

# In Situ Stress Measurements Using Hydraulic Fracturing in Jointed Rock in Hong Kong

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**SUMMARY** In situ stress measurements using the hydraulic fracturing technique have been conducted in Hong Kong during the preliminary design site investigation for the Cheung Ching Tunnel as part of the Route 3 Project. Measurements were conducted in shallow boreholes completed along the eastern low rock cover section of the proposed tunnel alignment on Tsing Yi Island. The magnitude and orientation of the minimum principal horizontal stress was of concern along this critical section of tunnel where rock cover varies between 16 and 30 metres above crown level. Information concerning the in situ state of stress was significant for the evaluation of the stability of the 1.5 km long, twin arrangement of unprecedented 17 metre span tunnels. Rock mass conditions along this section of the proposed tunnel alignment are characterised with closely spaced jointing and slightly to moderately weathered granitic rock. Near consistent minimum principal horizontal stresses were measured slightly in excess of overburden stress. Results from numerical modelling of the variable topography indicate a generally good agreement with the measured stresses.

## 1. Introduction

The increasing number of planned underground projects in Hong Kong has led to the requirement of investigating the in situ state of stress. Engineers involved with the design and construction of underground excavations must accept that the geologic medium in which an excavation is to be sited is subjected to an unknown pre-existing stress field. Unlike the design of structures on surface where the applied stresses are known with certainty and the behaviour of the structure can be accurately predicted, the unknown pre-existing stresses must be measured if the behaviour of an excavation is to be predicted for the assessment of stability.

The purpose of conducting in situ stress measurements in Hong Kong is to define the magnitude and direction of the in situ stress field such that this information can be considered for the overall evaluation of the stability of underground excavations. This information is of particular importance when large span underground excavations are considered and the mean joint spacings are small such that the ratio of excavation span to mean joint spacing is large. Information concerning the in situ state of stress will also provide valuable research related data for the overall seismic evaluation of Hong Kong.

The first in situ stress measurements have been conducted in Hong Kong using the hydraulic fracturing technique during the preliminary design stage site investigations for the Cheung Ching Tunnel as part of the Route 3 Project (Figure 1). These measurements were conducted in closely jointed and weathered granitic rock on Tsing Yi Island. The purpose of these measurements was to investigate the minimum principal stress existing along the eastern low rock cover section of the proposed tunnel alignment for the evaluation of the stability of the 17 metre span excavations along this tunnel section.

The contents of this paper include a brief description of the components and the origin of in situ stress in Section 2. In Section 3 the four main measurement methods are discussed and in Section 4 the traditional application of the hydraulic fracturing technique in non-jointed rock is discussed. The application, testing and results of the hydraulic fracturing technique in jointed rock in Hong Kong is discussed in Section 5. A discussion of these results is presented in Section 6 and conclusions regarding the technique are presented in Section 7.

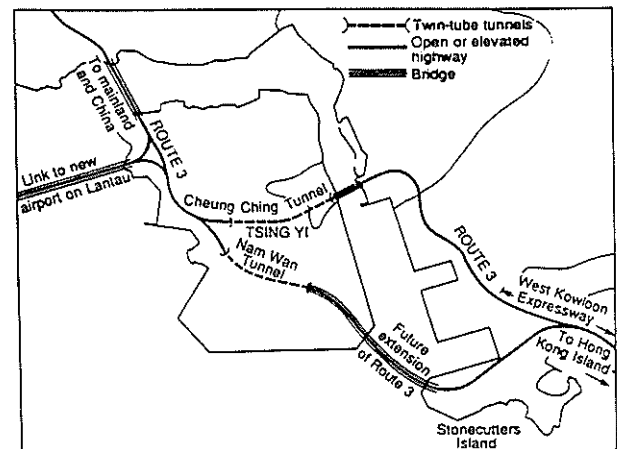


Figure 1 Cheung Ching Tunnel - Route 3 Project

## 2. In Situ Stresses

### 2.1 General

Stress is a mathematical tensor quantity that is defined by a magnitude, a direction and the plane on which it acts. Stress is a point property and the stress tensor can be represented for the unit cube in a matrix form and has six independent components as illustrated in Figure 2. There are three independent normal stress components denoted as  $\sigma_{xx}$ ,  $\sigma_{yy}$ , and  $\sigma_{zz}$  and three independent shear stress components denoted as  $\tau_{xy}$ ,  $\tau_{xz}$  and  $\tau_{yz}$ . At a particular orientation all three shear stress components diminish and all three normal stress components reach a maximum and are then referred to as principal stresses as denoted by  $\sigma_1$ ,  $\sigma_2$  and  $\sigma_3$  where  $\sigma_1$  is the maximum principal stress,  $\sigma_2$  is the intermediate principal stress and  $\sigma_3$  is the minimum principal stress. In some cases of in situ stress measurements it may be of only of interest to investigate the minimum principal stress such as for the design of pressure tunnel linings or for the evaluation of stability for large span excavations in jointed rock.

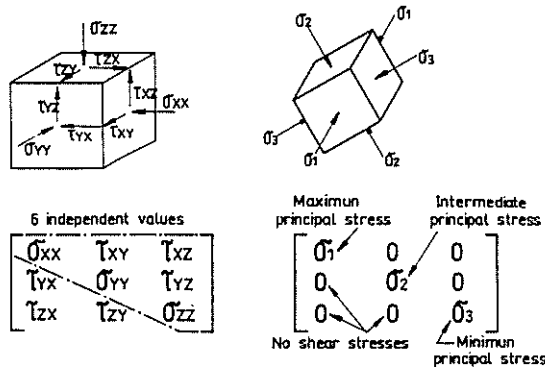


Figure 2 Stress Tensor

## 2.2 Origin of In Situ Stresses

The origin of in situ stresses stems from three main sources although secondary sources have also been recognised. The three main sources comprise gravitational, tectonic and residual stresses and secondary sources result from thermal and physio-chemical effects. Gravitational stresses are those stresses that are due to the weight of the superincumbent rock mass or overburden materials. Tectonic stresses are generated as a result of the relative displacements of the lithospheric plates. Residual stresses are often referred to as locked-in stresses and are the result of pre-existing effects and are most commonly attributed due to the sudden unloading of the earth's crust by erosion and de-glaciation. The effect of glaciers in places such as Canada and the Scandinavian countries has led to high horizontal stresses at relatively shallow depths. Residual stresses are commonly associated with variable topography such as is present in Hong Kong. The measurement of in situ stresses in a region of such variable topography are likely to yield anomalous in situ stresses with high horizontal directional stresses both along major ridges and depressions.

## 3. In Situ Stress Measurements

### 3.1 General

There currently exist four main methods to measure in situ stresses as recognised by the International Society for Rock Mechanics (ISRM, 1987). Different procedures and equipment as well as fundamental strategies are associated with each of these methods. Initial research into the measurement of in situ stresses in the 1960's focused on strain relief measurements assuming rock mass elasticity. Early developments in this research required that several overcoring strain relief measurements be conducted in order to determine the complete in situ state of stress. Further advances with this same work resulted in the development of a strain relief cell whereby a single test was adequate to determine the complete in situ state of stress (Leeman and Hayes, 1966). The hydraulic fracturing technique was developed as a direct measuring method in the early 1970's and is the result of the work of Hubert and Willis (1957) with early applications in the petroleum industry for the enhancement of oil well production.

Several other techniques have been developed and experimented in the attempt to measure the in situ state of stress. Some of these techniques have included large scale overcoring (Brady et al. 1976) as well as alternative forms of strain relief overcoring (Hiltsher et al. 1979). Only the four common methods of the ISRM are presented for the purpose of this paper.

### 3.2 Flat Jack

The Flat Jack method allows for the determination of a stress component parallel to and near an exposed excavation surface. The component of stress that is measured is not the virgin stress component and each measurement determines the stress only in a single direction, therefore six independent direction are required in order to determine the stress tensor.

The Flat Jack method requires the insertion of a welded flat envelope of material into a cut slot made in the side of an excavation. Prior to the cutting of the slot for the flat jack, measuring pins are installed and surveyed on either side of the slot. The component of stress acting perpendicular to the plane of the flat jack is determined as the pressure required to inflate the flat jack in order to return to the initial pin separation.

The Flat Jack method is not ideally suited for the determination of the undisturbed in situ state of stress due to practical considerations. The method however is commonly employed for the determination of stresses generated within tunnel linings long after construction for the purpose of providing design information for future excavations in a similar geological environment.

### 3.3 Borehole Deformation Gauge

The borehole deformation gauge method of measuring in situ stresses is based on the measurement of the changes in a borehole diameter during overcoring and has been modelled on a procedure developed by the United States Bureau of Mines (USBM). Both the deformation measurements and the elastic properties of the rock are considered in determining the 2D state of stress in the plane perpendicular to the axis of the borehole. In order to determine the complete stress tensor with this method test from three or more non-parallel boreholes must be conducted and analysed.

The distance away from an excavation for which this method can be used is only restricted by the accuracy of the overcore drilling and successful tests have been completed at distances of up to 200m. The borehole deformation gauge should be used only in relatively homogeneous rock with widely spaced jointing but can be used when boreholes are wet due to groundwater or from drilling water.

### 3.4 Soft Inclusion Strain Cell

Soft inclusion strain cells have been developed with 9 and 12 strain gauges in order to determine with a single test the complete stress tensor by relieving stresses through overcoring a borehole. The main problem associated with the use of strain cells are commonly related to inadequate bonding of the strain gauges to the rock.

The application of strain cells are ideally suited for homogeneous, isotropic rock where joints are widely spaced and the method is also limited by the accuracy of the overcore drilling. The determination of the components of in situ stress require the elastic properties of the rock.

### 3.5 Hydraulic Fracturing

The hydraulic fracturing technique can determine the three principal components of the complete in situ stress tensor using a single test with the assumption that the minimum principal stress,  $\sigma_3$ , is vertical along the orientation of the borehole. The technique is independent of the elastic properties of the rock and can be conducted at great depths. The technique is ideally suited for non-jointed rock or at least where the joints are widely spaced. The main problems associated with the technique are the determination of the instantaneous shut-in pressure as will be discussed later and leakage from the packer equipment.

4. Hydraulic Fracturing in Non-Jointed Rock

4.1 Measurement Procedure

Hydraulic fracturing is conducted within a sealed off section of a borehole. The sealing is commonly achieved using a double-straddle packer system consisting of two hydraulically activated rubber packers to pressurize a test interval until a fracture is created or a pre-existing fracture is re-opened within the rock material surrounding the test interval. According to the work of Hubert and Willis (1957) a hydraulic fracture will be created perpendicular to the minimum principal horizontal in situ stress. The first step in the measurement procedure is the selection of suitable test intervals upon inspection of the borehole core. Ideally, test intervals are selected where no jointing is present. Upon sealing off the selected test interval water is injected into the test interval at a controlled rate monitoring both time and test interval pressure. Once a hydraulic fracture is created the water injection is terminated and the decay in water pressure is closely monitored to examine the decay response to the moment the hydraulic fracture closes and water flow into the rock formation ceases.

The next step is the re-injection of water into the test interval to re-open the previously created hydraulic fracture. After the re-opening of the hydraulic fracture the water flow is again terminated and the pressure decay monitored. This re-opening cycle is repeated several times to confirm a consistent re-opening pressure. Figure 3 presents a typical Pressure-Time plot for a hydraulic fracture test. After a satisfactory pressure-time plot is recorded the test is complete and the double packer system is removed from the test interval. To confirm that a hydraulic fracture was created and to determine its orientation an impression packer survey of the test interval is conducted. Notable contributions for the application of hydraulic fracturing have been presented by Haimson (1974), Zoback and Pollard (1978) and Rummel et al.(1983).

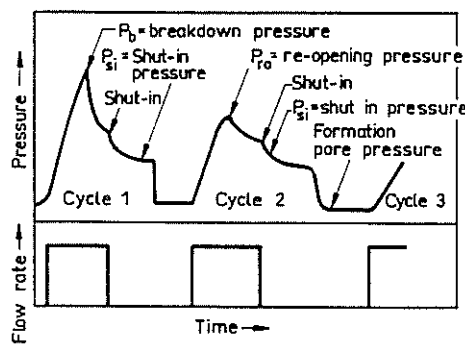


Figure 3 Typical Pressure-Time Plot

4.2 Equipment Requirements

The equipment requirements for conducting hydraulic fracturing tests are relatively simple however experienced staff are necessary to conduct the testing in order to obtain to meaningful information. Figure 4 illustrates a typical layout of the testing equipment including the double packer system, a surface readout and data acquisition unit and the impression packer survey tool. During the hydraulic fracturing test it is imperative that a surface readout of the pressure-time response is available in order to correctly monitor the behaviour of the test interval. Commonly used surface readouts include chart recorders or video displays from tape and computer disc data recording.

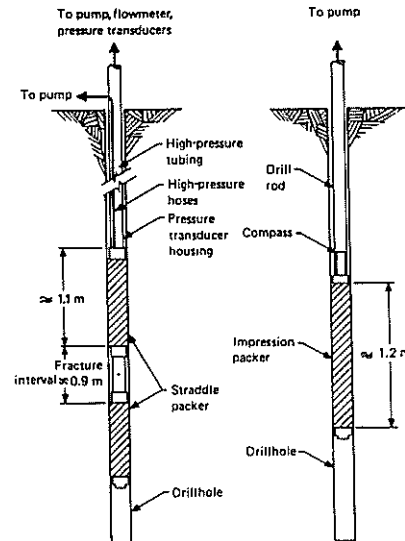


Figure 4 Testing Apparatus

4.3 Interpretation of Results

Interpretation of the results from hydraulic fracturing tests requires the identification of the breakdown pressure,  $P_b$ , the instantaneous shut-in pressure,  $P_{si}$  and the re-opening pressure,  $P_{ro}$  as highlighted on Figure 3. The breakdown pressure,  $P_b$ , is defined as that test interval pressure at which a new hydraulic fracture is created within the test interval. The instantaneous shut-in pressure,  $P_{si}$ , is defined as the test interval pressure at which the hydraulic fracture closes and water inflow into the rock mass no longer occurs. Shut-in pressures are commonly indistinct with hydraulic fracturing tests and several methods have been proposed for the determination of  $P_{si}$ . Aggson and Kim (1987) present an interesting comparison of five methods that define significantly different shut-in pressures. The re-opening pressure,  $P_{ro}$ , is defined as the test interval pressure at which the previously created hydraulic fracture is re-opened. The breakdown and re-opening pressures are clearly confirmed after repeated re-opening cycles have been completed.

In calculating the principal stress components from results of hydraulic fracturing tests it is assumed that the orientation of either the intermediate principal stress,  $\sigma_2$ , or the minimum principal stress,  $\sigma_3$ , is vertical. In the plane perpendicular to the borehole axis the stress components of interest are thus  $\sigma_h$  and  $\sigma_v$  which are respectively the minimum and maximum principal stresses and the calculation of these components for a vertical borehole in flat lying topography are based on the following expressions:

$$\sigma_h = P_{si} \tag{1}$$

$$\sigma_H = 3P_{si} - P_b - P_o + T(\text{for initial cycle}) \tag{2}$$

$$\sigma_H = 3P_{si} - P_{ro} - P_o(\text{for re-opening cycles}) \tag{3}$$

where  $P_o$  is the initial pore water pressure in the test interval and  $T$  is the tensile strength of the rock material. For the subsequent re-opening cycles  $T = 0$ . From the above expressions, if  $\sigma_h$  is less than the overburden stress,  $\sigma_v$ , then  $\sigma_h = \sigma_3$  and if  $\sigma_h$  is greater than  $\sigma_v$ , then  $\sigma_h = \sigma_2$ . The orientation of  $\sigma_h$  is perpendicular to the strike of the hydraulic fracture and  $\sigma_H$  is perpendicular to  $\sigma_h$ . The above expressions are valid for classical hydraulic fracturing in non-jointed, impermeable rock. In permeable rock formations poroelastic effects may be significant and must be considered (Detournay et al., 1989).

Hydraulic fracturing on pre-existing fractures has been investigated in detail by Cornet (1986) with the consideration of an inverse mathematical procedure for the determination of the principal stresses. Rummel (1987) has presented the calculation of stresses from hydraulic fracturing data based on fracture mechanics.

## 5. Hydraulic Fracturing in Jointed Rock in Hong Kong

### 5.1 Interpretation of Results

The interpretation of the results of hydraulic fracturing tests conducted in jointed rocks follows the same procedures as for non-jointed however the definition of the magnitudes of the breakdown pressure,  $P_b$ , the instantaneous shut-in pressure,  $P_{si}$  and the re-opening pressure,  $P_{ro}$  are not as easily selected from the pressure-time plot. The main reason for the greater difficulty in defining the characteristic pressures is due to leakage of the injected water from the test interval through joints present within the test interval. In order to define the characteristic pressures a detailed computer-based data analysis incorporating smoothing procedures with the construction of special cross-plots are commonly required (Baumgartner and Zoback, 1989). After a careful inspection of the observed hydraulic fracture orientation in the test interval in combination with the characteristic pressure values the determination of the in situ stress regime can be completed.

### 5.2 Testing and Results from Tsing Yi Island

A total of 25 tests were conducted in 4 boreholes at depths ranging from 8 to 50m below surface. Pressure-time data was recorded on a paper-chart recorder for continuous viewing as well as cassette tape recorder for subsequent re-processing. Test intervals were selected where either no or very few pre-existing joints were present.

An initial evaluation of the test data was conducted on site during the testing programme for comparative purposes. Out of a the total of 25 tests only 9 breakdown pressures could be confidently identified. A limited number of tests were unsuccessful due to leakage from the test interval that prevented pressure build up and thus new test intervals were selected. The detailed evaluation of all the test data was conducted after re-processing of the data at the office of MESY GmbH Bochum. Both pressure-time and logarithm plots were constructed for the determination of upper and lower bound shut-in pressures.

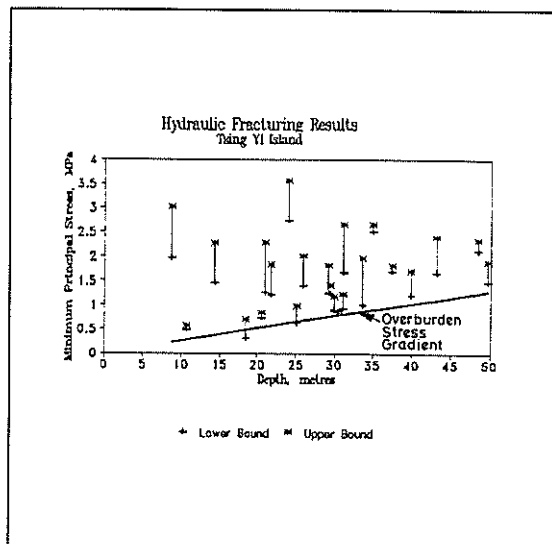


Figure 5 Tsing Yi Test Results

Test results presenting the pressure-time and flow rate-time data were typical for the Tsing Yi data. A plot of lower and upper bound minimum principal horizontal stresses versus depth for all the tests is presented in Figure 5. The results clearly indicate near consistent minimum principal horizontal stresses greater than the overburden stress. It is noted that the variability of the lower bound values is significantly less than for the upper bound values. The results of the impression packer surveys indicated an East-NorthEast orientation for the maximum principal horizontal stress, thus being sub-parallel to the proposed tunnel alignment.

### 5.3 Numerical Modelling

In an attempt to explain the interesting results from the hydraulic fracturing tests a numerical modelling analysis was conducted considering the variable topography of the site. The analysis was specifically aimed at investigating the variability of minimum principal horizontal stresses as determined from the different boreholes with the possible variability being a result of the topography at the site. The numerical modelling was also conducted to confirm the determined principal stress directions that were deduced from the results of the field tests. The 2D finite difference code, FLAC, was used to model the variable topography with initial stresses set using the hydraulic fracturing results from the one borehole located where the stresses were speculated to be not influenced by topography. The results of the numerical modelling analysis confirmed a general agreement to the measured stresses from the hydraulic fracturing tests. Figure 6 presents the calculated stress regime with borehole locations and Figure 7 presents a comparison of the results of the numerical modelling analysis to the measured stresses for one of the boreholes.

## 6. Discussion of Results

The results of the hydraulic fracturing tests indicating near consistent minimum principal horizontal stresses greater than the overburden stress are very encouraging upon consideration of the prevailing rock mass conditions. The uncertainty in determining the minimum principal horizontal stress has been represented in the form of upper and lower bound values. A non-linear variation of the minimum principal horizontal stresses was recorded for each of the four boreholes. This non-linearity can be attributed to the stress relaxation due to topography, inhomogeneous rock material properties and the influence of pre-existing joints on the prevailing stress field near the borehole.

## 7. Conclusions

The success of conducting hydraulic fracturing tests in closely spaced and slightly to moderate weathered granitic rock is very encouraging for the planning of further testing in similar rock mass conditions in Hong Kong. Although the absolute magnitude of in situ stresses is not of general concern in comparison to the strength of most rocks, the relative magnitude of the minimum principal horizontal stress compared to the overburden stress is of major concern for the evaluation of the stability of large span underground excavations. Hydraulic fracturing is a cost-effective method for the determination of the principal stress components versus other common methods having due regard of the rock mass conditions prevailing in Hong Kong.

## 8. Acknowledgements

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The hydraulic fracturing tests were carried out on site by Dr. Ing H. Konietzky and Dipl. Ing. Mr. P. Hegemann of MESY GmbH Bochum in consultation with the first author.

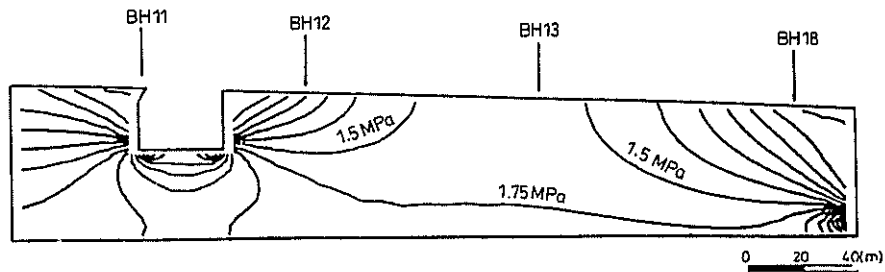


Figure 6 Calculated Stress Regime

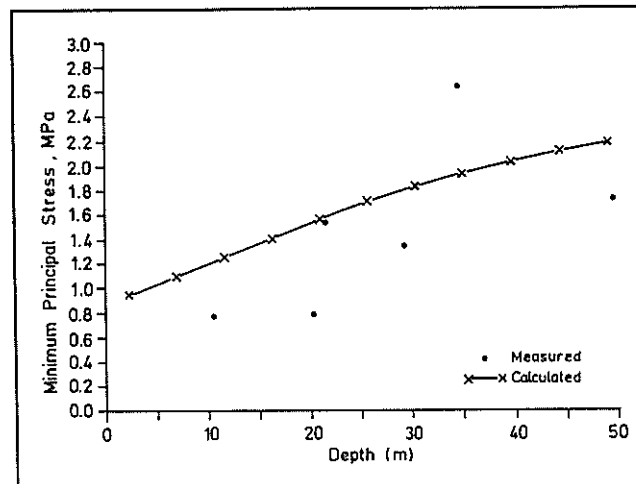


Figure 7 Comparison of Results for BH 12

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