align*}
\text{d} W_\gamma &= -dW_p - dW_F \\
&= dW_L - dW_E - dW_F 
\end{align*}
\]

(7)

where

\[
\begin{align*}
\text{d} W_L &= \frac{r \tau_f^2}{2E_f} - \frac{\sigma_T^2}{\tau_f^2} - \frac{2E \sigma_T}{(1-f)E_m \tau_f} \left( \frac{l_d}{r} \right)^2 \cdot 2\pi rd(l_d) 
\end{align*}
\]

(8)

\[
\begin{align*}
\text{d} W_E &= \frac{r \tau_f^2}{4E_f} - \frac{\sigma_T^2}{\tau_f^2} - \frac{4E}{(1-f)E_m \tau_f} \left( \frac{l_d}{r} \right)^2 \cdot 2\pi rd(l_d) 
\end{align*}
\]

(9)

\[
\begin{align*}
\text{d} W_F &= \frac{r \tau_f^2}{E_f} \left( \frac{2E \sigma_T}{(1-f)E_m \tau_f} \left( \frac{l_d}{r} \right)^2 + \frac{\sigma_T}{\tau_f} \left( \frac{l_d}{r} \right) \right) \cdot 2\pi rd(l_d) 
\end{align*}
\]

(10)

\[
\begin{align*}
\text{d} W_{\gamma*} &= 2\gamma* \cdot 2\pi rd(l_d) 
\end{align*}
\]

(11)

Using Griffith type of energy balance, which is

\[
\frac{\partial W_{\gamma*}}{\partial (2\pi rl_d)} = 2\gamma* \\
= - \frac{\partial W_p}{\partial (2\pi rl_d)} - \frac{\partial W_F}{\partial (2\pi rl_d)} 
\]

(12)

leads to a value of \(l_d\) as follows

\[
\begin{align*}
l_d &= \frac{\sigma_T}{r \tau_f} \left\{ \frac{fE_f}{E} + \frac{(1-f)E_m}{E} \left( \frac{\sigma_T}{\tau_f} \right)^2 \right\} \left\{ \frac{\sigma_T}{2\tau_f} \right\}^{1/2} 
\end{align*}
\]

(13)

where \(\gamma*\) is debonding surface energy, \(\sigma_T\) is given by eqn. (3) and
\[ \sigma_i = \sqrt{\frac{8E\gamma^*}{r}}. \tag{14} \]

Generally, the debonding process may be either a stable or an unstable growth. However, if we consider the derivative of eqn. (12), it can be pointed out that the analysis of the present model for the debonding crack is a steady state consideration. This is consistent with experimental observation. The extent of the debonding tends to be small if frictional shear stress \( \tau \) is large. In addition, the extent of debonding is also reduced when the interfacial toughness \( \gamma^* \) increases.

**EXPERIMENTAL INVESTIGATION**

This section describes the investigation of the micromechanism of matrix fracture by means of quantitative analysis, including the details of the uniaxial tension test and the thin section study of optical fluorescence microscopy.

**Specimen Preparation**

Fabrication of the specimens was achieved by using a plexiglass mold. The mold consisted of a base plate, two side plates, two end plates and two guide plates. The guide plates with 30 holes were used to provide alignment of fibers and to separate the anchorage portion from the test portion of specimen during construction. The anchorage part of the specimen was cast with the normal type epoxy resin right after steel fibers were aligned in the mold and fixed outside to the mold frame. Continuous steel wires obtained from Bekaert Co. Chicago, IL, with diameters of 0.2032 mm (0.008 in.), 0.4064 mm (0.0016 in.), and 0.8128 mm (0.0032 in.) were used as fibers. Neat cement paste made of Type I portland cement with a water/cement ratio of 0.35 was used as the matrix for the test portion. In order to facilitate the workability, 3.86 ml superplasticizer per kilogram of cement were added into the mixture. The specimens were cast horizontally with the embedded fiber perpendicular to the direction of casting. Three series with different fiber volume fractions, 0.767%, 1.534%, and 6.135% were employed in the experimental study. For the specimen series with 0.767% fiber volume fraction, fifteen steel wires with a diameter of 0.4 mm were used in each specimen. The number of fibers increased to thirty with the same diameter (0.4 mm) for the specimen group of 1.534% volume fraction. Thirty wires with 0.8 mm diameter were used for the specimen series of 6.135%. Specimens were cured in water and tested at 14-day age after the surfaces of the specimen were ground. In general, three groups of tests were identical. No artificial crack was introduced in the specimens.

**Test set-up and Procedure**

The specimen being held in the mechanical fixture is shown in Fig. 4. Two frictional grips were used in the set-up. One of them was connected to the servohydraulic actuator while the other was connected to the load cell. Two
Linear Variable Differential Transformers (1.127 mm, or 0.05 in., range), i.e., LVDTs, mounted on the opposite side of the specimen using a 76.4 mm (3 in.) gage length were used to measure the extension of specimen during the loading. The average output of LVDTs was also used as the feedback signal in the control of the servohydraulic system. The uniaxial tensile tests were performed at a rate of 0.000762 mm of LVDT’s output per minute. Once the specimens were loaded up to the designated displacement level, the mode of the control was switched to force-signal control and a command of holding position was selected at that time. The switching of the control mode was necessary due to the need of removing LVDTs from the set-up. After demounting the LVDTs from the specimen, two rigid steel blocks (50.8 mm by 25.4 mm by 12.7 mm in dimensions) were glued onto the opposite surfaces of the specimen by using a fast hardening epoxy to “freeze” the opening displacements of matrix cracks at that stress value. When the epoxy adhesive hardened, the specimen was unloaded and removed from MTS machine. The data of load, elongation and stroke were acquired through the TestsStar software in the controlling work station.

**Study of Optical Fluorescence Microscopy**

To quantify the debonding properties of the loaded specimen with minimal amount of disturbance, the "strain freezed" specimens were treated prior to the study of thin sectioned samples in an optical microscope. The specimen length was first trimmed off to the section restrained by the steel blocks. Then, an epoxy impregnation procedure (Fig. 5) for the specimen was performed by using low viscosity epoxy (LR white resin from Polysciences, Inc., Warrington, PA) and hardener mixed with fluorescent dye. The dissolution of the dye in epoxy utilized a less viscous medium for transporting the dye into the debonded zone. After impregnation, specimens were then cured at room temperature for 24 hours. The thin sections were prepared by sectioning the specimens longitudinally and grinding using grit size of 320, 400, and 600 with a non-aqueous polishing lubricant. Lapp cloths impregnated with diamond paste of 6 μm, 3 μm, 1 μm, and 0.25 μm were used in the final polishing of the samples. Fig. 6 schematically shows the preparation procedures of the thin sections.

The polished thin sections were examined with a fluorescence microscopy and an image analysis system to quantify the debonding behavior. It was ensured that the fluorescence dye together with LR white resin would penetrate into the interface if there exist a debonding crack. As an illustration, Fig. 7(a) shows two bright (upper and lower) lines along the boundary of fiber and matrix which directly characterize the size of the debonded surface. On the other hand, no dye penetration is seen for the case of bonded interface in Fig. 7(b). Furthermore, in order to observe an entire contact area of the debonded interface clearly, the fibers were taken out from the thin sections. A typical view of debonded zone after removing the fiber is shown in Fig. 8. The different brightness along the groove (fiber-matrix interface) of Fig. 8 is detectable to identify both the bonded and debonded regions. The transitional position (the
ending zone of debonding) was decided using an optical fluorescence microscopy, which was used to acquire images from the thin sections into an image analysis system. Consequently, quantitative measurement of fracture process zone (debonded length) was achieved from that image.

**EXPERIMENTAL RESULTS AND COMPARISON WITH ANALYTICAL PREDICTIONS**

The methodology described above was applied to three series of specimens (f=0.767%, 1.534%, and 6.135%), loaded up to specified values as summarized in Table I. A typical stress-strain curve of fiber volume fraction 1.534% is shown in Fig. 9. The four discrete loading levels (1), (2), (3) and (4) are indicated in this figure by the dotted lines. For the steel fiber reinforced cement composite, a history of matrix cracks can be observed during the testing. Studies of optical fluorescence microscopy have revealed the growth of debonding at the fiber-matrix interface for a series of stress levels as shown in Fig. 10, where four debonded lengths corresponding to four discrete loading levels (shown in Fig. 9) are demonstrated. Fig. 10 (a) shows the debonded length at a stress level of 4.8 MPa which is lower than the stress level of bend over point (BOP) of about 5.5 MPa. It can be seen from the figure that even at this low stress level, the debonded length already reaches about 1 mm along the fiber on both sides of the transverse matrix crack. This observation reveals that both transverse matrix crack and longitudinal debonding cracks develop before the BOP at which the matrix contributes to the composites tensile strength the most. Fig. 10 (b), (c) and (d) show the debonded lengths obtained at three subsequent stress levels, 6.4 MPa, 7.9 MPa and 11.0 MPa. An increase of the debonded lengths with respect to the applied stress levels can be observed from these figures. This proved out the validity of the theoretical assumption of "stable growth" for the debonding crack. The directly measured lengths at the different stress levels are plotted in Fig. 11. In the figure, the symbols of triangle are the test results of Moire Interferometry [23] and the symbols of circle are the test results of the present study. Both experiments were conducted on essentially identical steel fiber reinforced cement based composite specimen with a slight difference in fiber volume fraction, 1.53% and 1.3%. It can be seen from the figure that the test results from two different experiments are quite consistent. The debonded lengths in both cases increase with the increase of external stress. Also, the experimental measurements agree well with analytical results (solid curve in the figure). The predicted results are obtained by using the material properties reported in Table II with v_f=0.3 and v_m=0.2, where surface energy of the matrix γ=7.5N/m, surface energy of the interface γ'=5.5N/m and frictional shear stress τ_f=1.3MPa are from Li et al.[24] and Jenq and Shah[25].

It is also noted that the debonded lengths at the first few preceding cracks were longer than that at the succeeding cracks for the same stress level as can be seen by comparing Fig. 10(c), which shows the debonded length associated with the first matrix crack at a loading level of 7.9 MPa for the specimen.
reinforced with 1.534% fiber volume fraction, and Fig. 12, which shows the debonded length measured at the second matrix crack at same loading level for the same type of specimen. Such a phenomenon implies that the debonded length is a function of crack spacing. However, the differences in debonded length among the latter preceding and succeeding cracks tend to become smaller. This also can be predicted by the theoretical analysis as shown in Fig. 13 for $f=0.767\%$ if $a/r>>1$ is considered. It can be seen that the curve represents the debonding length at the first crack is higher than all the rest curves which represent the debonding lengths at the succeeding cracks. Moreover, it can be seen from the figure that the differences in the debonded length decrease as the number of cracks increases. The relationships between normalized debonding length and fiber volume fraction from eqn. (13) for the first crack (plotted in Fig. 14) implies that the debonding behavior is more sensitive to the low fiber volume fraction (e.g. $f<2\%$). This is also observed experimentally by comparing the experimental observations of present study for two cases shown in Fig. 10(b), demonstrating the debonded length of a specimen with 1.534% fiber volume fraction at the first matrix crack at a loading level of 6.4 MPa, and Fig. 15, showing the debonded length for a specimen reinforced with 6.135% fiber volume fraction with the rest conditions same with the former specimen. Thus, it can be concluded that the debonding phenomenon is retarded when fiber volume fraction increases.

Another model to determine the debonding length for the case of constant friction proposed by Hutchinson and Jensen [16] has been used to compare with experimental results and the theoretical prediction made by present model for the case that the first matrix crack size is large, i.e. $a/r>>1$ and $r/l<<0$. This comparison is shown in Fig. 16. According to the model in present study and that of HJ's, the debonded lengths denoted by $l_d$ and $l_d^{HJ}$ are determined respectively as follows:

$$l_d = \frac{\sigma_A}{2\sigma_f} \left[ 1 - \sqrt{\frac{E_f}{E} + (1-f)\frac{E_m}{E} \left( \frac{\sigma_A'}{\sigma_A} \right)^2} \right]$$  \hspace{1cm} (15)$$

$$\sigma_A' = f\sigma_A' = \sqrt{\frac{8E_f\nu^*}{\nu}}$$  \hspace{1cm} (16)$$

and
where $\sigma_A^{i}$ is the applied stress of initial debonding for present study and $\sigma_A^{(0HJ)}$ is for HJ’s model. The nondimensional constants $c_1$ and $c_3$ are functions of $E_m$, $E_r$, $v_m$ and $v_f$ [16]. Fig. 16 indicates that the deviation of the threshold stress of initial debonding predicted by the present work and HJ’s model is quite small. This suggests that the effect of Poisson’s ratios is not significant for the stress at initial debonding in the steel-fiber reinforced cement-based composite. Furthermore, for the higher loading the analytical predictions of present work are closer to the experimental results than that of the HJ’s model. The HJ’s model overestimates the debonded length. This may be attributed to the absence of frictional consideration for the HJ’s model when the jump condition in the fiber stress from above the tip to just below the tip is evaluated. Mathematically, it can also be examined that the value of eqn. (15) is smaller than that of eqn. (17) when applied load is large. Since eqn. (15) has an extra term of order $(\sigma_A/\tau_f)^{1/2}$, which is a result of the frictional sliding effect. Therefore, it is evident that the resistance of frictional sliding in the fracture criterion (energy balance analysis) plays an important role in determination of the extent of the debonding and cannot be ignored.

CONCLUSIONS

In the multiple cracking stage, fibers suppress the extension of matrix cracking through three effects: fiber bridging, interfacial debonding and frictional sliding. Debonding and sliding absorb part of the fracture energy and play an important role in enhancing toughness of the composites. The amount of mechanical energy absorbed by interfacial debonding and frictional work in the multiple fracture stage increases the stress capacity. In turn, more cracks and the smaller crack spacing are observed. With reference to the mechanics of debonding along fiber-matrix interfaces, it implies that the debonding can be suppressed by increased interfacial toughness, increased frictional shear stress and increased fiber volume fraction. The technique of optical fluorescence microscopy has been successfully used to measure the interfacial failure in the steel fiber-reinforced cement composite. It is shown that the debonding develops before the first matrix crack traverses the entire section of the specimen (BOP). The predictions of the debonded lengths agree well with the measured values. It is shown that the extent of the debonding is a steady-state process rather than an unstable growth.
Acknowledgments: This research was supported by the National Science Foundation Center for Science and Technology of Advanced Cement-Based Materials (ACBM) through Grant DMR-9120002. Partial support from Air Force Office of Scientific Research under grant F49620-92-J-0319 was also acknowledged.

REFERENCES:


**TABLE I — LOADING LEVELS FOR DIFFERENT GROUP OF SPECIMENS**

<table>
<thead>
<tr>
<th>Fiber volume fraction(%)</th>
<th>Stages of stress level (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
</tr>
<tr>
<td>0.767</td>
<td>2.2</td>
</tr>
<tr>
<td>1.534</td>
<td>4.8</td>
</tr>
<tr>
<td>6.135</td>
<td>4.2</td>
</tr>
</tbody>
</table>

**TABLE II — MATERIAL PROPERTIES OF STEEL FIBER REINFORCED CEMENTITIOUS COMPOSITES**

<table>
<thead>
<tr>
<th>Mix design (by weight)</th>
<th>(Cement:Water)1:0.35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curing time (days)</td>
<td>14</td>
</tr>
<tr>
<td>Matrix modulus (GPa)</td>
<td>14</td>
</tr>
<tr>
<td>Fiber modulus (GPa)</td>
<td>210</td>
</tr>
<tr>
<td>Frictional shear stress (MPa)</td>
<td>1.3</td>
</tr>
<tr>
<td>Surface energy of matrix (N/mm)</td>
<td>0.0075</td>
</tr>
<tr>
<td>Surface energy of interface (N/mm)</td>
<td>0.0055</td>
</tr>
</tbody>
</table>

* 3.86 ml superplasticizer per kilogram cement was used.
Fig. 1—Schematic drawing for a stage after bend over point for composites
Fig. 2—Schematic drawing for mathematical modeling
Fig. 3—An isolated element with debonding crack
Specimen set-up

Fig. 4—Uniaxial tension test set-up
Fig. 5—Schematic drawing for impregnation procedures
Fig. 6—Schematic drawing for thin sectioning procedures
Fig. 7—Different views for debonded and bonded interface after impregnation
Fig. 8—View of entire debonded area after taking out a fiber

Steel- fiber- reinforced cement
Diameter of steel wires = 0.4 mm
Fiber volume ratio = 1.534%

Fig. 9—Typical stress-strain curve for steel fiber reinforced cement
Fig. 10—Development of debonded length as a function of applied stress at first matrix crack for a specimen with $V_f = 1.534$ percent
Fig. 10 cont.—Development of debonded length as a function of applied stress at first matrix crack for a specimen with $V_F = 1.534\%$.
Fig. 11—Comparison of theoretical prediction with experimentally observed debonding length.

Fig. 12—Debonding length at second crack at stress level of 7.9 MPa for a specimen with $V_f = 1.534$ percent.
Fig. 13—Theoretical prediction of debonding lengths as a function of applied stress at different matrix crack positions
Fig. 14—Theoretical prediction of debonding lengths as a function of fiber volume fraction

† For the first matrix crack
Fig. 15—Debonding length for a specimen with $V_f = 6.135$ percent at first matrix crack and stress level of 6.1 MPa.

Fig. 16—Comparison of experimentally observed debonding length with predictions of two different models.
Effect of Lateral Stress on the Debonding and Pullout of Steel Fibers in a Cementitious Matrix

by C. K. Y. Leung and Y. Geng

Synopsis:

In many practical engineering applications of fiber reinforced concrete (FRC), fibers are subjected to significant lateral loading. The lateral stress may have a significant effect on fiber debonding and pull-out, and hence affecting the performance of FRC. In this investigation, a novel experimental set-up has been developed to carry out fiber pull-out test under various levels of lateral compression. Interfacial properties for steel fiber in mortar are derived from the measured fiber load versus displacement curves, based on a unified fiber debonding theory. As expected, the interfacial friction at the onset of sliding increases with applied compressive stress. Surprisingly, the pre-sliding interfacial resistance (which can be either the interfacial strength or the interfacial fracture energy, depending on whether interfacial debonding is strength or fracture governed) is also found to increase with lateral compressive stress. Also, with higher compression, there is a more rapid decay of interfacial friction when the fiber is sliding out of its groove. As a result, while lateral compression can significantly increase the peak pull-out load, the increase in total energy absorbed during the pull-out process is much less drastic.

Keywords: Composite materials; debonding; fiber reinforced concretes; interface; pullout tests; stresses
Fiber addition is the most widely used technique in improving the fracture toughness of cementitious materials. For effective toughening, the fibers should be able to debond and subsequently pull out from the matrix. Fiber debonding and pull-out have been studied by many investigators and various models have been developed, with either the strength-based [1-6] or fracture-based [7-12] approach. Interfacial parameters governing fiber pull-out and debonding (such as interfacial strength or toughness and interfacial friction) can be obtained from the fiber pull-out test, with specimens consisting of one or more fibers embedded in a cementitious matrix.

In the reported literature, fiber pull-out tests were almost always carried out in a configuration with zero applied stress in any direction perpendicular to the fiber. However, significant lateral stress can be present in many practical applications. For example, along shear cracks in beams and splitting cracks in components under compression, crack bridging fibers are under lateral compression. For cracks at the bottom of a two-way slab, there is lateral tension acting on fibers bridging the crack. Fiber pull-out test under uniform lateral confinement has been carried out by Pinchin and Tabor [13]. However, in practical situations such as those discussed above, significant far-field lateral stress is only acting in a single direction perpendicular to the fiber. In this investigation, a novel experimental setup is developed for fiber pull-out test under unidirectionally applied lateral stress.
Results on steel fiber under various levels of lateral compression are reported. Effect of lateral tension is currently under investigation and will be reported in a subsequent paper. From the measured fiber load versus displacement curves, interfacial properties are derived and their sensitivity to applied lateral stress evaluated. The results can provide insight to the effectiveness of fiber reinforcement in applications with significant lateral stresses acting on the fibers.

**MODELLING OF FIBER DEBONDING AND PULL-OUT**

The determination of interfacial parameters from the pull-out test record has to be based on a given fiber debonding and pull-out model. In this investigation, we divide the pull-out record into two parts: the pre-peak and post-peak regimes, as shown in Fig.1. The pre-peak regime is initially elastic followed by stable debonding until the peak load is reached. In the post-peak regime, the fiber has been completely debonded and is being pulled out of its groove.

To obtain the interfacial parameters, a unified model developed by Leung [12] is employed. In the model, debonding starts when the interfacial shear stress (obtained from a shear lag analysis) at any point along the interface reaches an effective shear strength $\tau_{\text{eff}}$. After debonding, the interfacial stress drops immediately to the interfacial friction $\tau_i$, which is assumed to stay constant in the post-peak regime. In other words, $\tau_i$ is assumed to remain constant until the whole fiber has been debonded. In the post-peak regime, $\tau_i$ will be a decreasing function of the sliding distance. This approach is valid for both strength-based and fracture-based debonding provided $\tau_{\text{eff}}$ is appropriately defined.

For strength-based debonding, $\tau_{\text{eff}}$ is a material parameter usually referred to as the interfacial shear strength $\tau_s$.

For fracture-based debonding,

$$\tau_{\text{eff}} = \tau_i + \sqrt{\frac{E_f \Gamma (1-\alpha) \rho^2}{r_f}}$$

where $\alpha = E_f V_f / E_c$; $E_c = E_f V_f + E_m V_m$
\[ \rho^2 = \frac{2G_m E_c}{E_f V_f E_m \ln \left( \frac{R^*}{r_f} \right)} \], and

\[ \ln \left( \frac{R^*}{r_f} \right) = - \frac{2 \log V_f + (1 - V_f)(3 - V_f)}{4(1 - V_f)^2} \]

In the above expressions, \( E_f, E_m \) are the fiber and matrix Young’s moduli, \( G_m \) the matrix shear modulus, \( V_f, V_m \) the fiber and matrix volume fractions, \( \Gamma \) the interfacial fracture energy and \( r_f \) the fiber radius. It should be noted that the above expressions have not considered the plausible presence of a compliant interphase between the fiber and the bulk cementitious matrix reported by some investigators[14,15]. The reduction of \( \rho \) due to the presence of a compliant interphase will be discussed later in this paper.

From the above expressions, for strength-based debonding, the measured \( \tau_{\text{eff}} \) is expected to be independent of \( \tau_i, r_f, V_f \) and \( \rho \). For fracture-based debonding, \( \tau_{\text{eff}} \) is not a material parameter but is a function of the fiber, matrix and interfacial properties as well as the fiber volume fraction and size. In particular, for the same fiber and matrix, if \( \Gamma \) remains constant, the change in \( \tau_{\text{eff}} \) is expected to be the same as the change in \( \tau_i \) as the lateral stress is varied.

According to the unified debonding theory developed by Leung [12], as well as other debonding theories [2,4,5], a sudden load drop will occur right after the peak load as a consequence of unstable debonding. \( \tau_i \) can then be obtained from the point after the drop. Since the peak pull-out load is a function of \( \tau_{\text{eff}}, \tau_i \) and \( \rho \) (defined above), \( \tau_{\text{eff}} \) can be deduced once \( \tau_i \) is known and \( \rho \) calculated.

In the post-peak regime, a decrease of interfacial friction with sliding distance has been observed by many investigators [4,16,17,18,19]. In this work, \( \tau_i \) as a function of sliding distance for various lateral stress levels is obtained by dividing the load at a given sliding distance by the remaining area of the fiber in its groove. The effect of lateral stress on the \( \tau_i \) vs sliding distance curve will be discussed but no empirical fitting of the curves will be attempted.
**SPECIMEN PREPARATION, EXPERIMENTAL SET-UP AND TESTING PROCEDURE**

The specimen for the measurement of interfacial properties consists of a single steel fiber embedded in a rectangular block of mortar (Fig.2). The dimension of the specimen is 25.4 mm x 12.7 mm x 9.5 mm (1" x 1/2" x 3/8").

The straight steel fiber is made from low carbon, cold drawn steel wire (Bekaert Corporation). The yield strength and ductility are 1040 MPa and 0.8 %, respectively. The diameter of the fiber is 0.5 mm and the embedded length of fiber in the mortar specimen is controlled to be either 5 mm or 10 mm.

Mortar used in this investigation is made from type III Portland cement and mortar sand. A water/cement/sand ratio of 0.5:1:2 is employed. The casted specimens are tested at the age of 7 days. The compressive strength obtained from cylindrical specimens is 43 MPa and the splitting tensile strength is 3.6 MPa.

A novel experimental set-up has been developed to carry out fiber pull-out test with lateral stress acting on the embedded fiber (Fig.3). The setup consists of two orthogonal loading chains, referred to as the pulling chain (along the fiber direction) and the lateral chain (perpendicular to the fiber). Each chain is composed of several blocks and rods, as well as a load cell. The part of the pull-out specimen with the embedded fiber is glued to a specimen holder while the exposed end of the fiber is held tightly by a grip. The holder and grip are attached to the load cells on the two separate chains through hardened steel rods passing through ball bearing blocks, which prevent any bending of the load cell that may affect the accuracy of load measurement. At the other side of the load cell is a specially designed rod with a key along part of its length, which goes into a carefully aligned block with a key-way machined to close tolerance. With the key in the key-way, the whole loading chain is constrained to move linearly without any rotation. The other end of the key rod (which is screwed) goes through a reaction block with a nut resting on it. In the pulling chain, the nut is behind the reaction block. By turning on a motor connecting to the nut through gears, the rotation of the nut will induce a pulling force on the chain. In the lateral chain, an end block is placed behind the specimen. By turning the nut (which is in front of the reaction block) in the lateral chain, the specimen will come into contact with the end block, resulting in lateral compression as lateral movement is restrained. In the pull-out experiment, displacement is
measured with a LVDT between the specimen holder and the grip.

Using the setup described above, two sets of specimens are tested with the embedded fiber lengths of 5 and 10 mm, respectively. The lateral compressive stresses applied are 0, 10, 20 and 30 MPa. Fiber is pulled out under displacement control with a loading rate of 2.65 µm/s, controlled by the motor and gear system at the end of the pulling train (Fig.3). Displacement control enables the measurement of post-peak softening behavior. It should be mentioned that the load in the lateral direction only varies by very small amounts during fiber pull-out and can be considered constant for all purposes.

RESULTS AND DATA INTERPRETATION

The pull-out test results for fiber embedded lengths of 5 mm and 10 mm under various levels of lateral compressive stresses are shown in Figs.4 and 5 respectively. The peak pull-out load is found to increase with lateral compression. From the results, two different types of post-peak fiber pull-out behavior, one with stick-slip effects, and the other without, can be observed (Fig.6). The stick-slip effect is more common in cases with high lateral compression. For example, almost all the curves for 30 MPa lateral compression show the stick-slip effect while less than half of the curves under zero lateral compression exhibit stick-slip. At the stick stage, load increases slowly (upward solid lines) and the friction is static. At the slip stage, load decreases suddenly (downward dash dot) and the friction is kinetic. For a case without stick-slip, the interfacial friction $\tau_i$ is simply obtained from the load level right after the unstable post-peak load drop. For a case with stick-slip, $\tau_i$ is obtained from the load level corresponding to the midpoint of the first stick curve (see Fig.6). According to stick-slip theory based on a simply spring-mass model[20], the value of interfacial friction obtained in this manner represents the kinetic value.

To obtain the value of $\rho$, the fiber volume fraction is required. Since the debonding theory is derived for the ideal case of a cylindrical fiber embedded in a cylindrical matrix, an equivalent volume fraction has to be found for the rectangular specimen used in this investigation. An approximate equivalent fiber volume fraction is obtained by taking the effective width of the specimen to be the same as the specimen height $H$ (which is 9.5 mm in this case). Then, with $V_f = \pi r_f^2 H$, $\rho$
is found to be 0.255.

After $\tau_i$ and $\rho$ are obtained, $\tau_{\text{eff}}$ can be determined from the peak load. Iteration is carried out until the theoretical peak load computed from the assumed value of $\tau_{\text{eff}}$ agrees with the experimental value. The values of $\tau_i$ and $\tau_{\text{eff}}$ as a function of lateral compressive stress are shown in Fig. 7. The results indicate that both $\tau_i$ and $\tau_{\text{eff}}$ increases with lateral compression.

In Figs. 8 and 9, the reduction of interfacial friction with fiber sliding in the post-peak regime is shown for the two fiber embedment lengths. Each curve in the figures is obtained as the average from four tests. To calculate the interfacial friction at a given sliding distance, the remaining embedded length is taken to be the total length of the fiber minus the sliding distance. In other words, the elastic deformation of the fiber, which is small compared to the fiber sliding distance, has been neglected.

**DISCUSSIONS**

The average interfacial friction and the effective shear strength for both embedded lengths are shown in Fig. 10. The agreement of results obtained from two different embedded lengths shows the reliability of the experimental technique. $\tau_i$ increases with lateral stress as expected due to the proportionality of kinetic friction with normal compressive stress at the interface. It is, however, interesting to find that $\tau_{\text{eff}}$ also increases with lateral compression and is actually increasing at a higher rate than $\tau_i$.

Since the determination of $\tau_{\text{eff}}$ relies on the value of $\rho$, and $\rho$ is obtained as an upper bound by neglecting the plausible presence of a compliant interphase, $\tau_{\text{eff}}$ values as a function of lateral compression have also been calculated using lower values of $\rho$. Note that a lower $\rho$ value can also be interpreted as a reduced effective fiber volume fraction in the pull-out specimen, resulting, for example, from an increased effective matrix volume. The results of the sensitivity study are shown in Fig. 11. It can be seen from Fig. 11 that the use of a lower value of $\rho$ would indeed provide lower $\tau_{\text{eff}}$ values as well as a lower slope of the $\tau_{\text{eff}}$ vs lateral compression relation. However, even with $\rho$ as low as 0.005, which corresponds to the
presence of an extremely compliant interphase, $\tau_{\text{eff}}$ is still found to increase more with lateral compression than $\tau_i$.

The trend is therefore a real material behavior. If interfacial debonding is strength-governed, this implies the interfacial strength increases linearly with lateral compression. If interfacial debonding is fracture-governed, the interfacial fracture energy is shown to increase linearly with lateral compression. The increase in pre-sliding interfacial resistance (i.e., interfacial strength or interfacial fracture energy) with lateral compression can be explained qualitatively with the help of a simple model in Fig.12, which shows a fiber with a rough surface embedded in a quasi-brittle matrix. In order for the fiber to slide relative to the matrix, the mechanical interlock (due to surface roughness) has to be overcome by local microscopic fracture in the matrix. The presence of lateral compression make it more difficult for such local fractures to occur and hence increase the pre-sliding interfacial resistance.

Figs.8 and 9 show that interfacial friction decreases with sliding distance as expected. At higher values of lateral compression, the initial interfacial friction is higher. However, the pull-out resistance will also decrease more rapidly with fiber sliding. In both Figs.8 and 9, curves for different lateral compression can be observed to cross over each other, indicating a lower pull-out resistance for a higher lateral compression beyond a certain sliding distance. One plausible explanation for the increasing decay in pull-out resistance with lateral compression is the 'abrasion and grinding effect'. The sliding of steel fiber can abrade the cement matrix, thus increasing the size of the cement matrix groove and reducing the misfit between the fiber and matrix. The particles are subsequently ground into smaller sizes and may act as small 'ball bearings' to reduce the interfacial frictional coefficient. The presence of the abrasion and grinding mechanism is supported by SEM observation of fiber/matrix interface for fibers before debonding, right after debonding and after pull-out of 1 mm and 10 mm [21]. It is expected that, under higher interfacial normal compression, the abrasion and grinding effects are more significant. As a result, both the misfit between the fiber and matrix and the frictional coefficient decrease at a higher rate.

The behavior of fiber reinforced concrete with lateral compression acting on fibers can be best illustrated by Fig.13 and 14, which show respectively the increase of peak pull-out load and total energy absorption (up to a sliding distance of 5 mm) with lateral compression. As shown in Fig.13, the peak pull-out load of a fiber can be significantly increased by the presence of lateral compression. A
compressive stress of 30 MPa can almost double the peak pull-out load. A high peak pull-out load implies high resistance to initial crack propagation and hence significant improvement of the strength of the composite. On the other hand, due to the higher rate of drop of post-peak interfacial friction with lateral compression, the increase of energy absorption with lateral stress is not as much as that of the peak pull-out load (Fig.14). The toughness improvement of fiber reinforced concrete is therefore not very drastic even when there is high lateral compression acting on the fibers.

CONCLUSIONS

With the presence of lateral compression on the fiber, both the interfacial friction at the onset of sliding and the effective interfacial strength increase, thus leading to significant increase of the peak fiber pull-out load. However, a higher lateral compression also results in more rapid drop of the post-peak pull-out resistance. As a result, the total energy absorption does not improve as drastically as the peak pull-out load. This paper focuses on the effect of constant lateral compression on fiber pull-out. The effect of lateral tension and variable lateral stresses is currently under investigation and will be reported in a subsequent paper.

REFERENCES


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**Fig. 1**—Typical pullout curve for a single fiber

**Fig. 2**—Single fiber pullout specimen

\[ L_e : \text{Fiber Embedded Length} \]
Fig. 3—Novel set-up to investigate effect of lateral stress on fiber pullout
Fig. 4—Experimental pullout curve for an embedded length of 5 mm
Fig. 5—Experimental pullout curve for an embedded length of 10 mm.
Fig. 6—Typical results on fiber pullout
Fig. 7—Interfacial friction and effective interfacial strength versus lateral compression.
Fig. 8—Average interfacial friction versus sliding for different values of lateral compression (embedded length $L_e = 5$ mm)

Compressive Stress
Solid: 0 MPa
Dash: 10 MPa
Dot: 20 MPa
Dashdot: 30 MPa

Fig. 9—Average interfacial friction versus sliding for different values of lateral compression (embedded length $L_e = 10$ mm)
Fig. 10—Average interfacial friction and effective interfacial strength versus lateral compression
Fig. 11—Effect of $\rho$ on effective interfacial strength

Fig. 12—Simple model for microscopic fracture involved in fiber debonding
Fig. 13—Peak pullout load versus lateral compression

Fig. 14—Energy absorption up to 5 mm sliding distance versus lateral compression
Fluorescence Microscopic Study of Fracture Process of the Interfacial Zone between a Steel Fiber and the Cementitious Matrix under Pullout Loading

by M. Kawamura and S. Igarashi

Synopsis:
Fracture of the interfacial zone between a fiber and the cementitious matrix plays a significant role in the mechanical behavior of fiber reinforced cementitious composites. For better understanding of debonding characteristics of a fiber in the composites, the behavior of the extension of cracks along the interface was examined under the fluorescence microscope. The correspondence between the features of fracture zones obtained by the microscopic study and the fracture toughness for the interfacial zone is discussed. Examinations under the microscope revealed that the debonding and the extension of interfacial cracks were not caused by a simple shear failure at the actual interface, but that were accompanied by local failures over relatively wide regions surrounding the steel fibers. The incorporation of silica fume resulted in the reduction in areas containing local failures along the interface. Fewer local failures in the interfacial zone in the steel fiber-silica fume-bearing cement composite were reflected by the decrease in the fracture toughness for the interfacial zone.

Keywords: Debonding; fibers (discrete fibers); fluorescence; fracture properties; interface; microscopy; microstructure; silica fume; toughness
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INTRODUCTION

The bond between a fiber and the cementitious matrix is one of the important factors for improving the mechanical properties of fiber reinforced cementitious composites. Numerous experimental and theoretical studies have been carried out to evaluate the bond strength between a fiber and the cementitious matrix, and to describe the pull-out behavior of a fiber from the matrix. However, there is still some controversies about the bond between a fiber and the cementitious matrix, since the bond strength and the pull-out behavior of a fiber are influenced by many factors such as the type and the shape of fibers, the properties of the cementitious material and the experimental conditions. Furthermore, microstructural features of the interfacial zone between the fiber and the cementitious matrix are different from those in the bulk cement paste phase. This fact makes the situation more complex, although it has been emphasized that the specific microstructure of the interfacial zone should be taken into consideration in the model to describe the mechanical properties of fiber reinforced cementitious composites [1].

It is generally accepted that the formation of dense microstructures around fibers can lead to the increase in the bond strength of fibers and the matrix [2,3]. The addition of a polymer and silica fume into the cementitious matrix is effective in the formation of a dense and strong microstructure. The authors have shown that one effect of the addition of silica fume on the microstructure of the interfacial zone between steel fibers and the cement paste matrix [4,5] is that the value of microhardness in the interfacial zone was increased by the addition of silica fume. However, the fracture toughness for the interfacial zone, experimentally obtained by a single fiber pull-out test, decreased, although the dense interfacial zones were formed around a steel fiber [4,5]. Namaan & Najim [6] concluded that the addition of silica fume did not greatly improve the bond between steel fibers and the matrix,
as may be expected from the improvement in the strength of the matrix. However, from an indirect estimation of the bond strength of steel fibers within the composites, Sorounshian & Bayasi [7] drew a conclusion that the increase in the flexural strength and the toughness of steel fiber reinforced concretes containing silica fume was attributed to the increase in the pull-out strength of fibers.

Several theoretical models proposed to describe the pull-out behavior of a fiber from the matrix are divided into the two approaches [8]. The one is based on the assumption that debonding occurs when the maximum shear stress exceeds the adhesive shear strength along the interface. In the other model, the fracture toughness for the interfacial zone is used as a criteria for debonding. In the theoretical models, both the progressive debonding along the interface and the subsequent frictional shear resistance against pull-out loading are assumed to occur simultaneously. However, it is not easy to confirm the debonded length along the interface during the pull-out loading. Furthermore, it has not been confirmed that the progressive debonding along the interface occurs in the manner assumed in the theoretical model.

The main objective of this study is to elucidate the process of the interfacial fractures between a fiber and the cementitious matrix under the pull-out loading. The interfacial zone subjected to the pull-out loading is examined with the fluorescence microscope. Effects of silica fume on the interfacial fracture are discussed relating the results obtained by the microscopic examinations to the values of fracture toughness for the interfacial zone.

**EXPERIMENTAL**

(1) Materials and Mix Proportions

The cement used is ordinary Portland cement. A river sand is used as an aggregate. Straight steel fibers are used. The length and the diameter of steel fibers are 20mm and 0.6mm, respectively. Physical properties and chemical composition of silica fume are given in Table 1 and 2. Mix proportions of the matrix in the specimens used in the single fiber pull-out test are given in Table 3.

(2) Single Fiber Pull-out Test

The schematic diagram of a specimen used in the single fiber pull-out test is shown in Fig.1. Each fiber had an embedded length of 30mm. The
interfacial slit for obtaining the fracture toughness for the interfacial zone based on the compliance method is made around a steel fiber embedded by removing a thin rubber tube about 18 hours after casting [4]. Specimens were cured in water at 20 °C. Pull-out tests were carried out at the age of 28 days. The critical strain energy release rate was calculated by [4]:

\[ G_c = \frac{P_b^2}{2\pi D} \frac{dC(a)}{da} \]  

\[ P_b = P_{t,\text{max}} - \pi D l_{f,\text{net}} \tau_f \]  

where

\( P_b \): load obtained by subtracting the contribution of friction force from the maximum pull-out load

\( D \): fiber diameter

\( C \): compliance

\( a \): interfacial slit length

\( P_{t,\text{max}} \): maximum load

\( l_{f,\text{net}} \): net embedded length of fiber

\( \tau_f \): frictional shear stress

The frictional stress(\( \tau_f \)) was estimated from the area under the pull-out load vs. displacement curve up to 0.1mm of a relatively small displacement after the maximum load[4].

(3) Fluorescence Microscope Examination

(a) Pull-out test -- A mold for making specimens is shown in Fig.2(a) and (b). A cement paste or a mortar was cast into the special mold in two different ways. In order to avoid the formation of water layer along the fiber and air voids in the cement paste, the mold, as shown in Fig.2(a), is placed within a vacuum desiccator and a steel fiber was fixed vertically at the center of the mold. The cement paste mixed in advance under a vacuum was poured into the mold. In specimens with a mortar matrix, a steel fiber was set in the direction perpendicular to casting direction, and specimens were made in atmosphere. Specimens were cured in water at 20 °C for 28 days. Pull-out loads were applied to a fiber, as shown in Fig.3. After loading up to a prescribed level, a slice including a steel fiber was cut off at a cross section parallel to the axis of steel fiber. The slice was then placed in a vacuum
desiccator, and the epoxy resin containing a fluorescent dye was impregnated into the slice. After the resin hardened at a room temperature, the cross section of the slice was ground down to a depth so that the longitudinal cross section of an embedded fiber appeared. Interfacial zones were examined with the incident light type fluorescence microscope.

(b) Wedge splitting test -- The shape and the size of specimens are shown in Fig.4. Embedded length of a steel fiber in the cement paste ahead of a notch was 30mm. A crack from the notch propagated as the wedge loading was applied. Cracked specimens which were loaded up to a prescribed level, were impregnated with the epoxy resin containing a fluorescent dye for the fluorescence microscopic examination.

RESULTS AND DISCUSSION

(1) Fracture Process of the Interfacial Zone under Pull-out Loading

(a) Virgin Specimens -- The interfacial zone between a steel fiber and the cement paste matrix in a virgin specimen produced under atmosphere, is shown in Fig.5. In the specimens produced under atmosphere, fibers were arranged horizontally. Bright fluorescent areas showing the impregnation of the resin containing a fluorescent dye into the spaces were seen along the interface, as shown in Fig.5. However, the bright areas due to the water layer around a steel fiber were not found in the interface between the vertical fiber and the paste cast under a vacuum. It was found that the direction of a fiber in the mold affects the formation of the interfacial zone. Fig.6 shows the interfacial zone between a horizontal steel fiber and 10% silica fume cement paste. Bright areas due to bleeding were also found along the interface. However, the width of the areas in this specimen was smaller than that in an ordinary cement paste specimen. Zonal areas with a modest brightness existed between the brightest area and the dark bulk cement paste matrix. The areas seem to be a transition zone in which porosity decreases toward the bulk cement paste phase. The width of the transition zone found in Fig.6 is about a few 10 μm, which agrees with the result from the microhardness measurements[4,5]. Furthermore, comparison in the brightness of the bulk cement paste phase between Fig.5 and 6 indicates that the microstructure of the hardened cement paste with silica fume is much denser than that of ordinary cement paste.

(b) Progressive Debonding during Pull-out Loading -- Fig.7(a) and (b) show the fracture process of the interfacial zones at a loading level of 14.7 N and 29.4
N, respectively, which corresponded to about 25% and 50% of the maximum pull-out load. These loading levels were in the ascending part of the pull-out load-displacement curve, in which partial debonding has occurred along the interface. From these micrographs, both partial debonding zones and bonding zones are found to exist along the interface [8,9,10]. The bright fluorescent areas along the interface showing debonding were only a part of the whole embedded length of a fiber. It should be noted that the fluorescent image along the interface appeared as zonal areas. At a load of 14.7 N, fluorescent areas of about 1.5mm in length and about 100μm in width were found along the interface ahead of the interfacial slit. Such bright areas showing debonding were not found further ahead. At a load of 29.4 N, the bright areas ahead of the interfacial slit extended up to 4mm, and widened by about 500μm. The width of fractured regions reduced with distance from the interfacial slit up to the distance of about 2mm from the interfacial slit. Fig.8 shows the interfacial zone in the 10% silica fume-bearing specimen at the pull-out load of half the maximum. Bright areas showing local failures in conical shape were found around the steel fiber. The greater the shear stresses induced, the wider the bright areas formed in the interfacial zone. The width of the widest area was about 2mm. The end of the conical areas was about 4mm from the interfacial slit. No bright area along the interface was found further ahead of the end of conical areas.

Progressive debonding appears to originate from the end of a slit as generally assumed in theoretical and as obtained in experimental studies of the pulling-out behavior of fibers (Fig.7 and 8). However, the length of a debonded portion was shorter than that estimated in those studies [8,10]. Fig.9 shows the normalized pull-out load vs. interfacial crack length curves obtained theoretically by Stang, Li and Shah [8], and experimentally by Bien and Stroeven[10]. At the loading level of half the maximum, the debonded lengths obtained by both the theoretical and experimental studies were 9mm. These differences in the debonded length obtained between the present experimental study and the theoretical studies may be due to the assumption made in those theoretical models [8,9]. In the theoretical studies, a rigid matrix and a thin boundary layer expressing the matrix shear lag are assumed [8,9]. However, examinations of bright areas formed along the interface in terms of the fluorescence microscope show that these assumptions seem not to be valid. As for the experimental study by Bien and Stroeven [10], they pulled out a steel strip from the sandwich specimen. Differences in stresses field along fibers between their two dimensional model tests and the single fiber pull-out tests in this study may be responsible for the difference in the debonded length between both. Debonding has been generally treated as shear failures along the real interface. As found in Fig.7 and 8, the debonding process along the interface is not simple shear failures between a fiber and the matrix, but the extension of debonding is accompanied by local failures in relatively wide zonal regions surrounding a steel fiber. Furthermore, the
decrease in the width of the zonal areas with distance from the end of a slit also suggests that frictional shear stresses along the debonded portions of interface varied. The assumption of a constant frictional shear stress along the interface appears not to express correctly a real stress distribution. Furthermore, the decrease in the width of bright areas with distance from the end of a slit implies that local failures at the initial stage of loading occurred in wider regions due to the stress concentration near the tip of a slit, and that failures caused by the subsequent pull-out loading were limited to smaller regions adjacent to the steel fiber.

(c)After Catastrophic Debonding.-- Fig.10 (a) and (b) show micrographs of the interfacial zone when the fiber was pulled out up to 1mm of displacement from the cement paste matrix. Bright fluorescent areas along the interface were found over the whole fiber. This result shows that debonding has already occurred along the whole length of fiber. It should be noted that the width of failure zones in the ordinary cement paste is greater than in the silica fume cement paste. Comparing bright areas between the interface at a stage of progressive debonding (Fig.7 and 8) and after the catastrophic debonding (Fig.10), the width of failure regions in the latter is found to be wider than the former. This result indicates that the debonded interfacial regions suffered further breakdown with the subsequent slippage of the fiber through the matrix. The fact that fewer local failures occur in the silica fume cement mortars than in the ordinary cement mortars was also clearly evident in the microscopic examinations, as shown in Fig.11 (a) and (b).

Characteristics of failure regions observed under the fluorescence microscope can be related to the fracture toughness for the interfacial zone obtained by the method proposed by the authors[4]. The critical strain energy release rates for the interfacial zone in various matrices are given in Table 4. The fracture toughness for the interfacial zone is found to be reduced by the addition of silica fume. Particularly, the reduction in the fracture toughness was greater in the silica fume mortars than in the silica fume cement pastes. Although in the silica fume-bearing cement pastes the microstructure is densified both in the interfacial zone and the bulk cement paste phase, there still exists interfacial regions which have lower strength than the bulk cement paste [4,5]. It appears that the microstructure of the hardened cement paste were homogenized by the addition of silica fume. The toughening mechanisms of hardened cement paste are responsible for deflection and branching of cracks due to unhydrated cement particles and Ca(OH)₂ crystals [11]. During the propagation of interfacial cracks, local failures around a fiber in the silica fume-bearing cement paste are limited to smaller regions around the interface because of the presence of denser and more homogeneous bulk phase in the matrix. Therefore, the decrease in the fracture toughness of the interfacial zone in the silica fume-bearing cement paste reflects such characteristic nature of their microstructure.
Aggregate particles act as an arrester of cracks. As shown in Fig. 12, however, fewer sand grains existed in the vicinity of the interface. Therefore, cracks are supposed to have easily propagated in the interfacial zone near the fiber because of a lessening of the effect of the sand grains as a crack arrester. In the steel fiber-mortar composite, interfacial cracks must have propagated not so tortuously as expected in the composite.

(2) Pull-out Process of a Fiber within the Cement Matrix under Wedge Loading

It is difficult to deduce the toughening mechanisms of fibers in the cementitious composites from the single fiber pull-out test. The fluorescence microscopic examinations can give us useful informations on the correspondence between the pull-out behavior of a fiber in the single fiber pull-out test specimens and real composites. The interaction of an inclined fiber with a crack propagating toward the fiber in the cement paste matrix is shown in Fig. 13. A crack path induced by the wedge loading is shifted at the intersection of the crack and the fiber. Branching of the crack at the intersection of a crack and a fiber (Fig. 13(a)) was also found by Bentur, Diamond and Mindess [12]. The interfacial zones near the crack faces were already debonded whereas not in the portions away from the crack faces. The width of bright areas showing debonding as well as local failures is so wide as to reach the bulk cement paste phase. These facts indicate that the progressive debonding along a fiber originated from the intersection of the fiber and the crack propagating toward the fiber. Yielding of parts of the matrix due to the local compression near the fiber was also found as bright triangular areas near the intersection (Fig. 13(b)). It is confirmed from Fig. 13 that the progress debonding and relatively wide zonal areas of failure along the interface assumed in the pull-out test, and yielding of parts of the matrix deduced by the flexural strength test, actually occurred in the composites. These results suggest that in order to produce high performance steel fiber reinforced mortars or concretes, the toughness and the strength of the matrix should be increased.

CONCLUSIONS

The fluorescence microscopy was found to be useful for examining the debonding process along the interface between a steel fiber and the cementitious matrix. The progressive debonding along the interface was confirmed in the pulling-out process of a fiber. Local failures of the extended regions of the
matrix around a steel fiber were also accompanied. The interfacial debonding in the system is not a simple shear failure along the actual interface. Other results obtained in this study are as follows:

(1) Voids due to bleeding and the interfacial zone were clearly confirmed with the fluorescence microscope.

(2) The addition of silica fume led to the reduction in the width of zonal areas with local failures along the interface between a fiber and the cement paste or mortar matrix. The reduction in the areas containing local failures along the interface in the silica fume-bearing specimens was reflected by the decrease of fracture toughness obtained by the single fiber pull-out test for the interfacial zone.

(3) The fluorescence microscopic examinations of the interaction between a crack propagating in the matrix and the fiber bridging the crack indicated that the toughness of the matrix itself contributed to the pull-out energy of fibers.

REFERENCES


### TABLE 1 — PHYSICAL PROPERTIES OF SILICA FUME

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<td>Bulk Density (g/cm³)</td>
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Fig. 1—Diagram of specimen for single fiber pullout test

Fig. 2—Mold for making specimens: a) Fiber fixed in vertical direction; b) Fiber fixed in horizontal direction; and c) Specimen size
Fig. 3—Loading apparatus to pull out a steel fiber for fluorescence microscope examinations

Fig. 4—Wedge opening loaded specimen
Fig. 5—Interfacial zone between a horizontal steel fiber and the cement paste matrix in a virgin specimen.

Fig. 6—Interfacial zone between a horizontal steel fiber and the 10 percent silica fume-bearing cement paste.
Fig. 7—Progressive debonding of interfacial zone between a vertical steel fiber and the silica fume-free cement paste: a) At a load of 14.7 N; and b) At a load of 29.4 N

Fig. 8—Interfacial zone in the 10 percent silica fume-bearing cement paste at the pullout load of half the maximum
Fig. 9—Comparison of debonded length along the interface

Fig. 10—Interfacial zone between a steel fiber and the cement paste matrix after catastrophic debonding: a) In the silica fume-free cement paste; and b) In the 10 percent silica-fume-bearing cement paste
Fig. 11—Interfacial zone between a steel fiber and the mortar matrix after catastrophic debonding: a) In the silica fume-free mortar; and b) In the 10 percent silica fume-bearing mortar.

Fig. 12—Distribution of fine aggregate particles in the interfacial zone.
Fig. 13—Interaction of an inclined fiber with a crack in the cement paste matrix: a) Branching of the crack (COD = 0.1 mm); and b) Yielding of parts of the matrix due to local compression (COD = 0.2 mm)
The Influence of Silica Fume on the Strength of the Cement-Aggregate Bond

by C. Perry and J. E. Gillott

Synopsis:

A small scale flexure test for the determination of cement-aggregate bond strength is described. Cylindrical test specimens were prepared by drilling cores in a perpendicular direction through slabs of rock against which mortar had been cast. A special casting procedure eliminated many sources of experimental variation and allowed the bond strengths of different mortars and rock types to be compared directly. Long term tests were conducted by coring the mortar/aggregate slabs at a number of curing times and coefficients of variation of 5 - 10% for bond and mortar strengths were obtained.

The effect on cement-aggregate bond strength of partial cement replacement by silica fume was evaluated for a number of aggregate types. For siliceous aggregates (glass, obsidian and quartzite), bond strength was increased significantly by the addition of silica fume and failure tended to occur away from the interface particularly in long term tests. For carbonate rocks (limestone and dolostone), similar bond strengths were obtained at seven days with and without the addition of silica fume. At later ages, silica fume interfered with strengthening of the cement-carbonate rock interface and lower bond strengths were obtained. For specimens not containing silica fume, bond strength increased more rapidly to glass and obsidian than to quartzite which showed essentially "inert" behaviour. This was tentatively attributed to strengthening of the transition zone by a pozzolanic mechanism involving reactive silica from the aggregate. A marked reduction in bond strength occurred with glass specimens containing boosted alkali content. This was attributed to alkali-silica reaction at the interface and was suppressed by addition of 15% silica fume.

Keywords: Aggregates; alkali-aggregate reactions; bonding; carbonate aggregates; cements; flexural strength; pozzolanic reactions; silica fume; siliceous aggregates; strength
Biographical Sketch

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ACI member Jack E. Gillott is an adjunct professor in the Department of Civil Engineering at the University of Calgary, Alberta, Canada. He received his Ph.D. from the University of Liverpool and his D.Sc. in engineering from the University of London. His research interests include concrete durability and the influence of microstructure and composition on the behaviour of materials.
INTRODUCTION

The interfacial region between the cement paste and the aggregate is considered to play a significant role in determining such properties as the strength, permeability and durability of concrete. It has been the subject of numerous studies, particularly in recent years (1, 2, 3) but considerable scope still exists for further research. The influence of aggregate mineral composition on the interfacial region, or so-called "transition zone", has received limited attention and progress in this area has been hampered, to some extent, by the lack of a reliable test for determining the strength of the cement-aggregate bond. The current work describes the development of a test procedure that is believed to offer a number of advantages over methods previously used. Although intended primarily for use in studying the influence of aggregate properties, the procedure was used to evaluate the effect on bond strength of partial cement replacement by silica fume. The present paper describes the test procedure and presents data on the influence of silica fume, cement alkali content, and rock type on the strength of the cement/aggregate bond.

EXPERIMENTAL DETAILS

Test Development

An essential part of the study was the development of a bond test procedure that could be used, in conjunction with other methods, to study the influence of aggregate mineralogy on the development of the transition zone between the aggregate and the bulk cement paste. A number of aggregate types were to be tested over a range of curing times and a simple flexural test procedure was considered appropriate rather than the time-consuming measurement of fundamental fracture properties.

A number of methods have been used in previous investigations to measure the tensile strength of the cement/aggregate bond. These include direct tension (4, 5, 6), flexure (1, 7, 8, 9), and splitting tension (10, 11).

A common factor evident in those studies which have included detailed experimental information is a large scatter of results for supposedly identical specimens. Coefficients of variation for bond strength have ranged from fifteen to thirty percent (1, 4, 7, 8) Such variability has required the preparation and testing of large numbers of specimens and has caused difficulty in detecting small differences or trends in bond strength.

In the majority of studies, test specimens have been prepared individually by casting cement paste or mortar against the prepared surfaces of small cubes, cylinders, or prisms of rock. This procedure is time-consuming and introduces "between specimen" variations in such factors as aggregate surface finish, degree of paste or mortar consolidation, and time between mixing and casting.
Preliminary tests by the authors using a cantilever test apparatus similar to that described by Alexander (7) confirmed the high variability of such methods. A prominent source of variation was the occurrence of small imperfections on the outside of the specimens at the mortar/aggregate interface. These were caused by entrapped air bubbles or local accumulations of mould release oil and had a marked effect on the measured flexural bond strength.

The approach used by Scholer (8) eliminated many of these sources of experimental error. Mortar was cast against thin slabs of rock and a number of small, cylindrical cores were obtained by drilling perpendicular to the interface. Bond strength was then determined by testing the cores in flexure. A similar technique, using sawn specimens, was mentioned by Alexander (1).

Scholer's method was considered ideal for monitoring small changes in bond strength over time and was used in the present work, and an earlier study (12), with a number of important modifications.

Entrapped air bubbles at the interface and in the body of the mortar have a marked effect on the bond and mortar strengths of small specimens. Air bubbles were successfully removed by placing the freshly cast mortar/aggregate slabs in a vacuum where they were subjected to prolonged vibration. This made it possible to obtain good reproducibility using specimens with a diameter of only 10.5mm.

The flexure test apparatus developed after extensive preliminary trials is shown in Fig. 1. Alexander (7) and Scholer (8) both used a clamped cantilever arrangement which would introduce additional and variable stresses at the interface and the lack of a defined failure plane caused difficulty in measuring mortar strength (8). As shown in Fig.1, the specimen was held between two curved supports having the same diameter as the specimen and load was applied by a plunger via a cylindrical roller fabricated from two small ball races. The upper curved support was knife-edged thus fixing the location and magnitude of the maximum bending moment. This arrangement gave a well-defined loading configuration and eliminated longitudinal restraint in the failure region. The importance of restraint was indicated in earlier tests using a knife-edged plunger which produced very variable results.

Testing was carried out under water, the importance of which has been emphasized by Alexander (1) and Hsu and Slate (4) and was confirmed in the present study. Dramatically lower bond and mortar strengths were obtained when the small cores were allowed to dry for only a few minutes.

The ability to obtain all test specimens from the same mortar/aggregate slab made it possible to minimize experimental variations and allowed some long term changes in bond strength to be detected (12). When studying the influence of silica fume and rock type on bond strength, the use of a two compartment...
mould helped in a similar manner to eliminate some sources of experimental variation. Different rocks could be used in each compartment of the mould with the same mortar cast against them under identical conditions. Alternatively, the same rock type could be used in each compartment with different mortars cast against them at the same time. In later tests, for example "80Q" in Table 2, the two alternatives were combined and each compartment contained the same two rocks (obsidian and quartzite) and either the control mortar or one containing silica fume. Fig. 2 illustrates this approach.

TEST SCHEDULE AND MATERIALS

Table 2 summarizes all the mortar mixes and mortar/aggregate combinations used to study the influence of silica fume on bond strength. The tests were carried out in two stages.

Initially, slabs of window glass were used with mortars having water/cementitious ratios ranging from 0.35 to 0.55 and either zero or ten percent silica fume. All mixes were proportioned to have a flow of approximately 100 (ASTM C124-71) which was achieved by increasing the sand content in those mixes having higher water/cementitious ratios and by using a superplasticizer (naphthalene sulphonate type) when silica fume was present. The sand consisted of two size fractions (75% 315 μm - 630 μm and 25% 160 μm - 315 μm) obtained from a local, unreactive concrete sand. Three water/cementitious ratios were used, 0.35, 0.40, and 0.55, and the corresponding sand/cementitious ratios were 1.0, 1.5, and 2.4 respectively. Silica fume was incorporated as a partial cement replacement by mass and a small adjustment was made to the 160 μm - 315 μm sand size fraction to allow for its increased volume. A low alkali cement was used in all mixes but in three instances the alkali content was boosted from 0.44% to 1.25% (equiv. Na₂O) by the addition of NaOH to the mixing water. The high alkali mixes are identified in Table 2 by an "A" at the end of the "SLAB ID".

In the second test series, a number of different rocks were used with mortar having either zero or ten percent silica fume and, in most cases, a water/cementitious ratio of 0.35.

Physical and chemical properties of the cement (ASTM Type I) and silica fume used in the study are summarized in Table 1. Properties of the glass and the rocks are described below and illustrated in Fig. 3:

GLASS - "G": Commercial window glass.
OBSIDIAN - "O": A glassy obsidian obtained from Wards Scientific; Source - Lake County, Oregon.
QUARTZITE - "Q": Mount Wilson Quartzite. A quartzite consisting of interlocking anhedral crystals of quartz (0.1 - 0.5 mm) showing strain extinction. Very small amounts of impurities, possibly clay or micaceous
minerals, are present.
Limestone - "L": Exshaw Limestone. A limestone containing oolites (0.25 - 0.50mm) and larger fossil fragments (0.5 - 1.5mm) in a matrix of fine calcite crystals which reach about 0.15mm; larger crystals show well developed cleavage and twinning. Some dolomite and non-carbonate minerals are also present.
Dolomite - "D": Nelson Dolomite. A dolostone composed of interlocking crystals of dolomite (0.05 - 0.2mm) which display euhedral rhombic morphology where they project into small cavities (0.5mm) which are relatively common.

PROCEDURES

Slabs of rock, 10 - 15 mm thick, were cut from single boulders, perpendicular to any bedding planes, and their surfaces were finished on a steel wheel using 120 mesh silicon carbide grit. The grit was replaced frequently to obtain the roughest surface possible. Surface roughness was measured using a Talysurf 4 stylus instrument and centre line average values ranged from 4.2 μm for the quartzite to 4.7 μm for the glass.

The slabs were kept moist until just prior to casting when they were surface dried using clean, compressed air. A standardized mixing sequence based on ASTM C305 was used for all mortars and consolidation was carried out on a small vibrating table using a vacuum chamber. Under continuous vibration, the vacuum was slowly increased to 60 - 65 cm of mercury, held at that value for four minutes, and then slowly decreased, giving a total cycle time of eight minutes.

The mortar/aggregate slabs were cured under water in a fog room for one day and then stored in a saturated lime solution. At each testing age the required number of test specimens was removed by coring, the rock side of the slabs being drilled first. Tests in which dye was painted on the inside of the core barrel showed that no contact was made between the core and the core barrel during drilling. Damage due to hydraulic loading of the cores by the drilling water was probably slight since specimens of very low bond strength could be obtained with only occasional breakage (Table 2).

For the determination of bond strength, the cores were positioned in the test apparatus described above (Fig.1) so that the interface was just slightly above the upper, knife-edged support. A proving ring/LVDT arrangement was used to transfer load to the specimen through the roller and the peak value was recorded. In some cases, mortar cores were drilled alongside the rock slabs as shown in Fig.2 and tested in a similar manner. All cores were submerged in water from the commencement of drilling until the end of testing.
RESULTS

General

The test results are summarized in Table 2. The number and letters at the beginning of each "SLAB ID" indicate the batch in which the mortar/aggregate slab was cast and the aggregate types used. Examples of the identification codes used are given with Table 2.

Bond strengths were determined at seven days and at a number of intermediate times up to the maximums given in Table 2. When clean separation of the mortar from the aggregate did not occur, the area of failed interface was estimated using a low power microscope. The exact failure path was not determined and bond failure was taken to occur in those areas where mortar was not adhering. When failure occurred exclusively through the mortar phase, the symbol ">" is shown in Table 2 in front of the measured values since the actual corresponding bond strengths were clearly higher.

Strength determinations were made on five to ten cores to obtain the average values given in Table 2. Normally five were sufficient to give a satisfactory coefficient of variation which averaged 6.9% for all bond tests and 4.6% for the control mortar tests. The bond strength of the limestone showed the highest variability at 9.4% and this was probably due to the larger crystal size and less homogeneous nature of the rock. The silica fume mortars were more variable and gave slightly lower strengths than the corresponding control mixes. This may be a result of the more viscous nature of the silica fume mortars and the increased difficulty in removing the last traces of entrapped air.

The strength values given in Table 2 are high and this may be accounted for by three principal factors: tests on larger specimens confirmed a marked specimen size effect, use of the flexural formula may overestimate the strength of round specimens, and levels of entrapped air were unusually low.

INFLUENCE OF SILICA FUME

Glass: At a testing age of 7 days, substantially higher bond strengths were produced by specimens containing silica fume than by control specimens not containing silica fume. For mixes with water/cementitious ratios of 0.35 and 0.40, bond strength increased by an average of 80% when silica fume was incorporated in the mix. The test results were very consistent and agreement between mortar/aggregate slabs was good. Clean separation between the glass and the mortar occurred for the control mixes containing no silica fume, whereas when silica fume was present, fracture took place partially through the mortar phase leaving small areas of mortar adhering to the glass surface. At a water/cementitious ratio of 0.55, the effect of silica fume was less pronounced despite the higher replacement level of 15%. The presence of silica fume
increased the bond strength by 25% and, as at lower water/cementitious ratios, it caused partial failure in the mortar away from the interface.

At a testing age of 28 days, the bond strength of the control mixes approached the corresponding mortar strengths and partial failure occurred through the mortar phase adjacent to the interface. Bond failure areas ranged from 40% to 90% of the total fracture surface at the three water/cementitious ratios studied. For the mixes containing silica fume, however, bond strength surpassed mortar strength and failure occurred almost exclusively through the mortar at a distance of 0.5 - 1.0 mm from the interface. At testing ages exceeding approximately three months, the control specimens also failed exclusively through the mortar phase.

The above behaviour is summarized in Fig. 4 for mixes with water/cementitious ratios of 0.35 and 0.40. The mortar strength curve represents the average behaviour of all mixes expressed relative to the 28 day value. Experimental points on the two bond strength curves, for specimens with and without silica fume, are expressed relative to the corresponding 28 day mortar strength. Bond strength of the control mixes increased slowly at first but reached an average value of 98.5% of the mortar strength at 28 days. Mortar strength was exceeded at an age of two to three months. When silica fume was present, bond increased rapidly and exceeded mortar strength at about 14 days.

Obsidian: The behaviour of the obsidian specimens was similar to that of the glass specimens. At 7 days, the control specimens showed clean separation at the interface, whereas the silica fume specimens showed only 30% bond failure and failed at higher strengths. Failure at later ages occurred through the mortar in both cases. The bond strength at 7 days was higher for obsidian than for glass for both the control and the silica fume mixes but did not appear to increase as rapidly with age.

Quartzite: The behaviour of the quartzite specimens was different from that of glass and obsidian. The bond strength of the control specimens increased with age but only very slowly and reached not much more than 60% of the mortar strength during the period studied (Fig. 5). Failure occurred cleanly at the interface with only traces of mortar adhering to the rock surface even after extended periods of curing. The presence of silica fume in the mix had no marked effect on the bond strength at 7 days but, as in the case of the glass and obsidian specimens, considerable strengthening occurred at later ages (Fig. 6). In the "10DQ" quartzite tests, the bond strengths of the control and silica fume mixes were unusually high and differed considerably from the "80Q" and "90Q" results. Between batch variations were typically much less pronounced and no obvious departure from the standardized experimental procedures had been made. Probably a major error such as use of the wrong cement had occurred. Only the "80Q" and "90Q" test results were used to plot the curves shown in Figs. 5 and 6 although the "10DQ" values, which showed essentially the same trends, were
considered when estimating the long term behaviour of quartzite with the silica fume mixes (Fig.6).

Limestone and Dolostone: Different behaviour again was observed in the limestone and dolostone tests. Both control specimens showed a slow but steady increase in bond strength with time and after extended curing came close to, but did not exceed, the corresponding mortar strengths. Silica fume caused no increase in the 7 day strength, or even caused a slight decrease, and it appeared to inhibit later strength gain. Failure occurred cleanly at the interface for the silica fume mixes even after extended curing and the bond strength remained low. Only the limestone results are plotted in Figs. 5 and 6 since the bond strength values in the "10DQ" (dolostone) tests were unusually high and no tests were performed at intermediate ages. Overall, the behaviour of the limestone and dolostone appeared to be similar.

**Influence of Alkali**

The control specimens containing increased alkali content and glass surfaces (5G40CA and 7G55CA) showed a pronounced loss of bond strength beyond about 7 days (Fig.7). When 15% silica fume was incorporated in the mix (7G55SFA), no reduction in bond strength occurred even after curing for as long as three years.

**Influence of Aggregate Type**

The change in bond strength over time for some of the rock types studied is compared in Fig. 5 for the control specimens and in Fig. 6 for the specimens containing silica fume. The figures illustrate the different behaviour of the limestone and quartzite specimens with and without silica fume. The limestone bond strength increased steadily in the control mixes but levelled out at a low value in the presence of silica fume. Quartzite, on the other hand, produced a relatively low bond strength in the control specimens but the strength increased rapidly when silica fume was present. The development of bond strength was therefore in sharp contrast for these two types of rock.

**DISCUSSION**

**Influence of Silica Fume**

A number of studies have indicated that when silica fume is incorporated in concrete the transition zone between the aggregate and the bulk cementitious phase becomes denser and less distinct (13, 14, 15). Silica fume particles, being smaller than cement, are thought to pack more densely near the interface and may act as nucleation sites for hydration products. In addition, calcium hydroxide, which tends to accumulate in this area and provide a source of weakness, reacts pozzolanicly with silica fume to produce stronger hydration products.
Strengthening of the transition zone is supported by a growing body of microscopic and other evidence (13, 14, 16, 17) but few studies have included the measurement of bond strength.

Odler and Zurz (11) reported increased bond strength for basalt, quartzite, marble and limestone with cement pastes containing five and ten percent silica fume, and similar results were obtained by Chen and Wang (18) for calcite. Silica fume also caused marked increases in interfacial fracture energy for calcite tested in flexure (Wang et al. (9)) and similar results were found by Mitsui et al. (17) for limestone tested in shear.

For siliceous aggregates (glass, obsidian and quartzite), the authors' work is in basic agreement with that of Odler and Zurz. The addition of silica fume caused a marked increase in bond strength which took place rapidly for glass and obsidian but was not observed until later ages for quartzite. This behaviour is illustrated in Fig. 4 for glass and Figs. 5 and 6 for quartzite. As previously noted, a superplasticizer was added to mixes containing silica fume to achieve comparable workability to the control mixes. The ability of this admixture to disperse cement particles and accelerate hydration may have contributed, at least at early ages, to the increased bond strengths. However, since some aggregates (quartzite, limestone, and dolostone) gave similar bond strengths at seven days, with and without silica fume, the influence of the admixture is believed to be small.

In the absence of silica fume, the carbonate rocks, limestone and dolostone, showed a steady increase in bond strength with time (Fig. 5 and Table 2) and at later ages bond strength approached mortar strength. Previous studies of carbonate rocks have given similar results (4, 5, 6, 10). When silica fume was present, however, very little increase in bond took place (Fig. 6 and Table 2) and long term strengths were much lower than those of the controls. Reduced tensile bond strengths with silica fume have not been observed previously (9, 18) but much smaller increases have been reported for carbonate rocks than for siliceous ones (11). Also, Mindess and Diamond (19) concluded from observations of fracture surfaces that the dolomite-silica fume paste interface appeared to be weaker than the andesite-silica fume paste interface. It is possible that under the conditions of the present study, silica fume interfered with the strengthening mechanism of carbonate rocks by reacting with calcium hydroxide in the interfacial region. The strengthening mechanism has not been identified with certainty but the formation of lime-rich \(3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{CaCO}_3 \cdot 11\text{H}_2\text{O}\) (15) and a basic calcium carbonate hydrate (14) have been proposed. Also, Struble (20) suggested that an observed reduction in calcium hydroxide in the transition zone of limestone was due to chemical reaction with the rock.

Tensile failure requires only a single plane of weakness, in this case the interface, and reduced bond strength does not conflict with extensive microscopic evidence for densification of the transition zone by silica fume.
Influence of Aggregate Type

The bond strengths of glass, quartzite and limestone are compared in Fig. 5 for the control specimens. Quartzite may be considered an "inert" aggregate, meaning that its bond strength roughly parallels mortar strength (4, 6), whereas limestone undergoes some type of strengthening process, as discussed above.

The bond strengths of glass and obsidian (not shown in Fig. 5) also increase with time and quite rapidly exceed mortar strength. Similarly, the bond strength of basalt has been shown to surpass mortar strength at about six months (1). Alexander (1) proposed a pozzolanic mechanism for strengthening of the cement/aggregate bond and related bond strength (mean of 7 and 28 day values) to the silica content for a number of extrusive rocks. It is possible that the transition zone of aggregates such as glass and obsidian which undergo mild alkali-silica reaction could also be strengthened by pozzolanic reaction. The alkali content of the cement and reactivity of the rock would be significant factors in this regard.

Influence of Alkali

The reduction in bond strength to glass of mortars with boosted alkali content (Fig. 7) is in agreement with earlier work by the authors (12) where similar reductions were attributed to alkali-silica reaction at the interface. The bond test procedure, therefore, may be useful for identifying alkali reactive rocks and perhaps distinguishing between proposed mechanisms of expansion. None of the unboosted specimens showed a reduction in bond strength even at later ages and, as expected from mortar bar expansion tests, fifteen percent silica fume was effective in preventing a reduction.

CONCLUSIONS

(1) A test procedure has been developed for determining the flexural strength of the cement-aggregate bond which allows relatively small differences or changes in bond strength to be detected. The bond strength of different mortars or rock types can be compared directly and can be followed over extended periods of time. Coefficients of variation for bond and mortar strengths in the range 5 - 10% were obtained.

(2) For siliceous aggregates (glass, obsidian and quartzite), bond strength was increased significantly by the addition of silica fume and failure tended to occur away from the interface particularly in long term tests.

(3) For carbonate rocks (limestone and dolostone), similar bond strengths were obtained at seven days with and without the addition of silica fume. At later ages, silica fume interfered with strengthening of the cement-aggregate interface.
and lower bond strengths were obtained for the silica fume specimens than for controls.

(4) For specimens not containing silica fume, bond strength increased more rapidly to glass and obsidian than to quartzite which showed essentially "inert" behaviour. This was tentatively attributed to strengthening of the transition zone by a pozzolanic mechanism involving reactive silica from the aggregate.

(5) A marked reduction in bond strength occurred with glass specimens containing boosted alkali content. This was attributed to alkali-silica reaction at the interface and was suppressed by addition of 15% silica fume.

REFERENCES


(3) Diamond, S., Proceedings of 8th International Congress on Chemistry of Cement, (Brazil), V.1, 1986, pp. 122-147.


### TABLE 1 — PROPERTIES OF PORTLAND CEMENT AND SILICA FUME

<table>
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<tr>
<th>Chemical composition</th>
<th>Portland cement (percent)</th>
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*Examples of abbreviations used:

"5G40CA": Batch number 5, glass aggregate, 0.40 water/cementitious ratio, control (no silica fume), and boosted alkali content.
"80Q3SSF": Batch number 8, obsidian and quartzite aggregates, 0.35 water/cementitious ratio, silica fume addition."
Figure 1. Apparatus for determining bond strength in flexure (all dimensions in mm).

Fig. 1—Apparatus for determining bond strength in flexure (all dimensions in mm)
Fig. 2—Underside view of paired mortar/aggregate slabs

a) Dolomite, Nelson Quarry, Ontario, (Ord. light)  
b) Mount Wilson quartzite (crossed polarizers)  
c) Limestone with oolites, Exshaw, Alberta (ordinary light)  
d) Limestone with fossil fragments, Exshaw, Alberta (crossed polarizers)

Fig. 3—Optical micrographs of rock types
Fig. 4—Bond strength to glass expressed as percentage of 28-day mortar strength — With and without silica fume, (w/c = 0.35 and 0.40)
Fig. 5—Bond strength expressed as percentage of 28-day mortar strength (S.F. = 0 percent; w/c = 0.35 and 0.40)
Fig. 6—Bond strength expressed as percentage of 28-day mortar strength
(S.F. = 10 percent; w/c = 0.35 and 0.40)
Fig. 7—Influence of cement-alkali content on bond strength to glass expressed as a percentage of 28-day mortar strength (average data, w/c = 0.40 and 0.55; S.F. = 0 percent)