INVESTIGATION OF CORROSION INDUCED CONCRETE CRACKING

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Abstract

Concrete is widely used in civil structures. Concrete cracking associated with quasi-brittle feature of concrete is commonly found in concrete structure. Prediction of concrete crack growth is important to maintenance and rehabilitation of concrete structures. Several concrete crack models have been developed in which cohesive crack model (CCM) has been widely used by incorporating with finite element (FE) analysis. One advantage of adopting CCM in modeling concrete crack is that it becomes possible to obtain the crack width information. This paper attempts to predict the concrete crack width by means of a numerical method. A state of the art review on fracture mechanics and its application to concrete structures is presented. In this review, fictitious crack model, two-parameter model and size effect model are briefly discussed as well as numerical modelling techniques for smeared and discrete cracks. A numerical model is then established to predict the fracture behaviour including globe response of specimen, size of fracture process zone, and finally the crack width. The globe response (stress-displacement curve) obtained from the FE analysis will be compared to the experimental data from literature for validation. This model can be possibly used to predict the crack width in other cases, such as corrosion-induced cracks.

Keywords: concrete, crack, numerical model, fracture mechanics.

1. Introduction

Concrete is a quasi-brittle material which means it continues to sustain the loading after it reaches to the peak load. This is known as softening behavior. Such behavior is a coupled mechanism which consists of separation of crack and unloading of the substrates. In order to model concrete cracking problems, especially in sake of predicting crack width, cohesive crack model (CCM) has been widely used by incorporating with FE analysis. CCM postulates that there is a nonlinear fracture process zone (FPZ) ahead of crack tip where strain softening occurs. This leads to removal of the singularity stresses at the crack tip when linear elastic fracture mechanics is considered. The FPZ is governed by the stress-crack opening displacement curves which, in various descending shapes, are regarded as the material properties and are really vital to the structural responses. CCM is incorporated into the finite element analysis via interface elements with normal and shear stresses and relative displacements across the interface as constitutive variables, but with no thickness. The cohesive interface is governed by the properties of FPZ while the rest of the structures keep linear elastic which suggests no crack occurs beyond the interface and experiences unloading once the peak load is reached. This interface must be stiff enough to sustain the loading and predict the same response as the uncracked concrete prior to crack initiation.

This paper attempts to predict the concrete crack width by means of numerical method. A state of the art review on fracture mechanics and its application to concrete structures is present. In this review, fictitious crack model, two-parameter model and size effect model are briefly discussed as well as numerical modelling techniques for smeared and discrete cracks. A numerical model is then established to predict the fracture behaviour including globe response of specimen, size of FPZ, and finally the crack width. The globe response (stress-displacement curve) obtained from the FE analysis will be compared to the experimental data from literature for validation. This model can be possibly used to predict the crack width in other cases, such as corrosion-induced cracks.

2. Fracture mechanics application to concrete cracking: The state-of-the-art

Based upon elastic mechanics, when a crack is present it alters the stress distribution and leads to the stress at the crack tip to approach infinity. The strength-based elastic mechanics, therefore, can not be applied to cracking problems. The first attempt in means of fracture mechanics was the energy criterion of failure proposed by Griffith (1921). This was then developed to the basics of Linear Elastic Fracture Mechanics (LEFM) when the J-integral (Rice 1968a, b) had been found to be equal to the energy release rate and Irwin (1957) reformulated LEFM in terms of the stress states around the crack tip. LEFM can be used for any materials as long as an assumption is met, that is, all the material is elastic except in the vanishing small region ahead of the crack tip. It can be allowed that some kind of inelasticity occurs around the crack tip as the stress there is inevitably high, the size of this region, however, must be very small regarding its perturbation to the linear performance of the whole structure to apply LEFM. Griffith's energy criterion and stress intensity factor are identical in predicting the fracture behaviour of elastic body. Bažant and Planas (1998) compared those two approaches and postulated that, firstly the stress intensity factor was additive while Griffith's energy release rate was not; secondly the stress intensity factor approach was limited to linear elasticity, while the concept of energy release rate could be extended to nonlinear materials.

Concrete is a quasi-brittle material which means the tensile strength deteriorates gradually after the peak loads in a number of softening forms. The reason is there is an inelastic zone developing ahead of the crack tip which is also referred to fracture process zone (FPZ). The influence of FPZ on fracture behavior of concrete has been regarded as huge and that leads to inability of LEFM to solve concrete cracking problems and nonlinear fracture mechanics (NLFM) is applied. That is bacause LEFM accounts only the energy required to create two cracked faces – for elastic materials, that is all needed as the stress will suddenly drop to zero and catastrophically failure occurs; while for quasi-brittle materials, additional energy should be taken into account to separate the two cracked faces as the stress between the two faces will be decreasing continuously. This leads to the fundamental NLFM theory of energy balance of Mode I quasi-brittle crack which can be expressed as (Shah *et al.* 1995)

$$G_q = G_{\mathrm{I}c} + G_\sigma \qquad (1)$$

Where G_q represents the energy release rate for Mode I quasi-brittle crack, $G_{\mathrm{I}c}$ is the energy rate consumed in creating two cracked faces which is equivalent to the material surface energy and can be calculated based on LEFM, and G_σ is the energy rate to overcome the cohesive pressure between the two cracked faces. A few of researchers have proposed models based on Eq. 1 to describe fracture of concrete (Cook *et al.* 1987; Cox and Marshall 1994; Jenq and shah 1985; Kobayashi *et al*. 1991). However, either mechanism can be approximately used individually to describe the fracture of quasi-brittle materials, that are, namely, Griffith-Irwin mechanism by assuming $G_\sigma = 0$ and Dugdale-Barenblatt mechanism by assuming $G_{\mathrm{I}c} = 0$. According to these two energy dissipating mechanisms, fictitious crack model, two-parameter model and Size effect model have been developed.

The ficititious crack approach postulates that the energy required to create the new surfaces is vanishingly small compared to that required to separate them and due to this assumption, the energy dissipation in fictitious approach is completely characterized by cohesive stress-separation relationship in the fracture process zone. Based on such an approach, two models are widely recognised and they are cohesive crack model (CCM) proposed by Hillerborg *et al*. (1976) and crack band model orientated by Bažant and Oh (1983). In the cohesive crack model, the three governing parameters are material tensile strength f_t, fracture toughness G_F and the shape of the $\sigma(w)$ curve. Most literature simplified the pre-cracking concrete mechanical property to isotropic linear elastic. It is supposed in CCM, that a crack will be initiated whenever and wherever the tensile stress exceeds to the tensile strength. After crack initiation, stable crack propagation requires an energy balance which is the strain energy release rate is equal to the fracture resistance, that is, energy rate for overcoming the cohesive forces. Bažant and Oh (1983) modeled the fracture process by a band of uniformly distributed microcracks with a minimum thickness of h_c which exhibits linear strain softening. The expression of CCM could be identical to that of crack band model if $h_c\varepsilon^f = w$ is identified, where ε^f is the fracture strain. A two-parameter fracture model represents the fracture process zone by an equivalent elastic crack which is traction free. Apart from the LEFM which uses one fracture criterion (e.g. critical stress intensity factor), this model employs second criterion to assess the propagation of crack in the form of crack tip opening displacement ($CTOD_c^e$). Size effect model considers a series of geometrically similar structures and describes their nominal strengths in function of G_f and c_f which are the critical energy release rate and the critical crack extension for an infinitely large structure respectively. The cohesive crack model, two-parameter crack model and size effect model were all designed to account for nonlinear behaivor of crack propagation in quasi-brittle materials such as concrete. They apply different energy dissipation mechanisms, and thus different failure criterions. However, they should be able to predict the same response for infinitely large structures because the FPZ is negligibly small which leads to application of LEFM. For Laboratory size structures, Hanson and Ingraffea (2003) point out that these three models do not always predict the same behaivor except in a range of FPZ properties for which they do agree.

To apply finite element method in modelling cracking of either plain or reinforced concrete, it has been proven to be difficult (ACI Committee 446.3R 1997) for either discrete crack approach or smeared crack approach. Discrete crack approach treats the crack as a “real” crack by introducing discontinuity in the geometry of the structure. The crack path should normally be assumed a priori and a mesh is arranged so that the path coincides with boundaries between elements (Shah *et al.* 1995). When the crack path is not known in advance, however, remeshing is needed. The smeared crack approach treats the crack as not a “real” one but a continuous medium with altered mechanical properties. There is not displacement discontinuity in the finite elements by this approach, instead, the mechanical properties of the cracked elements are modified by some deterioration law. A lot of discussions have been made on the advantages and disadvantages on these two crack models (ACI Committee 446.1R 1991; Bažant and Planas 1998; Shah *et al.* 1995). In terms of physical phenomenon, distributed damage or densely distributed parallel cracks can be well represented by smeared crack model, while if only a single crack is present or of interest, discrete crack model is more appropriate. Ideally, however, smeared crack model should be capable of representing the propagation of a single crack with reasonable accuracy (ACI Committee 446.3R 1997). The obvious advantages of smeared cracks could be the computational convenience and more appropriate representations for some physical observations, while the typical numerical difficulties are regarded as the strain localization instabilities and spurious mesh sensitivity of finite element calculations (Bažant and Planas 1998).

To obtain crack width information, only discrete crack model is applicable. Discrete crack model treats a crack as a geometry entity. In elastic material, LEFM implies there is stress singularity ahead of the crack tip, while in quasi-brittle material, the stress singularity is removed by introduce of FPZ which experiences strain softening once the peak load is reached. LEFM can be applied to concrete cracking when the FPZ is negligibly small compared with the size of the structure and the key is to calculate the stress intensity factor either with or without the singularity elements. The cohesive crack model is normally incorporated into finite element codes by employing interface elements. Zero-thickness interface elements are usually used to represent the cracks and nonlinear solution is required as the stiffness of the interface element is a nonlinear function of the crack opening displacement. To choose the stiffness of the interface elements is normally a problem. This might be due to the fact that there is no such an interface physically thus no existing values for its properties. Some researchers have suggested a few values after they successfully carried out the modelling, e.g. Brown *et al.* (1993) advised that the axial stiffness of the interface elements shall be about 50 times of that of adjacent concrete elements.

3. Interface cohesive crack model

CCM assumes the FPZ is long and narrow, and is characterized by a stress-crack opening displacement curve, shown in Fig. 1. The interface CCM is incorporated into finite element analysis with the crack path is known in advance. The interface represents the crack which is controlled by the traction-separation law shown in the following equation:

$$\sigma_c = f_{T-S}(\delta) \qquad (2)$$

The traction-separation law is intended for bonded interfaces where the interface thickness is negligibly small (Abaqus manual 6.8). This provides the best estimation for concrete because there is no actually real interface in it. The cohesive element contains the features of the crack initiation criterion and the crack propagation criterion which is the also the failure condition for the elements. As the damage is assumed to occur in the interface, the modelling of the bulk concrete should only include the unloading behavior, for which the linear elasticity is applied to the bulk concrete. The stress-strain relationship is shown in Eq. 3

$$\sigma_b = f_{elastic}(\varepsilon) \qquad (3)$$

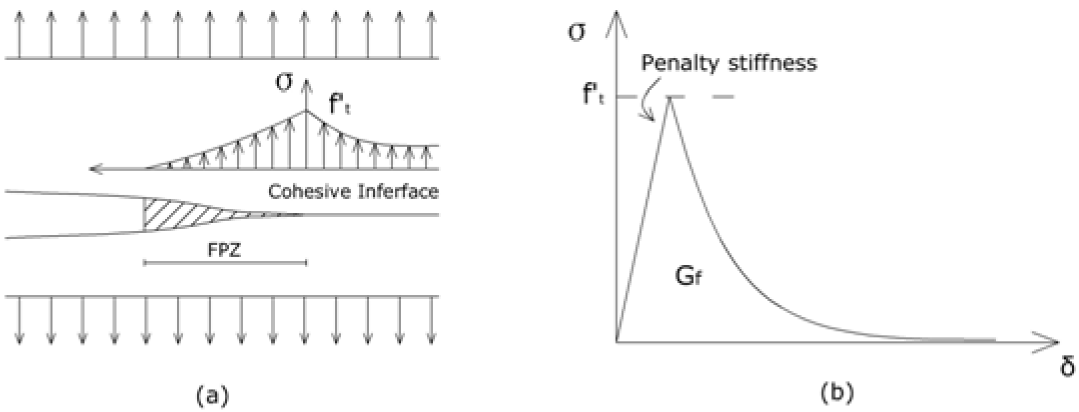

Figure 1: (a) Schematic of mechanism of cohesive interface (b) exponential softening traction-separation curve for cohesive material

The initial response of the interface CCM is assumed to be linear in terms of a penalty stiffness (K_p) which implies no energy loss before peak loads. This penalty stiffness must be stiff enough prior to initiation of crack to hold the two surfaces of the bulk concrete together, leading to same performance as no interface presents. However, K_p can not be too high as it will introduce convergence problems due to ill-conditioning of the element operator (Abaqus manual; ACI 446 1997). The softening is a characteristic of the material that must be determined from experiments. There are a lot of softening curves such as linear, bilinear and non-linear softening, and some researchers concluded that the detailed shape of the softening curve is less important than the values of fracture energy (Elices *et al.* 2002; Needleman 1990). In this study, exponential softening is adopted. FPZ starts at the point with zero tensile stress to the point with zero separation. In this region, the material is damaged but not failed as they can still transfer the cohesive stresses.

4. Finite element simulation

Finite element simulation is performed to analyse the Mode I fracture behaviour of plain concrete using a commercial finite element code (ABAQUS version 6.8). A rectangular concrete bar in direct tension with notches in the middle of two sides is executed. The dimensions are selected as 76×305mm and plain stress condition is assumed in the thickness direction. Fig. 2 shows a half of the specimen because of the symmetry of either geometry or mechanism. The cohesive interface is embedded in the middle connecting the two notches. The notches are circularly shaped which provide stress concentration around the end of cohesive interface. Rectangular shape of notch has been preliminarily tested and the stress concentration has been found to be present at the two corners which is not acceptable for the pre-defined crack path.

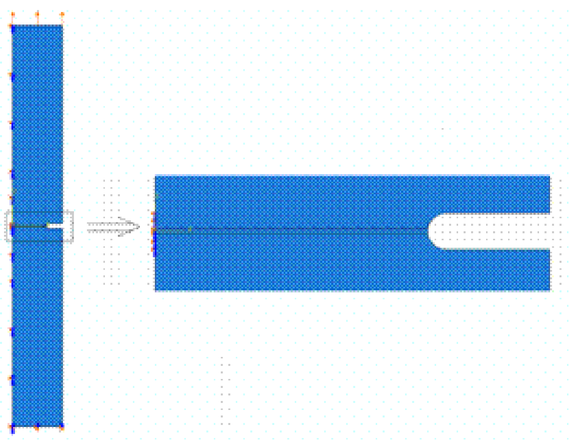

Figure 2: Geometry and boundary conditions of the specimen

4.1 Material properties

The material properties have been taken from the experimental results from Gopalaratnam and Shah (1985). There are two types of material: cohesive interface and bulk concrete. As the bulk concrete is considered to exhibit linear elasticity during the whole testing, only Young's modulus and Poisson's ration are available which are 33.5GPa and 0.18 respectively. The material parameters for the cohesive interface include the penalty stiffness , tensile strength , and the fracture energy for the exponential softening. They are based on the stress-crack width curve and are taken as 1675GPa, 3.62MPa and 0.131N/mm respectively.

4.2 Failure criterions for cohesive elements

The pipeline was run at the internal gauge pressure of 100 kPa and the room temperature between 25-35^{o}C. The preliminary result in Figure 7 showed a tendency of increasing corrosion rate. However, the rate of increase is small over the first month. If corrosion rate of 100 micrometer per year (or 0.1 mm/year) is used in this one-month period, a typical steel pipe of 4 mm wall thickness would fail in 40 years due to corrosion. However, the corrosion rate suddenly is very high (up to 3.5 mm per year) after 4 months, which requires further

4.2.1 Crack initiation

Crack initiation corresponds to the start of degradation of the response of a material point. Damage is assumed in this study to initiate when the maximum nominal tensile stress reaches to the nominal tensile strength.

4.2.2 Crack propagation – cohesive element failure

After cracking of one cohesive element is initiated, the stress of this element softens in an exponential manner. The failure of the element is governed by the softening curve actually which determines the fracture energy. When the energy release rate calculated reaches to the fracture energy input, the cohesive element is completely failed and the crack propagates. To evaluate the damage of the cohesive element, stiffness reduction variable D is nominated. D is defined in terms of energy fraction shown in the following equation:

$$D = G_r / (G_f - G_e) \qquad (4)$$

Where G_r is the energy rate to overcome the cohesive stress σ , G_e is the energy rate absorbed during elastic ascending part. The relationships of these energy parameters are illustrated in Fig. 3.

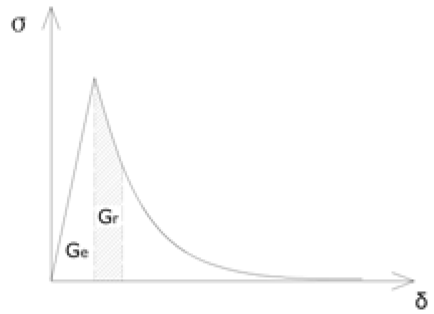

Figure 3: Illustration of various energy release rates

Then the residual modulus of elasticity of the cohesive element becomes:

$$E_r = (1 - D)E \qquad (5)$$

Where E is the original Young's modulus.

4.2.3 Viscous Regularization

It is usually a problem for materials experiencing softening behavior to get converged in the implicit analysis. This is because when concrete is cracked, sudden energy dissipation of energy will make the analysis dynamical problem while quasi-static analysis is expected in implicit analysis. Viscosity is used to regularize the traction-separation law by modifying the stiffness reduction variable D. Using viscous regularization with a small value of viscosity helps improve the rate of convergence of the model in the softening regime, without compromising results (ABAQUS manual 6.8).

4.3 Results

This specimen is simulated using two kinds of elements: (1) 4-node plane stress element for bulk concrete, and (2) a single layer of 2 dimensional cohesive elements through the thickness of cohesive interface. Fine mesh is adopted, shown in Fig. 4, with 0.5mm for the cohesive element size and 0.5-4mm for that of bulk concrete. The stress-strain/elongation curve of the cohesive element is produced in Fig. 5. It fits the input of the material properties for every point in the cohesive interface quite well. Figure 6 shows the stress-displacement relation of the specimen. It should be noted that, however, the displacement recorded is partial displacement across the cohesive interface with 41.5mm beyond each side. Figure 7 plots the tensile stress contour around the cohesive interface at loading level (1) 57 per cent of failure load, (2) 85 per cent of failure load, (3) 94 per cent of failure load, and (4) 99.7 per cent of failure load. It has demonstrated the stress concentrations as well as the damage propagations through the middle cohesive interface. Fig. 8 gives the sizes of developing and steady state fracture process zone. As FPZ is defined as the area surrounding a crack tip within which inelastic material behavior occurs, it should starts from the point with zero-separation to the point with zero traction, which is called steady state FPZ. The size of the steady state FPZ is a material constant and can be related to aggregate size, concrete strength, etc (ACI 4463r 97).

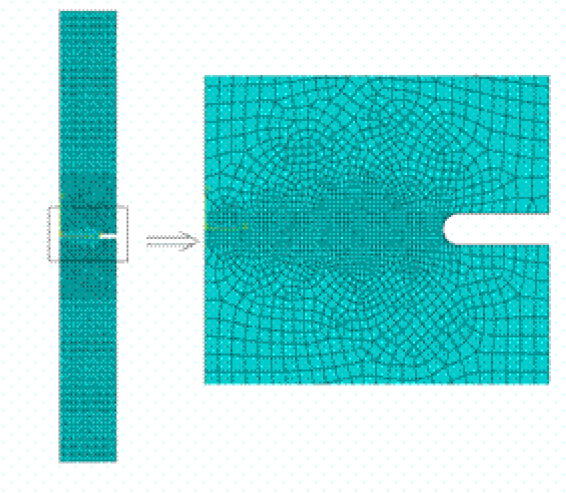

Figure 4: Element meshes of the whole specimen and near the cohesive interface

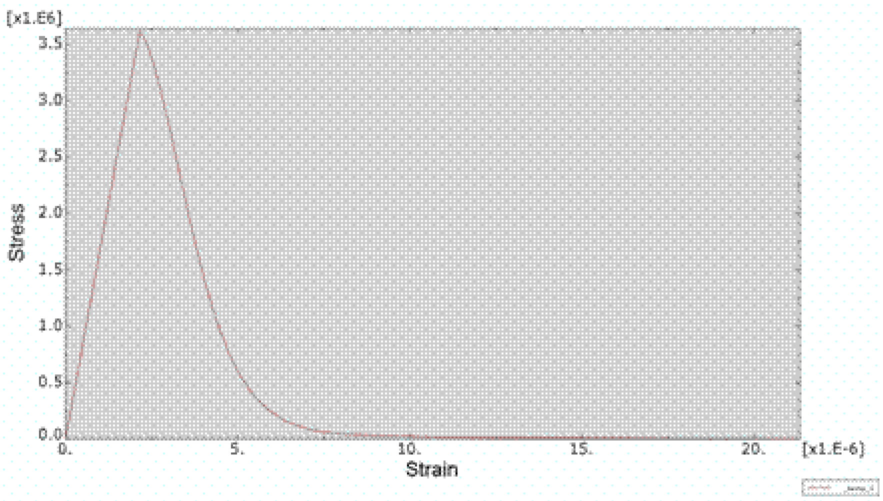

Figure 5: Stress-strain/elongation relation of the cohesive element

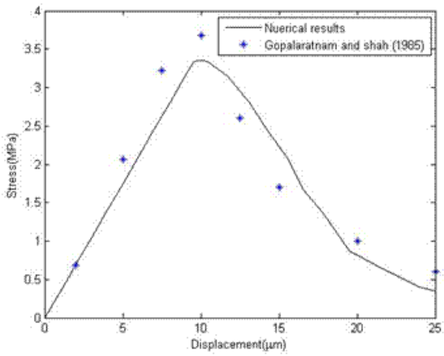

Figure 6: Stress-displacement curve of the concrete specimen and comparison with experimental results

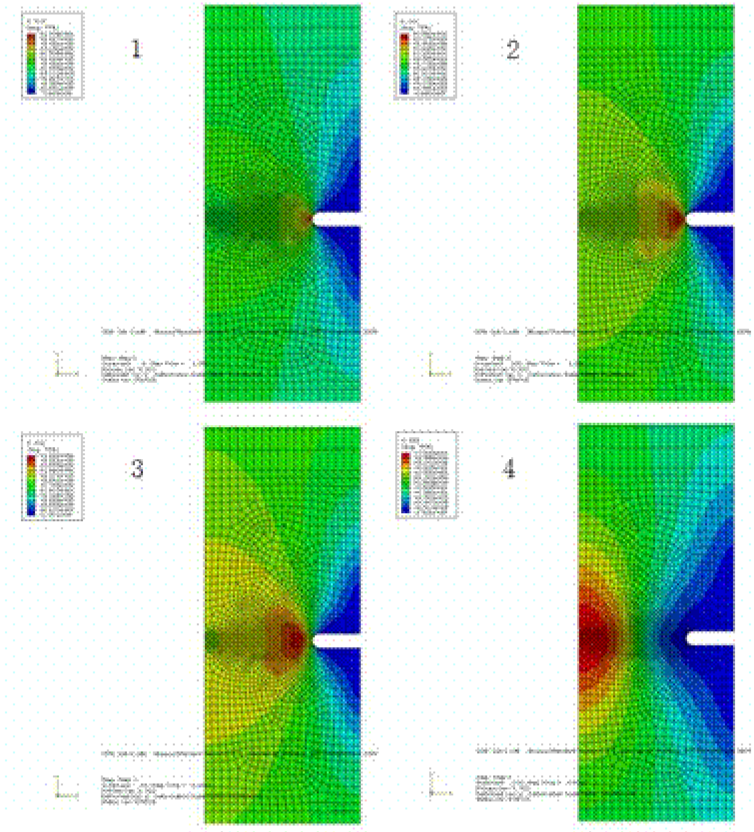

Figure 7: Tensile stress contour (S22) around the cohesive interface at different loading level

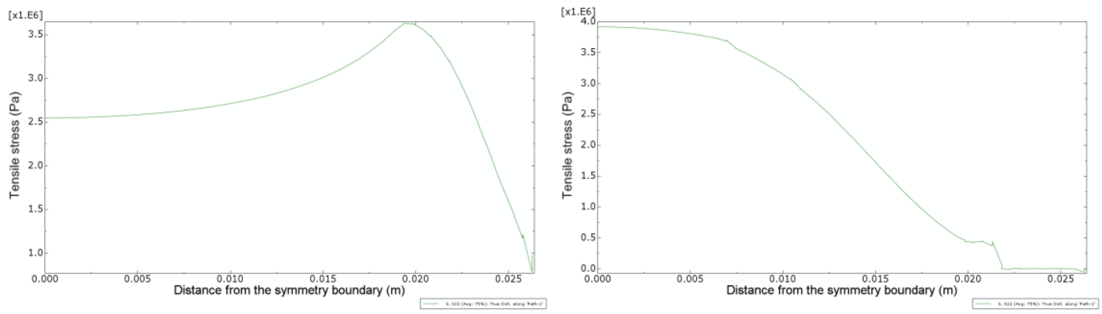

Figure 8: Sizes of developing and steady state fracture process zone

5. Validation of finite element modelling with experimental data

The cohesive interface failure process and the total response of the concrete specimen in direct tension have been simulated using ABAQUS. As an example, a typical comparison of the FE model with the experimental results from Gopalaratnam and Shah (1985) is shown in Fig. 6, demonstrating the relatively close agreement of load v.s. displacement behavior between the FE predictions and experimental data.

6. Concrete crack width

One advantage of adopting CCM in modeling concrete crack is that it becomes possible to obtain the crack width information. Fig. 9 presents the crack width with time in the position of right end of cohesive interface at the notch. The time has been calibrated into the real time during the experiments; otherwise the time component makes no sense. The crack width is extracted based on the separation of the cohesive from Fig. 9, the crack width experiences an abrupt increase before steadily increasing again. This abrupt increase corresponds to the softening after 5 μm elongation in Fig. 5 (tail of the curve), which in most cases is disregarded and thus crack width will suddenly increase from zero to a certain value, which could be 20 μm in this study.

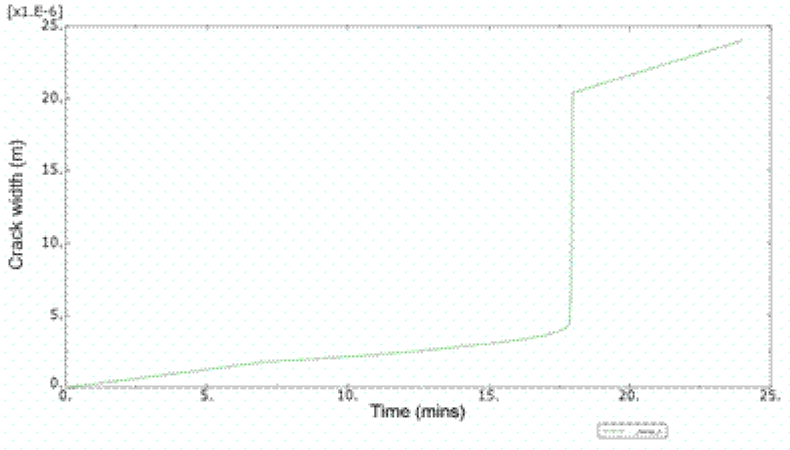

Figure 9: Crack width as a function of time

7. Conclusion

A finite element simulation based on cohesive crack model for concrete cracking was proposed to predict the fracture behavior of specimen in direct tension. An exponential softening model for the cohesive interface was derived from the experiments and implemented into a finite element-based cohesive zone model. Interface elements have been used to represent the pre-defined crack path. Fracture behavior of cohesive element has been produced and compared to the input for the material. The global behavior of the concrete specimen has been captured by the numerical simulation and verified with experimental results from literature. The crack propagation was monitored and the size of FPZ is measured which depends on the material properties. The crack width was finally obtained as a function of time which had been calibrated with the real time of experiments.

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