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UNIFIED TEST PROCEDURE FOR EVALUATING THE FRACTURE CHARACTERISTICS OF CONCRETE

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Abstract

Three draft RILEM recommendations for evaluating the fracture characteristics of concrete are currently under consideration. The main objective of the study reported is to combine the advantages of all three proposed methods into a single testing procedure by combining the testing arrangements for the G_F test with that of the Two Parameter Method (TPM) and applying them to compact centrally notched cylinder specimens subjected to three point bending. This compact geometry is also suitable for size effect studies. Numerical and experimental work carried out to validate the proposed testing procedure is reported. Three sizes have been tested in a closed loop set up. The analysis has been carried out using the authors' three dimensional finite element code which contains a nonlinear concrete model. Good agreement between the experimental and analytical results was achieved.

1 Introduction

Three draft RILEM recommendations for evaluating the fracture properties of concrete are currently under consideration, namely the G_F test, RILEM (1985), the Two-Parameter model (TPM), RILEM (1990a) and the Size Effect Law (SEL), RILEM (1990b). All three recommended test methods use notched beam test specimens subjected to three-point bending. The three test methods have their respective advantages and disadvantages. The main objective of the work reported here is to combine the advantages of all three methods into a single procedure which is suitable for obtaining fracture properties from cores taken from existing structures as well as new concrete samples.

A major disadvantage of the three draft recommendations is that the tests are carried out on rectangular beam specimens. This is acceptable for laboratory tests but these specimens are not suitable for assessing the fracture properties of concrete in existing structures from which most samples are taken as drilled cores.

The need to develop fracture test methods based on cylindrical test geometries is gradually gaining support. For example both Bittencourt et al. (1994) and Planas et al. (1994) have recently proposed the use of cylindrical test specimen geometries. Bittencourt et al. have reported on the use of short rod test specimens (as used in the ASTM and ISRM standard tests for the fracture toughness of metals, ceramics and rocks) for determining the fracture toughness of concrete. Planas et al. have proposed that the split cylinder test could be developed to determine all necessary fracture parameters. The same need for a practical testing arrangement, which includes laboratory testing of concrete as well as testing cores taken from existing structures, is the main driving force behind the approach adopted in the study reported here.

At the time of writing, an ACI/SEM Task Group on testing standards is considering proposals for a standard test method for concrete fracture. In parallel, a Task Group within the ASTM is also addressing the subject of an ASTM fracture mechanics standard for concrete. Unfortunately, the three competing RILEM recommendations for fracture tests have delayed the introduction of the basic notions of fracture mechanics into codes and standards. The unified testing procedure reported here should be viewed as a practical contribution to bringing together the strengths of the three RILEM recommendations into one testing method.

2 Test requirements

The essential features of a proposed test procedure, which will be acceptable to both the research community and to practicing engineers, is that the test should be simple in concept, technically sound, readily carried out and proven to give reproducible results. These features are satisfied to a large extent by the G_F test. Although the G_F test may be considered initially as providing only a single fracture parameter, G_F, the test can be readily adapted to include the determination of the elastic modulus and the tensile strength as well as the strain softening response of the concrete. The G_F test also has some disadvantages. In much of the experimental work reported on the G_F test, a size effect was apparent in the test results. A detailed study of the potential difficulties associated with the G_F test has been reported by Elices and his co-workers, see Elices et al. (1994), Planas et al. (1994) and Guniea et al. (1994). In these studies the effects of testing arrangement, sample characteristics and experimental procedure on the values of G_F obtained, were all investigated. In particular, they considered the magnitude of energy dissipated at the supports, inside the bulk of the most highly stressed regions and during the final stages of fracture i.e. the tail of the load-deflection curve. When all the sources of energy dissipation were taken into account, they concluded that an almost size independent G_F value was obtained.

One of the main attractions of the Size Effect Method is that it tackles directly the problem of the size scale effects in concrete. To be able to accommodate the known size effects observed in the fracture testing of concrete is one of the basic requirements of any proposed test procedure. The main advantage of the RILEM recommended fracture test, based on the SEM, is that it simply requires the measurement of the maximum loads at failure of geometrically similar notched specimens of different sizes and hence the tests can be carried out in most laboratories. Unfortunately, the need for three test specimen sizes does tend to make the SEM unattractive to the practicing engineer.

The main advantage of the TPM is that it provides a full description of the fracture behaviour of concrete. At the present time, the TPM test method is the only one to provide measurement of the critical stress intensity factor, K_{IC}^{S} , the critical crack tip opening displacement $CTOD_c$ and the elastic modulus from a single test. However, at present, it appears to have found little favour with either practicing engineers or code developers, primarily because the approach is not as amenable to inclusion in finite element concrete models as are G_F based models.

The above brief review of the strengths and weaknesses of the three RILEM fracture recommendations, together with the need to assess concrete structures, leads to the conclusion that cylindrical laboratory specimens should be proposed for codes and standards. Cylindrical laboratory specimens are normally produced with length/diameter ratios of two. Cores usually have relatively long lengths in comparison with their diameter but the total length available for all tests is normally limited, and therefore compact geometries are favourable. Furthermore, compact specimens are suitable for size effect studies. These notions have led to the development of the unified test procedure on notched cylindrical test specimens, subjected to three point bending, as described in this paper.

3 Experimental details

3.1 Test geometry and loading details

The test geometry and loading arrangement used in the study are illustrated in Fig. 1. The cylinders had a nominal length/diameter ratio of 2 and the notch depth was half the diameter. The notched cylinders were loaded in three point bending over a span of approximately 1.8 x diameter. The exact dimensions, span lengths, notch depths and average weight of each specimen type is given in Table 1.

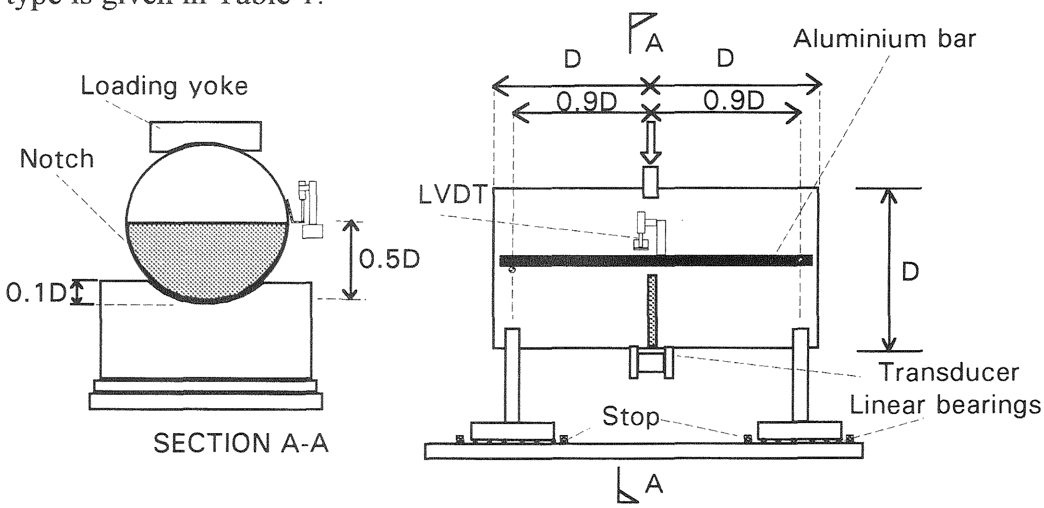

Fig. 1. Arrangement of testing rig

Table 1. Test cylinder details

Specimen Type	Length mm	Diameter mm	Span mm	Notch depth mm	Notch thickness mm	Weight kg
Small (S)	147	76	135	38	3	1.5
Medium (M)	298	150	267	75	3	12.2
Large (L)	590	289	539	140	6	91.4

Special support and loading collars were used to distribute the concentrated loads at the points of contact with the test specimen. Linear bearings were used beneath the support cradles so as to allow free horizontal movement to take place during the test.

A clip gauge was placed across the mouth of the notch and the load was controlled by means of a closed-loop testing arrangement via this clip gauge transducer. An LVDT was used to measure the central deflection of the point application of the load at the centre of the beam relative to the horizontal axis of the test specimen. This central deflection was recorded from an aluminium bar connected to the concrete above the support lines. Thus, both load v crack (or notch) mouth opening displacement (CMOD) and load v central deflection were recorded during the test. The load deflection curves have been used to calculate G_F values for the three tests.

3.2 Mix details and strength

All the test specimens were prepared from the basic mix which has been used extensively in the laboratory at Cardiff. The mix proportions were 1 : 1.8 : 2.8 by weight representing cement, fine aggregate and coarse aggregate ratios. The water/cement ratio was 0.5 by weight. The cement was Ordinary Portland cement, the fine aggregate was sea dredged sand and the coarse aggregate was crushed limestone (10mm maximum size).

The test specimens were kept in their moulds and effectively sealed for 28 days. Thereafter, the specimens were removed from their moulds, notched and allowed to dry in the laboratory until testing at 6 weeks (from casting).

The concrete compressive strength, f_c, and Young's modulus, E, were measured by using three cylinder compression tests. The cylinder size used was 100 x 200 mm. The results were as follows;

f_c (mean) = 50.2 N/mm^2 (Max. variation = ±1 N/mm^2)

E (mean) = 30.6 kN/mm^2 (Max. variation = ±1.5 kN/mm^2)

3.3 Results from fracture tests

Typical load (P) v deflection (δ) and load v crack mouth opening displacement (CMOD) curves for the tests are shown below in Figures 2 and 3. The deflection plotted is that measured at a point a small distance from the centre-line. For the small and medium tests, the central deflection may be taken as 1.05 x the measured value and for the large tests the central deflection is 1.035 x measured value.

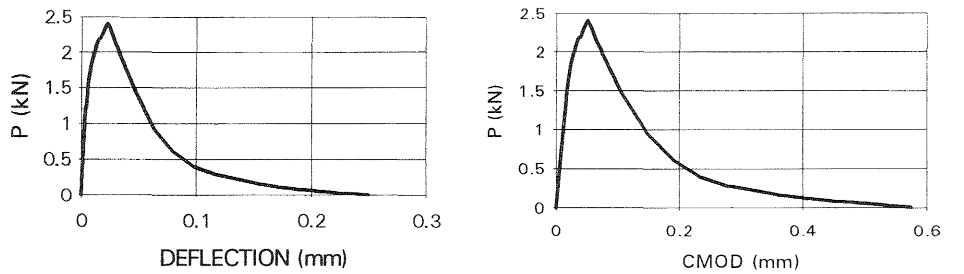

Fig. 2. Results from test S2

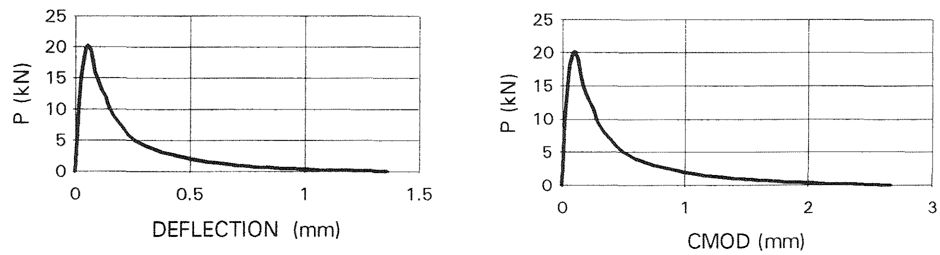

Fig. 3. Results from test L3

G_F was measured from the area under the load-deflection curve divided by the plane area of the final fracture surface. Adjustments were made for self weight and for the fact that deflections were measured slightly off-centre.

The peak loads and G_F values obtained are given below in Table 2.

Table 2. Peak loads and fracture energies from test series

Type	Peak Applied loads in kN				G_F in N/m			
Ref	1	2	3	**Aver.**	1	2	3	**Aver.**
Small	1.77	2.40	2.48	**2.44**	51(1)	70	72	**64.3**
Medium	6.30	6.87	6.28	**6.5**	118	107	85	**103**
Large	21.1	21.0	20.2	**20.8**	95	186(2)	149	**143**

(1) Calculating G_f from the load-cmod curve gave 66 in place of 51.

(2) 186 appears high but does agree with the value calculated from the load/cmod curve.

4 Numerical study

4.1 Theoretical details

The numerical analysis was carried out with a three dimensional finite element program named Cardinal. The program was developed by the authors some seven years ago for the analysis of steel-concrete composite structures, see Jefferson (1990). The program contains a non-linear concrete model which incorporates a combined fracture and plasticity model. Only the fracture part of the model is of relevance to the present study and therefore only this component will be described.

The fracture component of the model is essentially similar to the non-orthogonal crack model developed by De-Borst and Nauta (1985). The only significant difference in the implementation is that a crack flexibility rather than a crack stiffness formulation was adopted. Since the non-orthogonal crack model is well established only a brief description will be given here.

The incremental stress-strain relationship for a material point containing one or more active cracks is governed by the following relationship;

$$\underline{d\sigma} = \underline{D_{ef}}\ \underline{d\varepsilon}$$

in which;

$$\underline{D}_{ef} = \left[\underline{I} + \underline{D}_e \cdot \underline{C}_{fr}\right]^{-1} \cdot \underline{D}_e$$

$\underline{I}$ = Identity matrix

$\underline{D}_e$ = Elastic stress-strain matrix

$$\underline{C}_{fr} = \sum_{i=1}^{n} \underline{N}_{\sigma}^{i\,T} \cdot \underline{c}_{fr} \cdot \underline{N}_{\sigma}^{i} = \text{Summed crack flexibility matrix.}$$

$$\underline{c}_{fr}^{\,i} = \begin{bmatrix} 1/E_T & 0 & 0 \\ 0 & 1/(\beta\cdot G) & 0 \\ 0 & 0 & 1/(\beta\cdot G) \end{bmatrix}$$

$\underline{de}_{fr}^{\,i} = \underline{c}_{fr}^{\,i} \cdot \underline{ds}^{i}$ Local incremental stress-fracture strain relationship

n = Number of cracks at a point

$\underline{de}_{fr}^{\,i}$ = Increment of local fracture strain for crack i

$\underline{ds}^{i}$ = Increment of stress on crack plane i

G = Elastic shear modulus

β = Shear retention factor

E_T = Slope of the stress-fracture strain curve

$\underline{N}_{\sigma}$ = Stress transformation matrix

The local stress-fracture strain curve is controlled by either a bilinear or exponential softening function. The bilinear function due to Petersson (1981) was used for the present analysis (see Fig. 4).

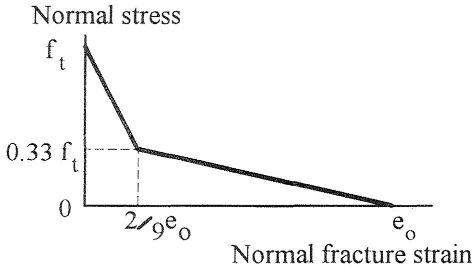

Fig. 4 Bilinear softening curve

The concrete model incorporates both a distributed and localised fracture model. The localised model, used for the present study, follows the crack band model of Bazant and Ho (1983).

Multiple cracks are permitted to form at one integration point but a tolerance is applied which limits the maximum angle between cracks. For the present study this was set to 0.6 in terms of the dot product of the unit normals to the crack planes.

4.2 Analysis of test specimens

The concrete, cradles and yoke were all modelled with standard 20-noded isoparameteric brick elements. A 15 point integration rule was used for the elements. The interfaces between the steel supports (including the loading yoke) and the cylinder were simulated with 16-noded contact elements which were given high normal stiffnesses but relatively low tangential stiffnesses.

One quarter of the cylinder was analysed using 121 brick elements. The finite element mesh used for the analysis is shown below in Fig. 5 and a plot showing the cracks at peak load is shown in Fig. 6.

The results from the analysis of test M2 are presented in Fig. 7 in the form of load v deflection and load v CMOD plots.

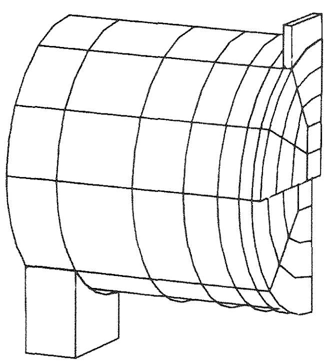
Fig. 5. Plot of f.e. mesh.

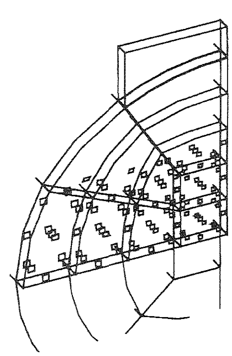
Fig. 6. Crack plot at peak load

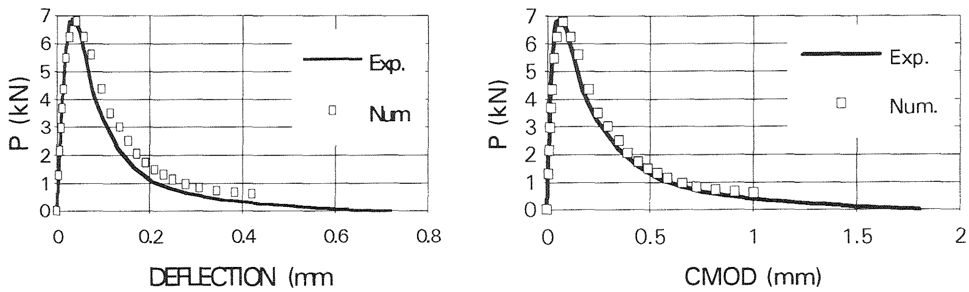

Fig. 7 Experimental and numerical load v displacement curves for test M2

5. Discussion and conclusions

The results from the test series on compact test cylinders are promising and suggest that the geometry and testing arrangement is suitable for finding the fracture properties of concrete.

The smallest cylinder was prone to the greatest variation in fracture properties since the un-notched height was less than four times the maximum coarse aggregate size. This suggests that a shallower notch, of say 0.25D, may be preferable.

The finite element code is clearly capable of reproducing the results of the tests although further work is required to understand the inter-relation between the parameters used in the analysis and those measured in experiments. This work is continuing.

The size effect in the G_F values is of particular concern since it raises questions over the assumptions of the blunt and cohesive crack models which have found such favour with numerical analysts. While the explanations of Planas, Elices and Guniea help in the understanding of this problem the issue is, as yet, far from resolved.

Finally, since it behoves researchers to periodically question the objectives of their work, it is worth mentioning that, as part of some on-going investigations into the response of mass concrete walls to earthquake loading, the authors', together with the consultants with which they are working, have found that the secondary response spectra at the top of the dock walls are senstive to the fracture properties used in analyses. This is leading to a forth-coming testing programme on cores taken from the dock walls and provides one of the primary motivations behind the work reported here.

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