

Mechanical Properties of the Interfacial Transition Zone: A Review

by S. Mindess

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	σ	= concrete strength (compression or flexure)
	b_0, b_1, b_2	= linear regression coefficients
		= 480, 2.08, 1.02, respectively, for compression
		= 290, 0.318, 0.162, respectively, for flexure
	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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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), Beaumont 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

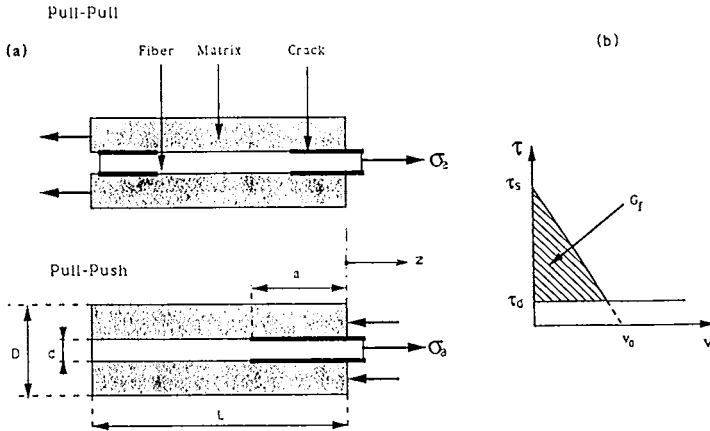


Fig. 1—**a)** Geometry of fiber pullout tests analyzed; and **b)** Interface shear stress versus relative displacement

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 τ versus relative stress displacement v , as shown in Fig. 1b, where τ_s = initial bond strength (initial cohesion), τ_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 σ_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 ϕ .

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 τ_d .

(2) The final stage, in which the two cracks have joined into one and the residual strength τ_d has been reached at both ends. (3) The intermediate stage, in which one must distinguish two cases: (a) The two cracks join before τ reaches τ_d at the ends, (b) the shear stress τ_d is reached before the cracks join.

The solution of the maximum value of σ_a yields the following result:
For $\omega L < 2 \arccos \frac{\tau_d}{\tau_s}$:

$$\sigma_a = \text{Min} \left(\frac{8\tau_s}{\omega d} \sin \frac{\omega L}{2}, \frac{8\tau_s}{\omega d} \sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} \right) \quad (4)$$

Otherwise:

$$\sigma_a = \frac{8\tau_s}{\omega d} \sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} + \frac{4\tau_d}{\omega d} \left(\omega L - 2 \arccos \frac{\tau_d}{\tau_s} \right) \quad (5)$$

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, ωd and ωL are both proportional to the $\sqrt{d}$). The applied pullout stress σ_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,

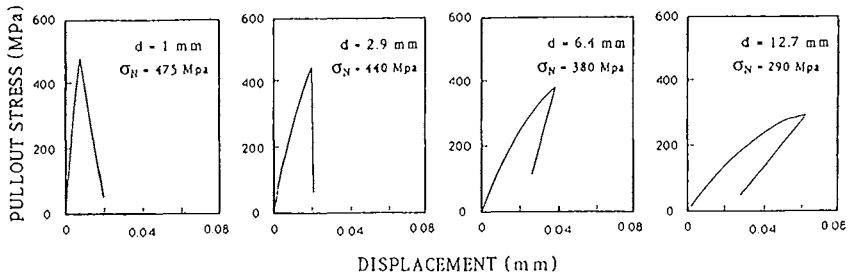


Fig. 2—Diagrams of (pullout stress) versus displacement for tests of similar geometry and different sizes

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 τ_s and τ_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 σ_0 is the residual fiber strength corresponding to the residual interface bond stress τ_d .

Now the asymptotic behavior will be examined. In the limit of small sizes, $d \rightarrow 0$,

$$\sigma_N = 4\tau_s \frac{L}{d} = \text{constant} \quad (6)$$

In the limit of the large sizes, $d \rightarrow \infty$,

$$\sigma_N = \sigma_0 + \frac{\sqrt{8\tau_s E_f v_0}}{\sqrt{d}} \left(\sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s} \right) \quad (7)$$

Here σ_0 is the residual pullout stress of the fiber when the interface is completely debonded and softened to τ_d ,

$$\sigma_0 = 4\tau_d \frac{L}{d} \quad (8)$$

According to (7), the basic form of the size effect for the large sizes is

$$\sigma_N - \sigma_0 \propto \frac{1}{\sqrt{d}} \quad (9)$$

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:

$$\sigma_N = \frac{Bf'_t}{\sqrt{\beta}} \sin \sqrt{\beta} \quad \text{if} \quad \beta \leq \frac{\tau_d^2}{4}; \quad \sigma_N = \frac{Bf'_t}{\sqrt{\beta}} \quad \text{if} \quad \beta > \frac{\tau_d^2}{4} \quad (12)$$

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 (Bažant, 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 (Bažant, 1987; Bažant and Pfeiffer, 1987; Bažant 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 asymptotes with equations (6) and (7). This yields

$$Bf'_t = 4(\tau_s - \tau_d) \frac{L}{d}, \quad d_0 = \frac{8\tau_s E_f v_0}{(Bf'_t)^2} \left(\sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s} \right)^2 \quad (13)$$

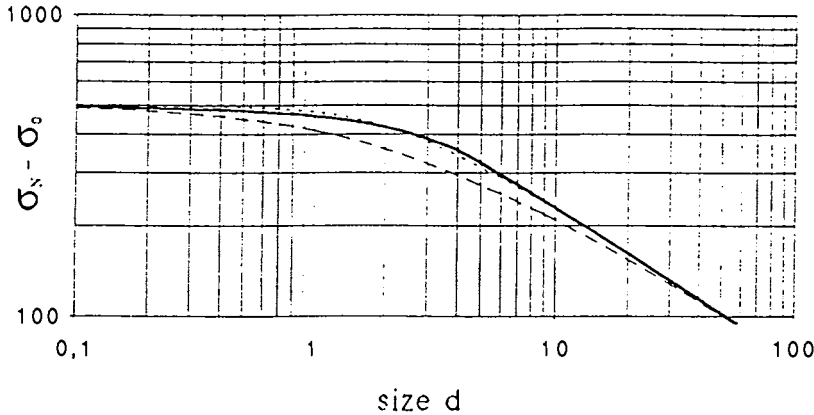


Fig. 3—Comparison of calculated size effect (solid curve) to approximate size effect law proposed by Bažant (dotted curve) and generalized size effect law proposed by Bažant (dashed curve)

Except for the presence of σ_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

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

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

For materials with residual strength, represented here by τ_d , σ_N must be replaced with $\sigma_N - \sigma_0$ where σ_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;

$$\text{for } \omega L \leq \arccos \frac{\tau_d}{\tau_s} : \quad \sigma_N = \frac{4\tau_s}{\omega d} \sin \omega L \quad (14)$$

$$\text{for } \omega L > \arccos \frac{\tau_d}{\tau_s} : \quad \sigma_N = \sigma_0 + \frac{4\tau_s}{\omega d} \left(\sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s} \right) \quad (15)$$

in which ω 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 Bf'_t and d_0 . Then, matching the asymptotes yields the following expressions for the interface properties:

$$\tau_s = \frac{d}{4L} Bf'_t + \tau_d \quad (16)$$

$$v_0 = \frac{1 + \phi}{4} \frac{(Bf'_t)^2}{\tau_s E_f} d_0 \left(\sqrt{1 - \frac{\tau_d^2}{\tau_s^2}} - \frac{\tau_d}{\tau_s} \arccos \frac{\tau_d}{\tau_s} \right)^{-2} \quad (17)$$

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/m². The value of τ_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.

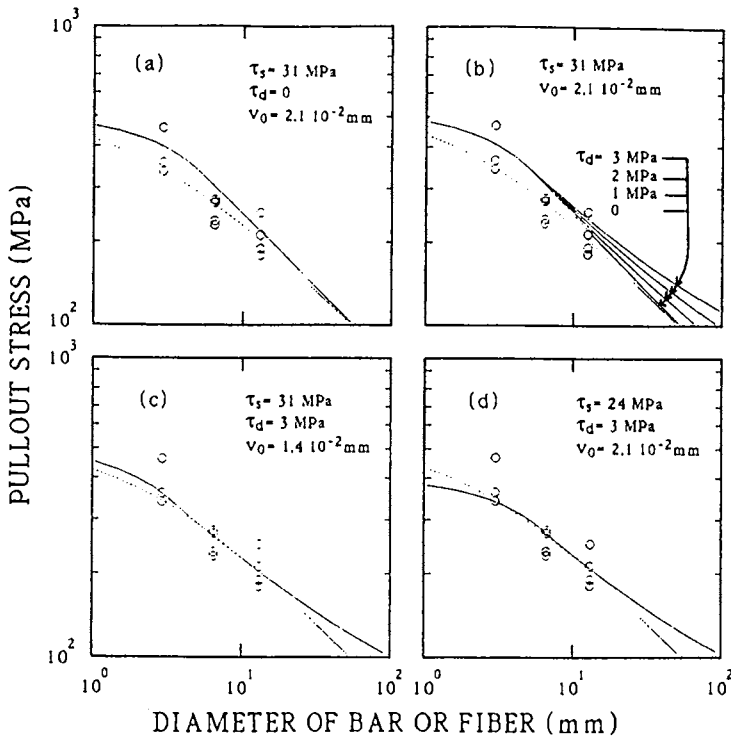


Fig. 4—Test data for bar pullout, used as an example, and comparisons with present solution (solid curves) and general approximate form of the size effect law of Bazant (dashed curve)

When the size effect law is to be matched to experimental data on σ_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 $m = 1$: $Bf'_t = 1/\sqrt{C}$ and $d_0 = C/A$.

To demonstrate the procedure, we will use the test data of Bazant 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 (Bazant and Sener, 1988) were $Bf'_t = 500$ 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 Bažant (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-

tes and polypropylene or steel fibers are summarized in later sections. More details of the complete program are given by Taerwe (3). 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 ℓ). 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 feed back 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 feed back 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 Δh 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, Δh 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 Δh .

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 $0.75f_c$, a spontaneous unloading in the σ - ε 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 $0.6f_c$, 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 $0.2f_c$, 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 $\varepsilon_t = \Delta d/d$, 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 $0.6f_c$. 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 σ - ϵ_i 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

$$\omega = V_d/V = (V - V_n)/V \quad (1)$$

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 ω is the same for specimens with the same h/d -ratio. If the hypothesis can be extended to the evolution of ω 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 $\sqrt{f'_c}$ -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 $35 \times 250 \mu m$ and $35 \times 660 \mu m$ 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/m³) significantly improves the toughness of HSC. Also the addition of polypropylene fibers (5 kg/m³) results in a slightly improved toughness.

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TABLE 1 — SURVEY OF TEST SPECIMENS (BASIC SERIES)

Designation	Diameter, mm	Height, mm	Compressive strength, MPa	ε_1^* , per thousand
HS1	155	380	87.5	2.67
HS2	155	380	89.7	2.54
HS3	155	380	93.0	2.66
HS4	155	380	93.0	2.78
HS5	192	483	96.6	2.76
HS6	192	483	96.6	2.71
HS7	192	483	94.8	2.77
HS8	155	380	90.9	2.67
HS9	155	380	95.2	2.82
HS10	155	380	94.1	2.70
HS11	155	380	94.1	2.74

* ε_1 is the axial strain at peak stress

ACKNOWLEDGMENT

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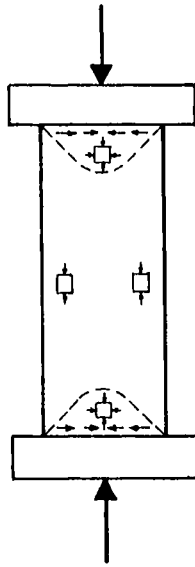


Fig. 1—Disturbed zones in specimens with $h/d = 2.5$

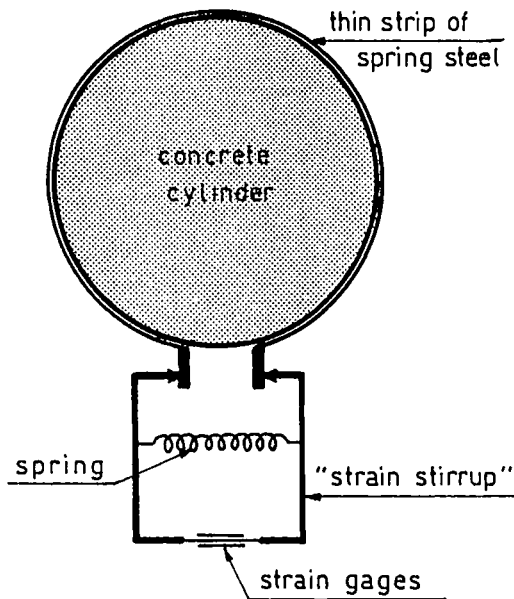


Fig. 2—Strain stirrup for measurement of circumferential strain

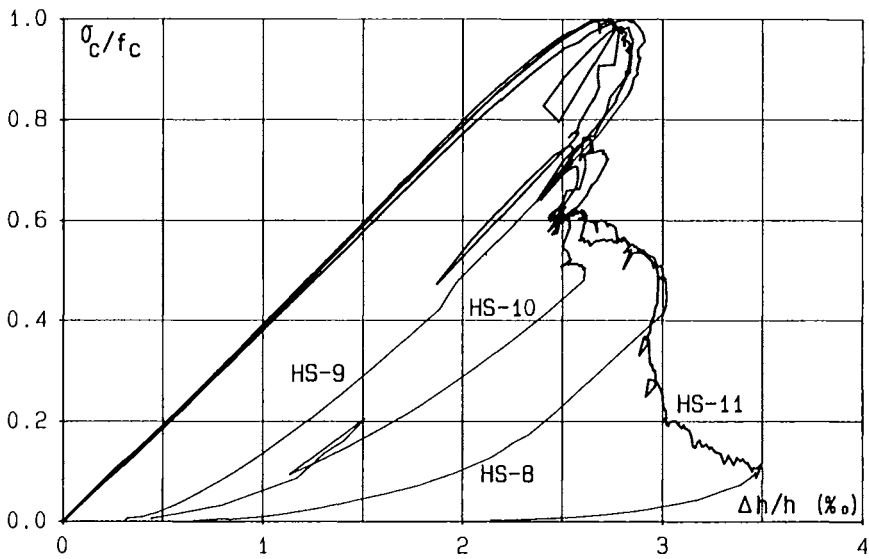


Fig. 3—Stress-axial strain curves of four specimens

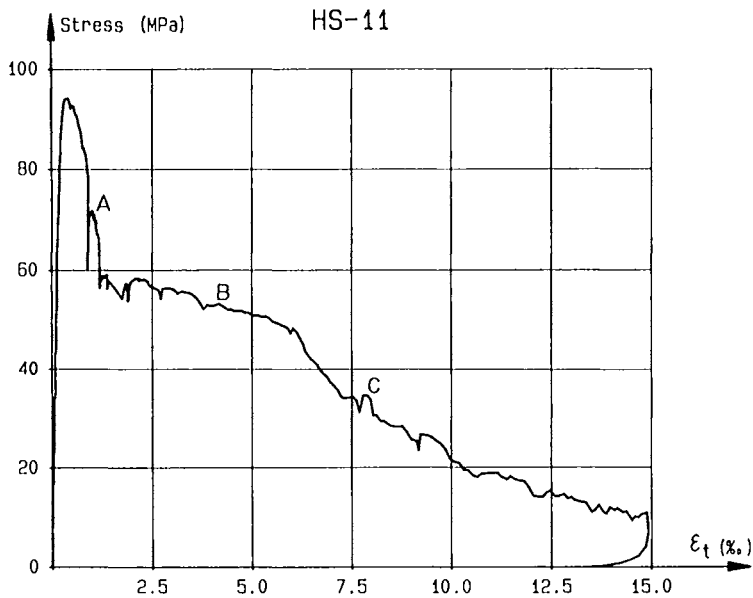


Fig. 4—Circumferential strain for specimen HS11

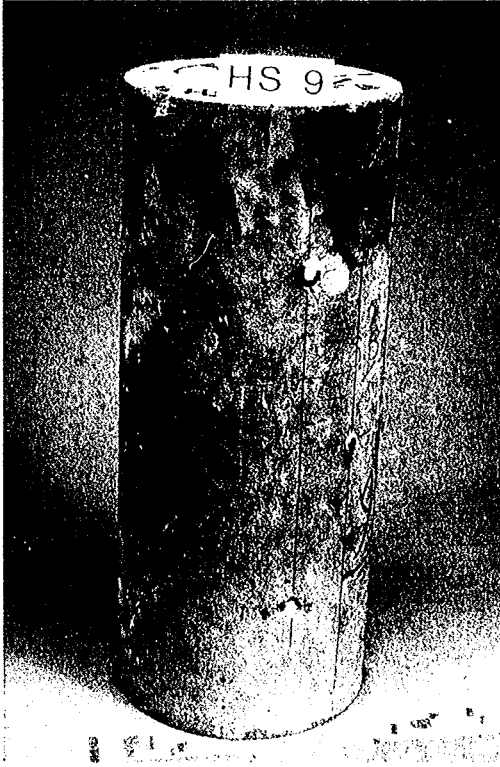


Fig. 5—Specimen HS9 after loading

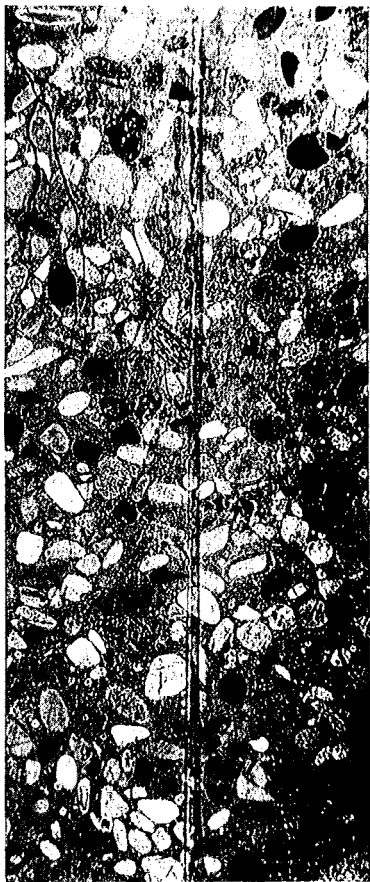


Fig. 6—Cross-section of specimen HS10



Fig. 7—Detail of upper left corner of Fig. 6

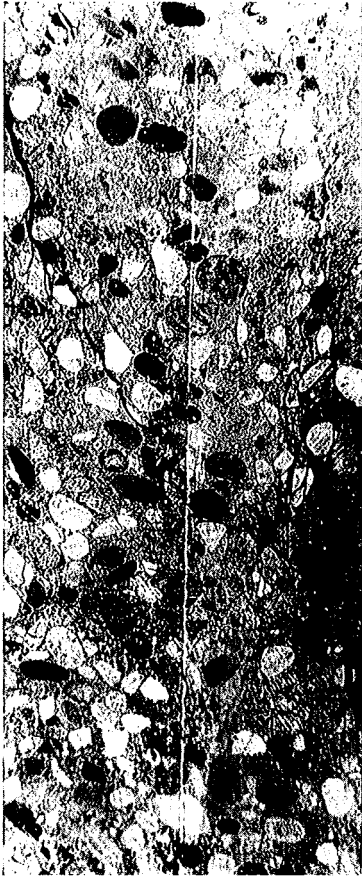


Fig. 8—Specimen HS11



Fig. 9—Specimen HSR5

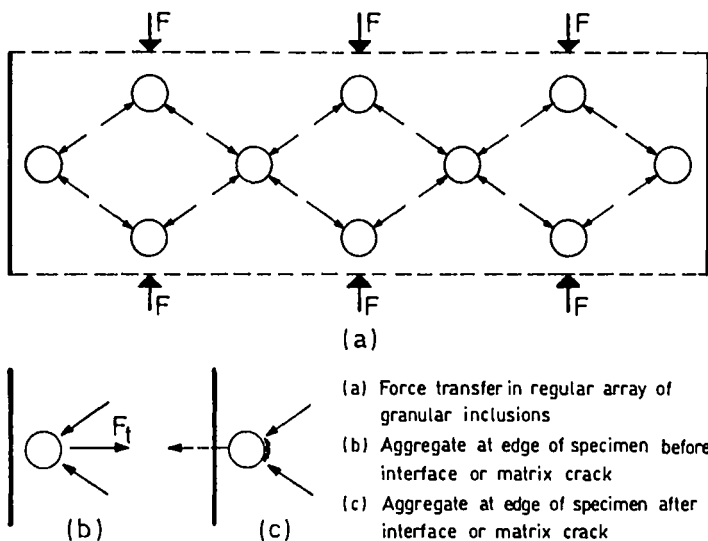


Fig. 10—**a)** Force transfer in regular array of granular inclusions; **b)** Aggregate at edge of specimen before interface or matrix crack; and **c)** Aggregate at edge of specimen after interface or matrix crack

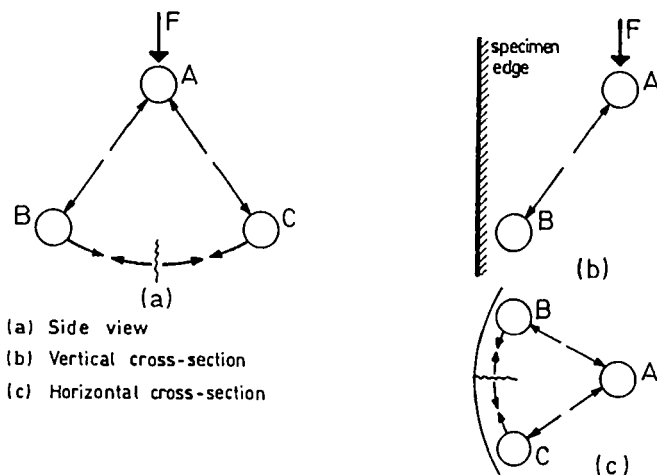


Fig. 11—**a)** Side view; **b)** Vertical cross-section; and **c)** Horizontal cross-section

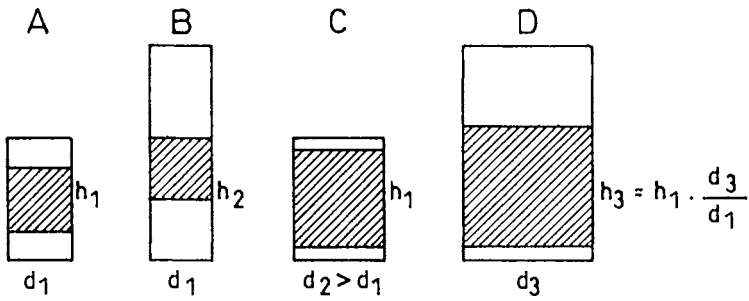
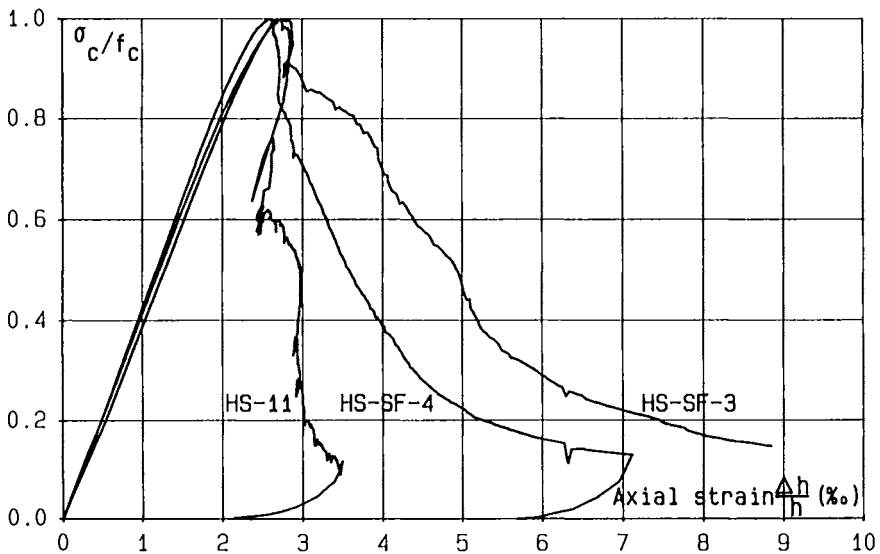
Fig. 12—Characterization of size effect by h/d ratio

Fig. 13—Influence of steel fiber addition on stress-strain curve of HSC

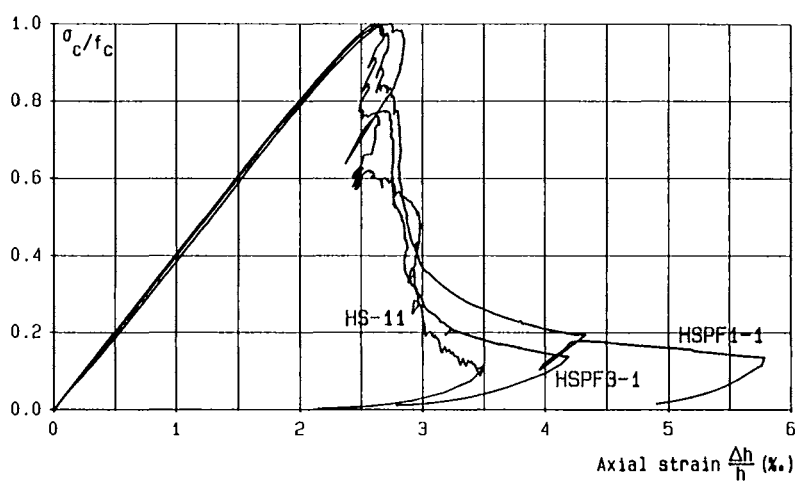


Fig. 14—Influence of polypropylene fiber addition on stress-strain curve of HSC

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

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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) :

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

where σ is the near-tip stress, λ 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 noticeable.

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 (0.51×10^{-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 (0.51×10^{-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 $f_t = 1.5$ MPa (220 psi), and the critical interface crack opening displacement $\text{COD}_c = 0.013$ mm (0.0005 in.). This COD_c was measured at the crack 'mouth', and can be also designated as the 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 ν_1 occupies the upper half-plane, $y > 0$, and a material with elastic properties E_2 and ν_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 :

$$\frac{\sigma}{\sigma_{cr}} = 1 - \frac{w}{w_{cr}} \quad (2)$$

as shown in Figure 4, where σ_{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

$$K_c = K_{ac} + K_{bc} = 0 \quad (3)$$

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 :

$$\Delta u_a = (\Delta v + i \Delta u)_a = \frac{c_1 + c_2}{2\sqrt{2\pi}} \frac{K_{ac} r^{1/2+i\epsilon}}{(1+2i\epsilon) \cosh(\pi\epsilon)} \quad (4)$$

where Δv and Δu 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 $k_j = 3 - 4\nu_j$ for plane strain $k_j = (3 - \nu_j)/(1 + \nu_j)$ and μ_j is the shear modulus ($j=1,2$), r is the distance from the crack tip, and ϵ is the bimaterial elastic constant defined as :

$$\epsilon = \frac{1}{2\pi} \ln \left\{ \frac{k_1/\mu_1 + 1/\mu_2}{k_2/\mu_2 + 1/\mu_1} \right\} \quad (5)$$

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 :

$$K_{bc} = (K_1 + i K_2)_b = \int_0^{l_b} K(a) T(a) da \quad (6)$$