Relationship between fracture toughness and tensile strength for geomaterials

Relation entre la dureté d'une fracture et la résistance à la traction des matériaux géotechniques

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Synopsis: It is argued that the development of significant regions of tensile stress can often lead to failure in foundation engineering, especially for foundations in weak and weathered rock. As a result, accurate predictions of performance can be obtained only if this tensile failure is accounted for. It is also argued that the methods currently used for predicting tensile failure are unsuitable and that a more realistic model of tensile failure, i.e. as predicted by fracture mechanics theory, should be adopted.

The application of fracture mechanics theory requires the determination of the fracture toughness of the rock or soil. However, standard sampling techniques are often unable to supply suitable samples for fracture toughness testing, and hence an alternative method of estimating fracture toughness may be required. This paper investigates the relationship between tensile strength and Mode I fracture toughness for a wide variety of rocks and soils. It is shown that a reasonably strong correlation between these two parameters exists across the spectrum of geotechnical materials analysed, and as such provides a reasonably accurate method of estimating fracture toughness from tensile strength.

1 INTRODUCTION

Traditionally, foundation engineering has been concerned with the analysis and design of foundations in soils. However, since an increasing number of large structures are being founded in weak and weathered rock, a greater emphasis is being placed on design in these stronger but more brittle materials. Unfortunately, only a relatively small quantity of research has been carried out in this field. As a result, the methods currently used for design tend to be very conservatively based on past experience or on the extrapolation of traditional soil mechanics approaches.

In the past, very little attention has been paid to tensile failure in foundation engineering. This is somewhat justified due to the relatively few situations in which tension failure occurs in foundations in soil. However, as described below, due to the more brittle and dilatant nature of weak rock, some form of tension failure is likely to occur well before failure in compression. As a result, it is essential that foundation design should make some allowance for tensile failure.

As will also be explained below, the methods currently used for predicting tension failure are unsuitable, and a more realistic model of failure is required. Such a model is obtained by using fracture mechanics theory.

To apply fracture mechanics theory, a parameter which quantifies a material's resistance to fracture is required. This parameter is called the fracture toughness and may be determined for elastic materials by relatively simple laboratory tests. Unfortunately, due to complications arising from inelastic behaviour, the fracture toughness of soils and rocks is size dependent. This may mean that samples larger than those obtained from standard site investigations may be required to obtain an accurate estimate of the fracture toughness of the material. In such instances it is necessary to use an alternative method of estimating fracture toughness.

This paper presents some of the results of an investigation into the fracture characteristics of a weak rock and demonstrates that a reasonably strong correlation between tensile strength and Mode I fracture toughness exists for this rock. By incorporating test data from other rock and soil types, it is shown that a similar correlation exists for all of the materials investigated.

2 TENSION FAILURE IN FOUNDATION ENGINEERING

Tension failure can occur via two basic mechanisms:-

(i) as a result of the application of forces that directly impart tensile stresses into the rock or soil mass, and

(ii) as a result of the dilation of shear zones that indirectly impart tensile stresses into other parts of the rock or soil mass.

An example of the first mechanism involves tensile failure during pressuremeter testing. When the pressuremeter is introduced into the soil or rock mass, a compressive stress field that is equal, in theory, to the insitu horizontal stress exists around the probe. Theory also predicts that an increase in the pressure in the pressuremeter probe results in an increase in the stress in the radial direction and an equal and opposite decrease in stress in the circumferential direction;

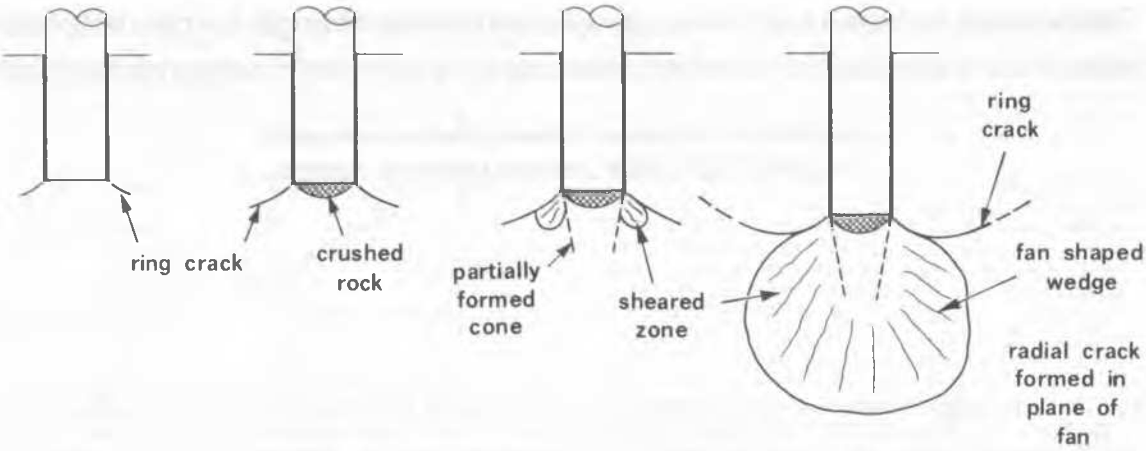

Figure 1 Failure processes around pile tip

i.e.the stress in the circumferential direction becomes increasingly tensile. Haberfield and Johnston (1986) argued that for the majority of pressuremeter tests conducted in weak rock, and for pressuremeter tests conducted at shallow depths in soil, the circumferential stress adjacent to the probe can exceed the tensile strength of the rock or soil. As a result radial cracks may initiate at the cavity wall and propagate into the soil or rock mass.

Since the test is conducted below the ground surface, direct confirmation of this failure mechanism is difficult to obtain. However, Briaud (1986) reported the formation of radial cracks during shallow pressuremeter tests in clay. Also, indirect confirmation has been obtained by both laboratory (Haberfield and Johnston, 1987) and numerical (Haberfield and Johnston, 1988) investigations.

An example of the second mechanism involves tensile failure of shallow and deep footings in weak rock. In this case, the applied load initially causes a region of high tensile stresses to form at the footing or pile edge as shown in Fig. 1 (Johnston and Choi, 1985). As a result the ring crack illustrated in Fig. 1 is formed. As the applied load is increased, plasticity theory predicts that a bulb of sheared material will be formed below the footing or pile tip. Since weak rock dilates considerably (Johnston 1988), the volume of this sheared zone must increase, which in turn imparts tensile stresses into the elastic portion of the rock mass surrounding the bulb. Once these tensile stresses exceed the tensile strength of the rock, a radial crack forms. Experimental confirmation of this failure mechanism has been obtained by both field pile tests (Williams, 1980) and laboratory tests (Choi, 1984; Johnston and Choi, 1985).

As indicated by these two examples, the effects of tensile failure must be included into any analysis in order to obtain a reasonable prediction of performance. Only then can an economic safe design be forthcoming.

3 PREDICTION OF TENSILE FAILURE

Due to the complexity of geometry, material behaviour and boundary conditions associated with foundation engineering problems, the prediction of performance generally requires some form of numerical solution. The finite element method is one numerical method which has been used extensively in this field.
The solutions to problems involving tensile failure are usually solved by adopting a limited tension formulation similar to the one suggested by Zienkiewicz, Valliappan and King (1968). Unfortunately this type of analysis is really only suitable for predicting the onset of catastrophic tensile failure, such as during tensile strength tests. It is incapable of making accurate predictions of performance in problems which involve gradual crack propagation (Haberfield, 1987). This is because the above formulation fails to model crack propagation realistically i.e. it continues to model the material as a continuum even after tensile failure has occurred. To obtain an accurate prediction of performance after initial tensile failure, crack propagation must be modelled directly. This may be achieved by using fracture mechanics theory, and, in particular, linear elastic fracture mechanics or LEFM.

4 THE APPLICATION OF LEFM TO GEOMATERIALS

Although fracture mechanics theory has been used extensively in the area of hard rock mechanics, it is only recently that it has started to gain acceptance in the soft rock and soil mechanics fields. Hence, to ensure that the reader has at least a basic knowledge of fracture mechanics theory, a short description of the application of LEFM to geomaterials is included below.

LEFM basically involves the determination of the magnitude of the singularity at a crack tip in an elastic body. This value is called the stress intensity factor and is usually denoted by K. The stress intensity factor is a function of the crack and body geometry, and the stresses in the vicinity of the crack tip. As explained below, by conducting a test on an initially cracked specimen, the stress intensity factor at failure, K_C, can be determined. This value is called the critical stress intensity factor or fracture toughness of that material, and is a measure of the material's resistance to fracture. This property may be used to determine crack propagation by comparing the stress intensity factor at the crack tip (as determined by LEFM) with the fracture toughness of the material. If the stress intensity factor is greater than the fracture toughness, then the crack will propagate in a direction which is perpendicular to the minor principal (or maximum tensile) stress field, until failure or until the stress intensity factor at the crack tip becomes less than the fracture toughness.

In general, cracks may propagate in any one or any combination of three independent ways as shown in Fig. 2. These are known as Mode I or opening mode, Mode II or sliding mode and Mode III or tearing mode. For a large number of situations in which tensile stresses occur, only Mode I cracking is applicable, and hence a parameter known as the Mode I fracture toughness is required. This parameter is usually denoted by K_{IC}. For the remainder of this paper only Mode I cracking is considered.

The problems associated with applying LEFM to geomaterials arise from the inability of the material to remain elastic throughout loading. The very high stresses which occur at the crack tip cause the material to fail locally, resulting in a region of failed material at the crack tip called the crack tip yield zone. In hard rock technology, this yield zone is generally referred to as the micro-crack zone.

The influence of a crack tip yield zone on the performance of a cracked specimen (as predicted by LEFM), depends upon the size and shape of the yield zone with respect to the overall dimensions of the specimen; i.e. if the crack tip yield zone is small compared to the size of the specimen, then the effect of the yield zone is also small and the material can be considered to be behaving elastically. However, as the crack tip yield zone increases in size relative to the size of the specimen, more of the material will be behaving inelastically and as a result the yield zone will have a larger effect. One direct implication of this size effect is that in fracture toughness determinations, test specimens must be of a significant size to ensure mainly elastic behaviour and, as a result, a reasonably constant value of fracture toughness. This constant value of fracture toughness is called the plane strain fracture toughness and it is this value which is applicable to LEFM.

5 TEST MATERIAL

Researchers at Monash University have been using experimental, analytical and numerical techniques to investigate the behaviour of soft rock, and in particular a locally occurring soft rock called Melbourne mudstone, for several years. The early experimental studies carried out on natural rock yielded results which displayed a significant amount of scatter. This scatter complicated the interpretation of the experimental results and consequently, it was often very difficult to obtain sufficiently accurate or reliable data with which to validate the analytical or numerical models. To overcome this problem a synthetic rock was developed. This material, known as Johnstone, is homogeneous and isotropic, and may be reliably and simply reproduced with a range of properties. These properties are similar to those of naturally occurring soft rocks, and can be controlled and accurately determined. The saturated water content of the Johnstone is an excellent indicator of its material properties. Full details of the manufacturing process and the properties of Johnstone may be found in Johnston and Choi (1986).

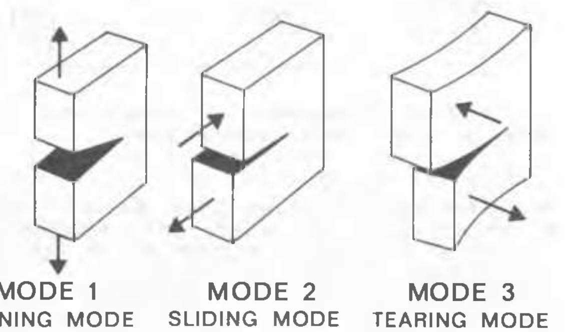

Figure 2 Modes of Fracture

6 FRACTURE TOUGHNESS DETERMINATION OF JOHNSTONE

As part of a major investigation into the development and propagation of stress and strain induced cracking in soft rocks (Haberfield, 1987), methods of establishing the plane strain fracture toughness of soft rock were examined. After careful consideration of the various factors which might influence the results obtained, one of the principal techniques finally adopted involved the use of three point loading of single edge cracked beams (SECB) as shown in Fig. 3. The beams had overall dimensions of depth, W, thickness, B, and length of a little more than span, S, and were cut and machined from larger manufactured blocks of Johnstone of constant saturated water content. At the mid-point of each beam, a notch of width δ, and length a, was cut carefully into the underside of the beam by means of a specially set-up bandsaw.

The test is based on one of a number of standard test methods for fracture toughness testing of metals which are described in ASTM Standard E399 (1983). This particular test basically involves loading a simply supported SECB specimen at mid-span at a specified rate, and taking continuous measurements of the notch width against load. The notch width was measured by a clip gauge which was mounted over the notch mouth. From the plot of load versus notch width, the load at failure, P_Q, was determined. The fracture toughness, K_Q, for the size of the specimen tested was then determined from Eq. 1 (ASTM E399, 1983). K_Q is usually called the apparent fracture toughness and can be adopted as the plane strain fracture toughness, K_{IC}, if it is size independent.

$$K_Q = \frac{P_Q S}{B W^{3/2}} f\left(\frac{a}{W}\right) \qquad (1)$$

where $f(\frac{a}{W})$ is a function of $\frac{a}{W}$.

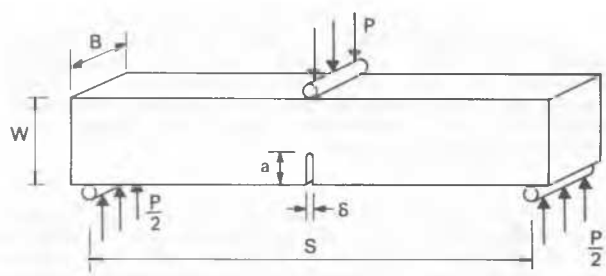

Figure 3 Test arrangement of three point loading of single edge cracked beams

The investigation mentioned above, showed that the fracture toughness of Johnstone was dependent on the span and depth of the specimen and independent of the thickness. For specimens with dimensions in the ratio of S:W:B of 4:2:1 the size effect can be interpreted as being dependent on one characteristic dimension, in this case W. The influence of W on the fracture toughness of Johnstone is illustrated in Fig. 4, which indicates that a specimen of this configuration must have a depth, W, of at least 60 mm to ensure that a reasonable estimate of the plane strain fracture toughness is obtained.

7 TENSILE STRENGTH DETERMINATION OF JOHNSTONE

There are a number of different methods for determining the tensile strength of rock. These include direct uniaxial tests on dogbone shaped specimens, Brazilian tests on discs cut from cylindrical cores, ring tests on hollow discs cut from cylindrical cores and 3 or 4 point bend tests on prismatic beam specimens. For the case of the fracture toughness tests on the SECB specimens, the tensile strength of the Johnstone was determined by three or four point bend tests on un-notched prismatic specimens (Haberfield, 1987). These specimens were re-machined directly from the broken SECB specimens, and had dimensions in the ratio of S:W:B of 4:2:1, with W varying between 35 and 65 mm. As two tensile test specimens were made from each SECB specimen, one specimen was tested in three point bending and the other in four point bending. The tests were conducted at the same loading rate as the fracture toughness tests and were analysed according to the method described by Vutukuri et al. (1974).

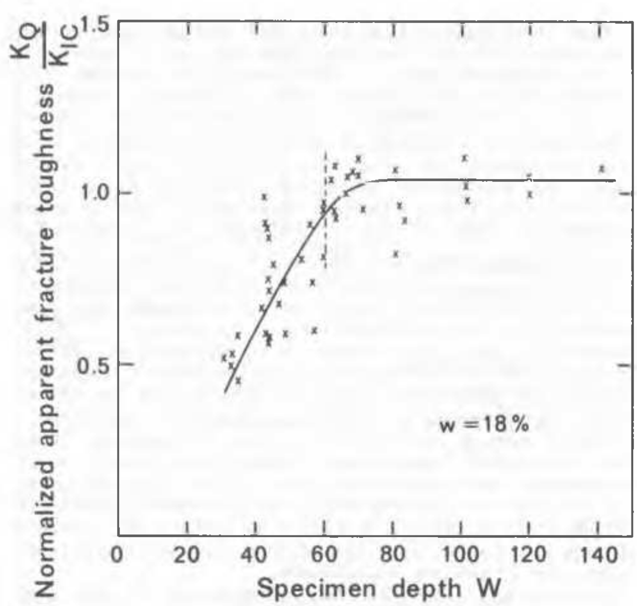

Figure 4 Influence of beam depth on fracture toughness

The results obtained from both tests were in agreement, and displayed no appreciable size effect. The results also agreed with the values of tensile strength determined by using the Brazilian, ring and direct methods (Choi, et al. 1988).

8 TENSILE STRENGTH/FRACTURE TOUGHNESS RELATIONSHIP FOR JOHNSTONE

From these experimental investigations, it was found that the plane strain fracture toughness of Johnstone increased with decreasing saturated water content (Fig. 5) This relationship is similar to the one between water content and tensile strength obtained by Choi, et al. (1988).

The relationship between fracture toughness and tensile strength is illustrated in the log-log plot of Fig. 6. Although the plot contains a moderate amount of scatter, it does indicate that there is a definite relationship between fracture toughness and tensile strength for this material.

9 TENSILE STRENGTH/FRACTURE TOUGHNESS RELATIONSHIP FOR OTHER GEOMATERIALS

Fig. 6 also includes the results from a number of tests on Melbourne mudstone. These tests are described in detail in Haberfield (1987). A search of the fracture mechanics literature revealed a number of fracture toughness investigations which had been carried out on hard rocks. Details of these results may be found in Haberfield (1987) and they are also presented in Fig. 6.

Considering that Fig. 6 includes the properties for a wide variety of rock types, incorporating a wide range of strengths, mineral compositions and formation methods, there appears to be a reasonably strong correlation between tensile strength and fracture toughness.

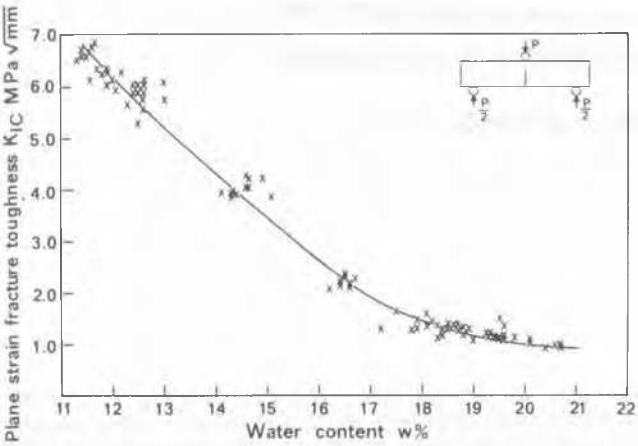

Figure 5 Variation of plane strain fracture toughness of Johnstone with saturated water content

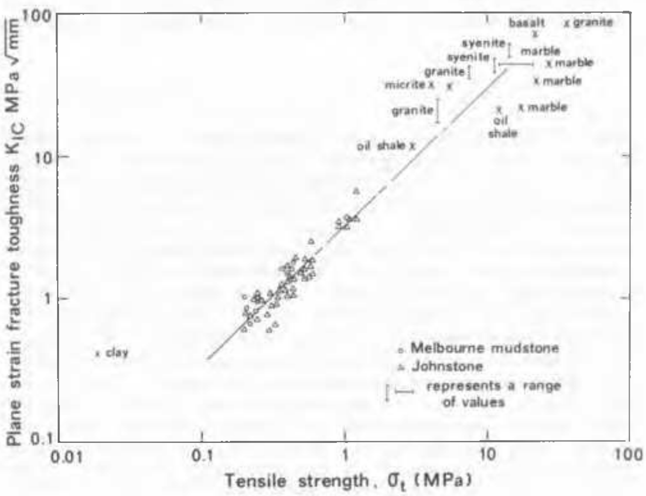

Figure 6 Relationship between plane strain fracture toughness and tensile strength for some rocks and soils

The authors were able to find only one reference in the literature pertaining to the fracture toughness of a soil. The results of this study (Lee et al., 1982) are also included in Fig. 6. The point representing this particular material (see Fig. 6) lies above the extrapolated trend of the other stronger materials. As is explained below, this is indicative of the relatively higher ductility of clay with respect to rock.

In fracture mechanics the ratio $(K_Q/\sigma_t)^2$ is used as both an indicator of the relative ductility of the material, and as a basis for estimating a minimum specimen size for the determination of plane strain fracture toughness. In general, the higher the ratio, the higher the ductility and the larger the specimen required. Fig. 6 indicates that this ratio ranges from approximately 9 for the harder rocks to 11 for the Johnstone and Melbourne mudstone and 466 for the clay. Hence it appears that this clay is much more ductile than the rocks, and also that much larger specimens than those described earlier may be required for the determination of fracture toughness of clay.

10 CONCLUSIONS

It has been argued that there are many situations in foundation engineering, especially when weak and weathered rocks are involved, where tensile failure governs behaviour. In such cases it is essential that tensile failure be modelled accurately using fracture mechanics theory. Hence it is imperative that accurate estimates of plane strain fracture toughness be obtained. Unfortunately, for a number of geomaterials and in particular clays, the samples obtained by standard site investigation techniques are unsuitable for fracture toughness testing purposes. However, as shown above there appears to be a strong correlation between plane strain fracture toughness and tensile strength, and hence in such cases, a reasonable estimate of fracture toughness may still be obtained if the tensile strength of the material is known.

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