Chapter

Hydraulic Fracturing in Porous and Fractured Rocks

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Abstract

There are various methods to determine in situ stress parameters, each having its own advantages and limitations. Among the methods available, the hydraulic fracturing method is the most adopted method for in situ stress measurements because of its simplicity and reliability. But the legitimacy of this method becomes questionable in fractured and porous rocks as the amount of experimental work has thus far been limited, especially in the case of its validity in fractured and porous rocks. The relatively slow rates of pressurisation have ensured that when fracture initiation occurs, the sudden increase in volume may lead to a marked drop in pressure in the fractured section, which is easily recognised from the pressure record. This is because pressure cannot be developed if the rate of leakage in the formation is equal to or higher than the flow rate applied for fracture initiation.

Keywords: hydraulic fracturing, fractured rocks, porous rocks, high flow rate, overcoring

1. Introduction

Hydraulic fracturing provides only plane principal stresses, and no information on the other components of the tri-axial stress field is available [1]. In hydraulic fracturing, continuous water pressure is applied in confined area which tends the rock to be tensile and while pressure exceeds the strength of rock, water escapes in weak plane formed [1–15].

Haimson studied on various rock specimens of variable pore pressures. Around 400 specimens have been tested under rational loading conditions. All the specimens failed under tensile manner. He was the one who pointed out the role of water pressure in fracture propagation. His study proved that water pressure increases the pore pressure in turn unable to obtain actual results. The reliability and validity of this method is also questionable when dealing with porous and fractured rocks encountered in underground mines [2–8, 16].

The main objective is to develop a proper and add-on technique for hydraulic fracturing for stress measurement in porous and fractured rocks. Hydraulic fracturing tests were conducted by using different flow rates of water inside the fractured rocks and high viscous fluid in porous strata. The stresses evaluated by this method was correlated with normal flow rate hydraulic fracturing method at the same locations where the rock mass was not fractured, and to circumvent the effect of the porousness, by overcoring technique since porosity does not have any influence on overcoring procedures. The correction factor was introduced during stress evaluation by hydraulic fracturing method in fractured and porous rocks. Normal flow rate is flow rate of fluid during hydraulic fracturing test ranging from 6 to 8 l/min.

This new technique will be helpful in conducting the stress measurements in porous and fractured rocks, which will be highly beneficial to both mining and hydropower related excavations.

The measurement of the state of in-situ rock stress provides essential data for the rational design of underground excavations based on the principles of rock mechanics [13].

The hydraulic fracturing test to determine the stress tensor is rather simple and robust, and it also gives the required magnitude and orientation of the maximum principal stress [17]. Several techniques and equipment have been developed, and are still being developed, to measure this parameter [8].

The main disadvantage of this hydraulic fracturing method when compared to other methods such as overcoring, flat jack and stress-meter, is its limitation when applied to porous and fractured rocks [14, 18]. Rock mass may contain natural occurring discontinuities, including fractures which dissipate fracturing liquid. Hence, it is more difficult to use the hydraulic fracturing process to determine stress conditions in porous and fractured rocks. Whereas in 'non-fractured rock mass', i.e., rock mass without fractures, this limitation is not there. As rocks in a large number of underground coal mines belong to this category, i.e., porous and fractured rocks, finding a methodology to accommodate such rock conditions is essential.

If a high flow rate of fluid is used, experience has shown that there is a tendency of induced fractures to rotate and change the direction of the initial fracture. As the direction of the induced fracture is one of the input parameters for the evaluation of hydraulic fracturing stress, any change in the direction of fracture due to the influence of some external factor, like the flow rate, will give rise to an anomalous pressure or stress value [15].

If, instead of water, a higher viscosity fluid is used for fracture initiation, pressure can be readily developed inside the induced or pre-existing fractures which can be taken for evaluation of stress, but the influence of viscosity on the evaluation of magnitude and direction of stress is not validated.

The above discussed two points show that the limitation in adopting hydraulic fracturing method in porous and fractured rocks is rather due to non-availability of proper technique than the principle of hydraulic fracturing.

2. Hydraulic testing of pre-existing fractures method

Hydraulic testing of pre-existing fractures (HTPF) method provides an evaluation of the complete stress tensor (six components), independent of borehole orientation and material properties.

A portion of a borehole is closed off by use of two inflatable rubber packers adequately pressurized so that they hold on to the borehole wall (**Figure 1**). The water is pumped under continuous flow rate into the portion, gradually increasing the pressure on the borehole wall until a fracture is begun in the rock, or a pre-existing fracture is opened. Pumping is halted, allowing the interval pressure to deteriorate. Several minutes into the shut-off phase, the pressure is released and allowed to return to ambient circumstances. The pressure cycle is repetitive several times maintaining the same flow rate. Key pressure values used in the computation of the in-situ stresses are plucked from the pressure-time record. The repeated cycles deliver redundant interpretations of the key pressures. The attitude of the induced hydraulic fracture, or of the pre-existing fracture, is achieved using an

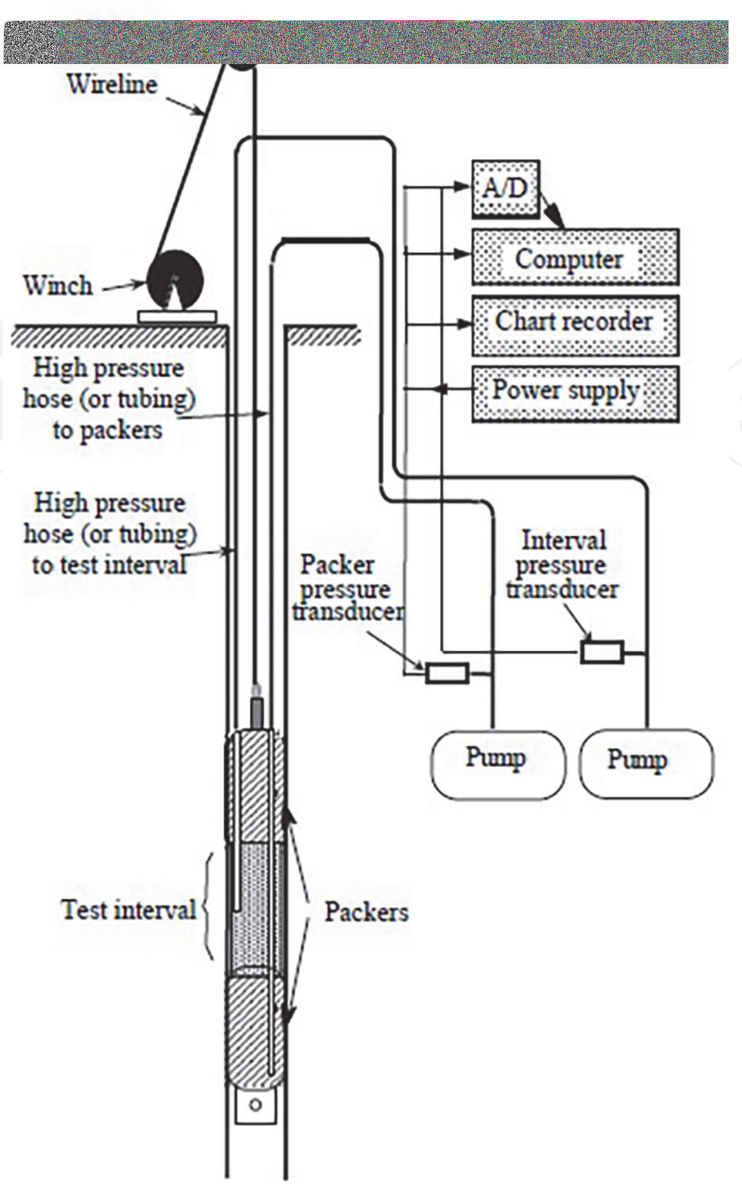

Figure 1.
Typical HTPF test equipment setup.

oriented impression packer. Hydraulic fracturing orientation is related to the directions of the principal stresses [1–15].

HTPF, tests yield an evaluation of the normal stress supported by fracture planes with different known orientations, and the complete stress evaluation results from an inversion of these results.

The main difference between HTPF and Hydraulic fracturing tests are certain assumptions made; otherwise, the process remains same. The following are certain assumptions:

1. There is no theoretical limit to the depth of measurement, provided a stable borehole can access the zone of interest [3].

2. The method assumes that isolated pre-existing fractures, or weakness planes, are present in the rock mass, that they are not all aligned within a narrow range of directions and inclinations, and that they can be instantly opened by hydraulic tests. When the straddled interval includes multiple fractures, it is necessary to verify that only one single fracture has been opened, for the opening of pre-existing fractures change the local stress field [6].

3. Fractures used in stress computations are delineated on the borehole wall under the assumption that their orientation persists away from the hole [5].

4. For a complete stress tensor determination, the method requires a theoretical minimum of six tests.

5. The procedure is applicable for all borehole orientations. It is independent of pore pressure impacts and does not involve any material property determinations.

6. It presumes that the rock mass is consistent within the volume of interest. When tested fractures are isolated from one another by more than 50 m, a hypothesis on stress gradients is essential.

Following are the assumptions in hydraulic fracturing technique:

1. There is no theoretical limit to the depth of measurement, provided a stable borehole can access the zone of interest and the rock is elastic and brittle.

2. Principal stress directions are obtained from the fracture demarcation on the borehole wall under the notion that fracture attitude persists away from the hole.

3. Evaluation of the maximum principal stresses assumes that the rock mass is linearly elastic, homogeneous, and isotropic. It involves considerations of pore pressure effects.

2.1 Hydraulic fracturing in fractured rocks

In Haimson's thesis [10], about 400 tests on hollow cylindrical and cubical specimens of 5 different fractured and non-fractured rocks were conducted under constant tri-axial external loading and increasing borehole fluid pressure [12]. In all of the samples tested, the induced hydraulic fractures were always found to be tensile and no shear failure was observed. The fractures in all the rock types were either vertical or horizontal depending on the applied stresses. These fractures are observed in pairs, mostly parallel to the nearly vertical wellbore axes, and on diametrically opposite sides of the borehole walls [15].

Haimson and Fairhurst [19]showed that the pumped flow increases the pore fluid pressure in fractured/porous formations and produces additional stresses and displacements (**Figure 2**).

Hence it is difficult to get the breakdown pressure (Pc) or the peak pressure in the first pressure cycle in normal flow rates in fractured rocks. Before reaching its peak, pressure typically declines even if pumping is continued at the initial flow rate as the pressure required to induce a hydraulic fracture in HF tests, or fracture reopening in hydraulic fracturing tests on preexisting fractures (HTPF) tests is not sufficient enough. It clearly indicates the following:

1. Critical pressure cannot be reached

2. The slope declines in the pressure–time curve,

3. Declining slope with constant flow rate in subsequent cycles

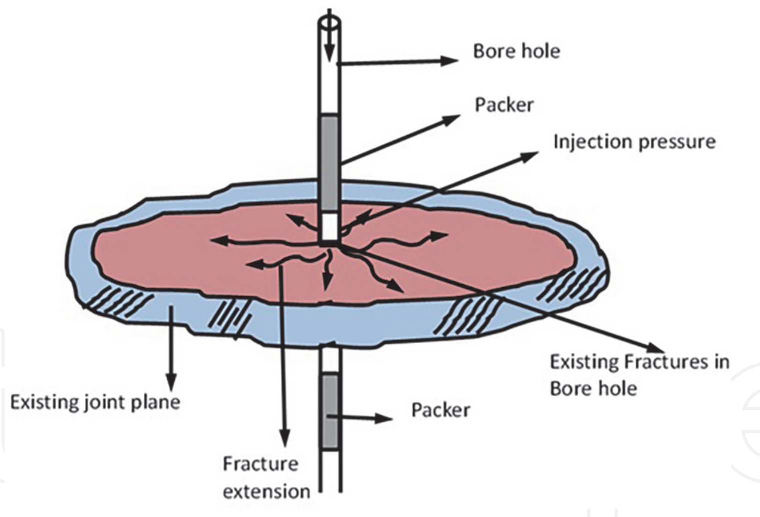

Figure 2.
Existing fractures in a borehole.

4. Shut-in pressure cannot be reached, signifies that maximum fluid is infiltrated in the fractures. (The shut-in pressure denotes at which a hydrofracture pauses generating and closes following pump shut-off. The determination of the shut-in pressure P_{si} is when a sharp break is detected in the pressure-time curve after the initial fast pressure drop following pump shut-off) [9].

In normal conditions or in good rock mass, the shut-in pressure (*P*si) reaches, after the pump is shut off following breakdown or fracture reopening. But in the present case in fractured rocks, shut-in pressure cannot be achieved even after repeated cycles. The first difficulty is the pressure decay just before shutting off, and the other difficulty will be not getting shut in pressure to calculate the minimum principal stress [17, 20].

2.2 Difficulties for stress measurements in fractured rocks by various methods

a. The overcoring test method does not permit the testing of rock mass with preexisting fractures with in the test section. The presence of fractures at or near the strain gauges results in erroneous measurements. In addition, the presence of fractures prevents a suitable length section of core being obtained for biaxial testing and determination of the elastic properties of the rock.

b. Flat jack measurements give only induced stress of the area, hence this method is also not suitable.

c. Classical method of hydraulic fracturing test is not suitable as the new fracture cannot be created in the zone of already existing fractures.

d. HTPF method of normal flow rate of water is also not suitable to reopen the existing fractures as the pressure is not sufficient to create the fracture or reopen the existing fracture.

2.3 Solution through innovation

- High flow rate technique with HTPF method can be suitably used in fractured rocks

- High viscous liquid instead of water can also be used for stress measurements in fractured rocks

The above solutions are originally proposed by the author. It will be patented soon at Office of Controller General of Patents, Government of India.

2.3.1 Methodology adopted

1. Hydraulic fracturing measurements were conducted by using different flow rates of water inside the fractured rocks. The stresses evaluated by this method was correlated with normal hydraulic fracturing method at the same locations where the rock mass was not fractured.

2. Hydraulic fracturing measurements were conducted inside the boreholes by using a viscous fluid. The stress evaluation was made using latest software. The stresses evaluated by hydraulic fracturing with viscous liquid method were correlated with stress measured by overcoring method. The stress measured by overcoring method was used as bench-mark for validation as this method does not influence the presence of porous feature in the rock. The correction factor was introduced in the stress evaluation by hydraulic fracturing method in porous rocks.

2.4 High flow rate technique in fractured rocks

In the literature on hydraulic fracture experiments it is generally assumed that a crack will initiate when the tensile stress at the borehole wall exceeds the tensile strength of the rock. It is possible, however, that in regions under tectonic shear stress, shear failure could be induced in the rock about the borehole at much lower fluid pressures than would be required to produce tension cracks, simply by lowering the effective pressure (confining pressure minus pore pressure) to the point where the shear strength of the rock is exceeded [14].

Haimson [11] showed that the compressional strength of the rock mass depends on effective pressure and differential stress. He suggested that a sample subjected to a given confining pressure and differential stress could be made to fail in shear or tension simply by controlling the pore pressure [21]. One way of testing this hypothesis would be to vary the pore fluid injection rate. At slow injection rates the water or any other fluid which is having low viscosity would have time to be drawn-out into the fractured zones and lower the effective pressure, whereas at fast injection rates a steep pore pressure gradient would develop near the borehole. If fluid were pressurized fast enough, even though the shear strength of the rock near the borehole would be surpassed, the load on the area would be supported by the neighbouring rock in which the pore pressure was still low. In this way, shear failure of the sample would not occur and instead, a tension crack would form when the tensile strength of the rock near the borehole was exceeded [7].

In settings with extreme overpressure, pore-water pressure approaches the pressure required for natural hydraulic fracturing. Unlike other fractured seals, hydraulic fractures remain open only if pore pressure exceeds fracture pressure [13].

To test this hypothesis, a series of 24 experiments was conducted at different zones inside the EX-size boreholes (core drilled boreholes of 38 mm diameter) where the rock mass is highly fractured. In all these experiments, the differential stresses were ranging from 10 to 200 bars and the fluid injection rates were varying by 4–16 l/min. It was assumed that the failure mechanisms (shear or tension) observed for different injection rates would be controlled by the pore pressure distribution in each test at the time of failure. The results are validated with normal flow rate of HTPF method in good rock mass zones of the same bore holes. Rock mass quality are characterised using a rating system. The rock mass is categorised into different classes (i.e., very good to very poor), incorporating the combined effects of different geological and geotechnical properties. This enables the comparison of rock mass conditions throughout the site and the delineation of regions of the rock mass ranging from 'very good' to 'very poor', thus providing a map of the boundaries of rock mass quality. The details of the investigations, stress evaluation procedure in fractured rocks and the results are given below.

2.5 Balloon phenomena

At slow bloating rate it is very difficult to inflate the punctured balloon as the air will be leaked through the hole, but at the heavy bloating rate it is possible to inflate the punctured balloon even though the leakage exists. Hence the solution is the rate of bloating should be much higher than the rate of the leakage through the puncture (**Figure 3**). The same balloon phenomena are applicable in the case of hydraulic fracturing testing in fractured rocks. At slow injection rates the fluid would have time to diffuse into the fractures and lower the effective pressure, whereas at fast injection rates a steep pore pressure gradient would develop near the borehole, i.e., If fluid were injected fast enough, even though the shear strength of the rock near the borehole would be exceeded, the load on the sample would be supported by the surrounding rock in which the pore pressure was still low. In this way, shear failure of the sample would not occur and instead, a tension crack would form when the tensile strength of the rock near the borehole is exceeded (**Figure 4**). Hydraulic fracturing is initiated when the fluid pressure exceeds the minimum principal compressive stress by the tensile strength of the host rock. Typical in-situ tensile strengths of rocks are in the order of 0.5–6 MPa (Haimson & Rummel [22], Amadei & Stephansson [23], Enever & Chopra [24]) [2, 19]. The propagation is made possible by the linking up of discontinuities in the host rock ahead of the hydraulic fracturing tip. Discontinuities are significant mechanical breaks in the rock, normally with low or negligible tensile strengths.

2.6 Brief about project area

The experiments have been conducted inside the underground tunnels of proposed underground powerhouse and intake drift area of at Teesta Stage-IV Hydroelectric

Figure 3.
Balloon phenomena similar to hydraulic fracturing test in fractured/porous rocks.

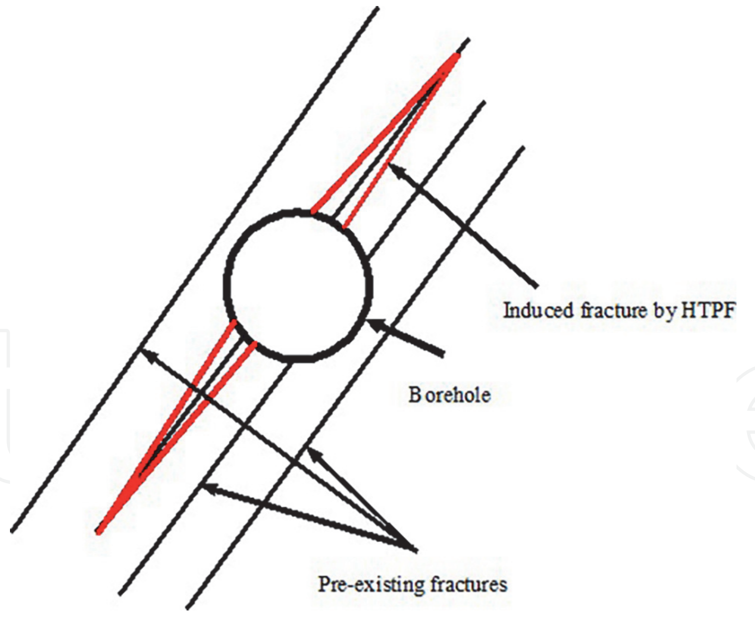

Figure 4.
Induced fracture/reopening of existing fracture in fractured rocks by high flow rate technique.

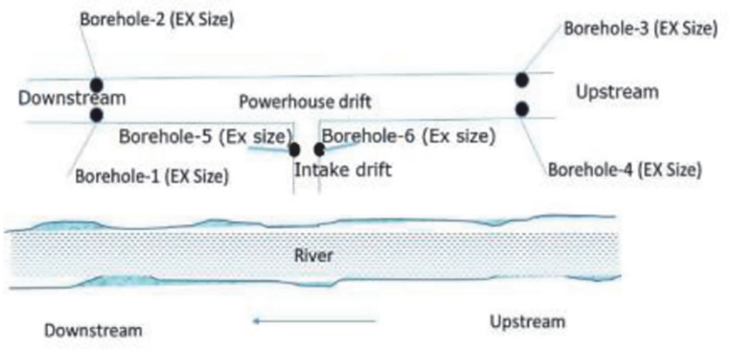

Figure 5.
Configuration of boreholes at powerhouse area-Teesta Stage-IV HEP.

project (**Figure 5**). Teesta-IV Hydroelectric Project was conceptualized in North Sikkim district, Sikkim for harnessing the hydro-power potentiality of Teesta River. The project is located in village Sangklang near Mangan in North Sikkim District.

The geology of the project area is represented by, quartzose phylite with garnet like crystals & ferruginous quartzite. The borehole logging and the cores retrieved from the boreholes are shown in **Figure 6**a–**c**.

2.7 Investigation procedure

Experiment procedure involves (**Figure 7**) selection of study area in fractured and non-fractured rock mass, conducting hydraulic fracturing tests in study area by

Figure 6.
Cores retrieved from the borehole at (a) and (b) powerhouse area; (c) intake drift.

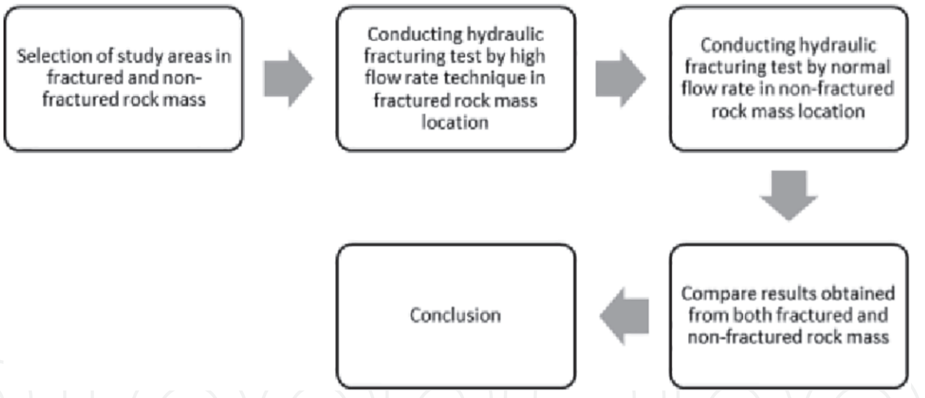

Figure 7.
Flowchart describing methodology and experimental procedures.

high flow rate in fractured rock mass and normal flow rate in non-fractured rock mass. At last, compare the results obtained with fractured and non-fractured rock mass and concluding with results.

After the hydraulic fracturing assembly was positioned at a pre-determined test section where the rock mass is highly fractured (selected based on core inspection **Figure 6**). The back flow from the fracture into the interval section was observed by short valve closures during the venting phase. Finally, the packers were deflated, and tool was moved to the next test section. After all the hydraulic fracturing tests were conducted in all the boreholes, an impression packer tool with a soft rubber skin was run into the holes to obtain information of the orientation of the induced or opened fracture traces at the borehole wall (**Figure 8**).

Experiment 1 (powerhouse upstream)

In trial 1, the experiments were conducted in the EX-size hole at the depth of 10–16 m where the rock mass was completely fractured. This particular zone was selected after careful observation of core logging data. The injection unit was placed at this depth for the pressurization. The pressure was injected at a rate of 6 l/min for a duration of 50–250 sec and the pressure was instantaneously increased up to 50 bars. Critical pressure could not be reached which eventually dropped to zero at the end of cycle. The shut- in pressure could not be achieved even though the pump was shut-off at certain peak levels (**Figure 9**). It clearly indicated that water has

Figure 8.
Tracing of fractures from impression packer at powerhouse downstream area.

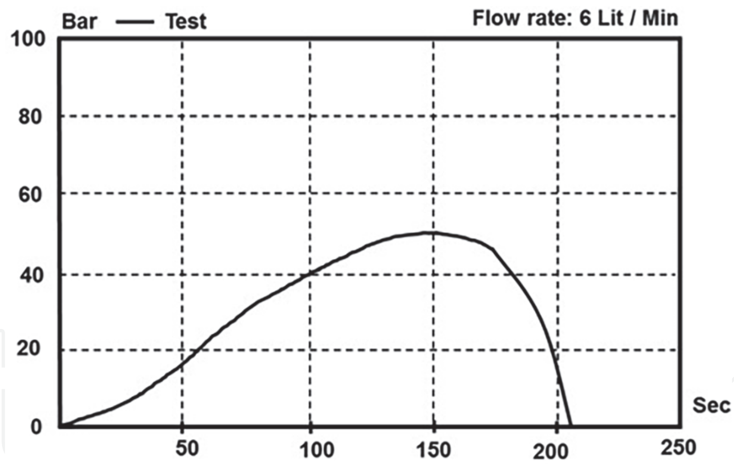

Figure 9.
Experiment 1.

been escaped from the existing fractures and the required pressure could not develop to reopen the fracture. Normal stress required for reopening of the pressure could not build up across the fracture. The pressure time diagram for the flow rate of 6 l/min is given below (**Figure 9**).

Experiment 2

In trial 2, the experiments were conducted in the EX-size hole at the depth of 10–16 m where the rock mass was completely fractured. This particular zone was selected after careful observation of core logging data. The injection unit was placed at this depth for the pressurization of the zone. The pressure was injected at a rate of 9 l/min for a duration of 50–250 sec and the pressure was increased up to 60 bars. Critical pressure could not be reached but there was a decline in the pressure which eventually dropped to zero at the end of cycle. The shut- in pressure could not be achieved even though the pump was shutoff at certain peak levels (**Figure 10**). It clearly indicated that water has been escaped from the existing fractures and the required pressure could not develop to reopen the fracture. Normal stress required for reopening of the pressure could not build up across the fracture. The pressure time diagram for the flow rate of 9 l/min is given below.

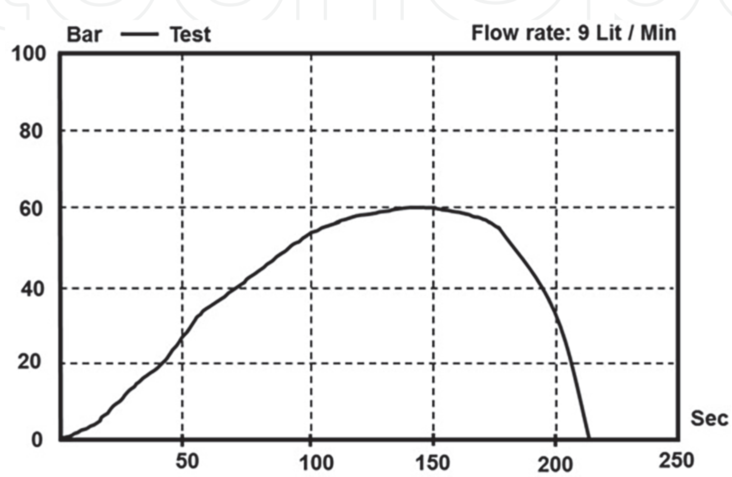

Figure 10.
Experiment 2.

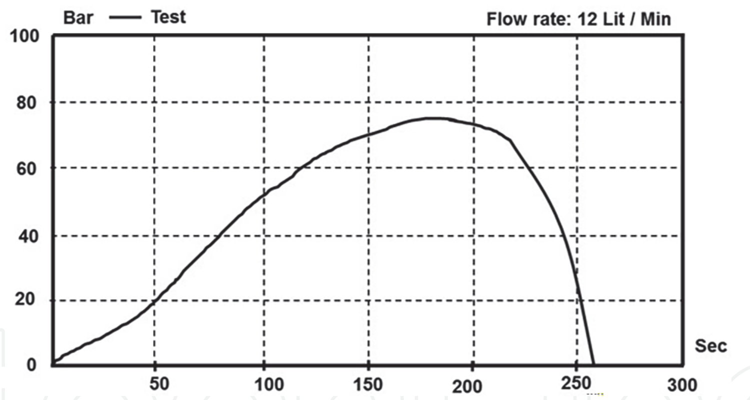

Figure 11.
Experiment 3.

Experiment 3

In trial 3, the experiments were conducted in the EX-size hole at the depth of 10–16 m where the rock mass was completely fractured. This particular zone was selected after careful observation of core logging data. The injection unit was placed at this depth for the pressurization. The pressure was injected at a rate of 12 l/min for a duration of 50–250 sec and the pressure was increased up to 70 bars. Critical pressure could not be reached but there was a decline in the pressure and which eventually dropped to zero at the end of cycle. The shut-in pressure could not be achieved even though the pump was shutoff at certain peak levels (**Figure 11**). It clearly indicated that water has been escaped from the existing fractures and the required pressure could not develop to reopen the fracture. Normal stress required for reopening of the pressure could not build up across the fracture. The pressure time diagram for the flow rate of 12 l/min is given below.

Experiment 4

In trial 4, the experiments were conducted at the same depth of 10–16 m where the earlier experiments were conducted with the flow rate of 6, 9 and 12 l/min. But in this case the flow rate was instantaneously increased to 15 l/min. The pressure was injected at a rate of 15 l/min for a duration of 80 sec and the pressure was automatically increased up to 95 bars. In the first cycle a clear critical pressure could be reached and there was no declining of pressure abruptly. Shut in pressure obtained at 50 bars after the pump was shut off. It clearly indicated that water flow has been required 15 l/min for the existing fractures to reopen.

Where P_{si} is the shut-in pressure, represented as the point of intersection between the tangent to the pressure curve immediately after pump shut-off and that to the late stable section of the pressure curve (**Figure 12**) (Enever and Chopra, 1986). The pressure time diagram for the flow rate of 15 l/min is given below (**Figure 13**).

3. Stress evaluation procedure and results

The in-situ stress measurements were conducted under the following situations:

i. Influence of topography.

ii. Presence of anisotropic rock.

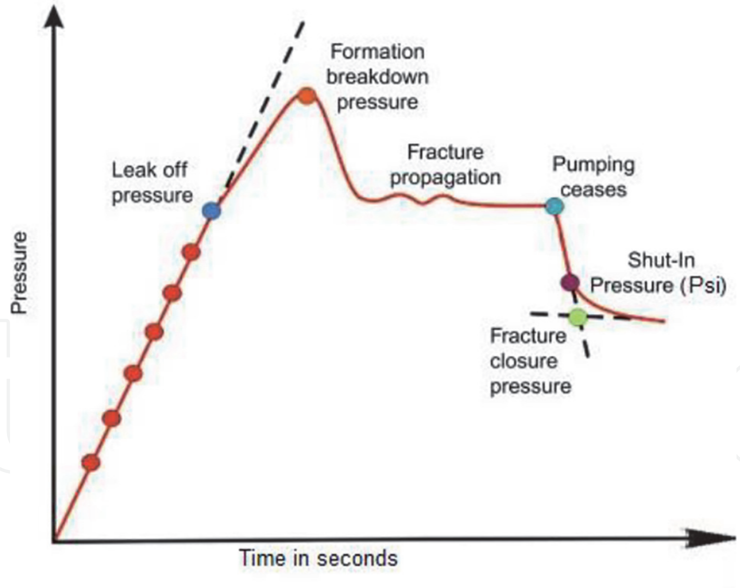

Figure 12.
Shut-in pressure related to Hydraulic fracturing.

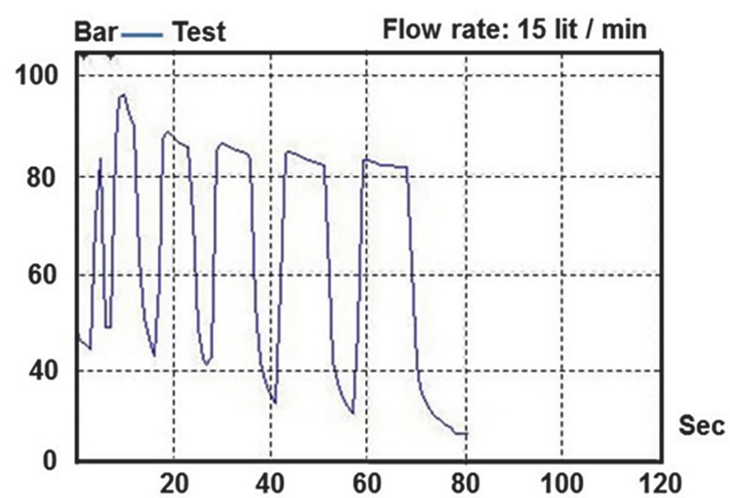

Figure 13.
Experiment 4.

Topography is the study of the land surface and forms the basis for landscapes. For example, topography refers to mountains, valleys, rivers, and craters on earth surface. If a tunnel is being excavated beneath a land consisting of different rock covers or overburden layers, anisotropic conditions are imminent [18].

3.1 Fracture orientation analysis—PLANE

The orientation (strike, dip angle and dip direction) of induced fracture traces obtained from impression packer testing is determined with the program PLANE in consideration of the borehole diameter and orientation [17]. Also, it differs with fracture traces as shown below.

Case I: Vertical borehole—angle from north to mark (0–360 degrees)

Case Ia: Fracture traces parallel to the borehole axis. Distance from mark (reference line) to fracture trace (**Figure 14**).

Case Ib: Inclined fractures (**Figure 15**).

Case II: Inclined borehole—angle from vertical line to reference mark (0–360 degrees)

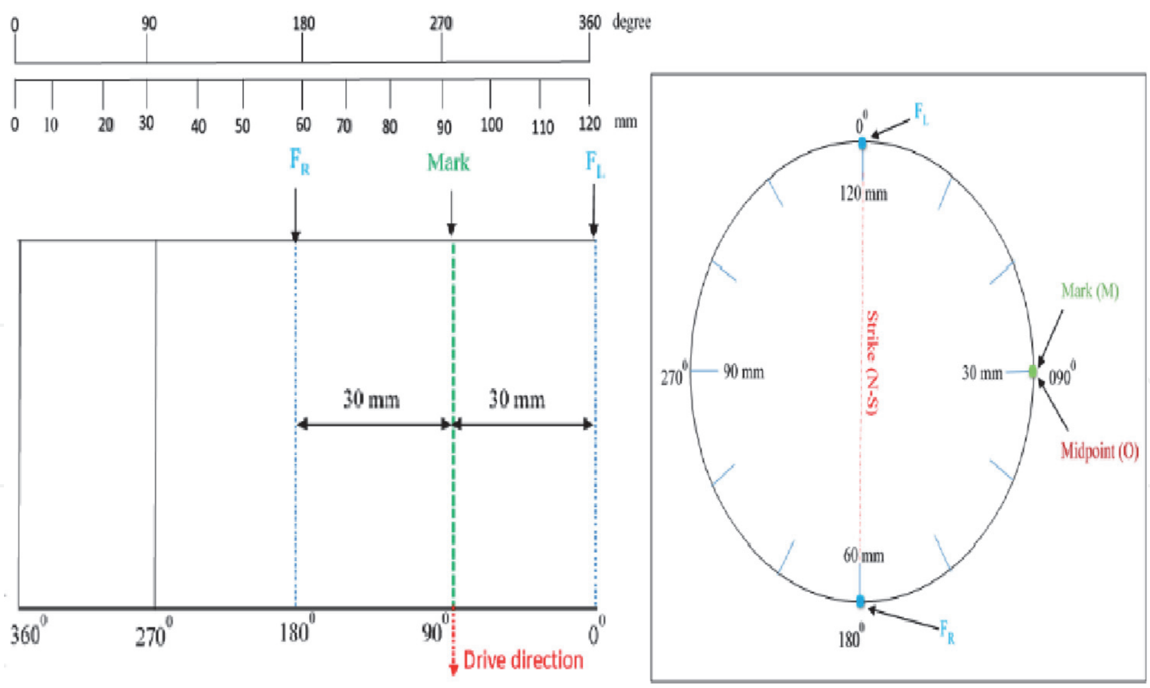

Figure 14.
Trace of the fracture is parallel to the mark found both sides.

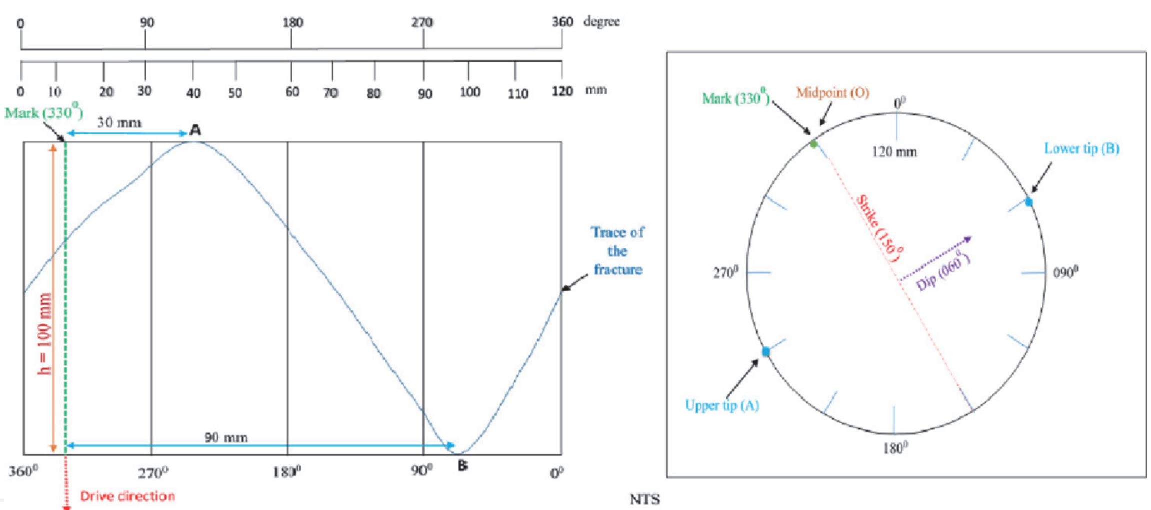

Figure 15.
Inclined fractures.

Case IIa: Fracture traces parallel to the borehole axis.
Case IIb: Inclined fractures

In all cases the result of the calculations is the strike direction (North Over East), the direction of inclination (North over East) and the inclination (90 degrees = vertical) of the fracture plane.

3.2 Data Interpretation code GENSIM

Impression packer tests suggest that in many cases hydraulic testing had been carried out along differently orientated fractures in the rock mass. The interpretation of these data requires sophisticated methods like the GENSIM rather than simple classical hydrofrac hypothesis.

The GENSIM algorithm assumes that the vertical is a principle stress axis and the vertical stress is equal to the weight of overburden rock stress. Stress–depth dependence is neglected within the program through which GENSIM is limited to short depth intervals

$$\sigma_h = (P_{si} - n2.\sigma_V)/(m2 + l2.\sigma_H/\sigma_h) \quad (1)$$

Where, l, m, n in the equation are the cosines of the direction of the induced fracture planes obtained during study; P_{si} is the shut-in pressure in MPa obtained from pressure record for certain depths and σ_H/σ_h is the stress ratio condition which prevails in ground conditions as 1.5, 2 and 2.5 considered for determining the minor principal horizontal stress.

Results in **Table 1** were done by using shut-in pressure data as given in **Tables 2–4** derived from the measurements in the borehole and varying the ratio $\sigma H/\sigma h$ and the strike direction of σH in the horizontal plane.

The stress gradient is plotted to observe any induced stress due to excavation or topography of rock cover (**Figure 16**). It is clearly understood that results determined are free from any influence.

Principal stresses	Intake drift	powerhouse drift (upstream)	powerhouse drift (downstream)
Vertical stress (σ_v) in MPa (calculated with a rock cover 160 m and density of rock = 2.7 g/cc)	4.24	5.08	6.35
Maximum horizontal principal stress (σ_H) in MPa	6.81 ± 1.26	5.46 ± 1.905	8.88 ± 0.855
Minimum horizontal principal stress (σ_h) in MPa	4.54 ± 0.84	3.64 ± 1.27	5.92 ± 0.57
Maximum horizontal principal stress direction	N 20 degrees	N 120 degrees	N 110 degrees
$K = \sigma_H/\sigma_v$	1.60	1.35	1.39

Table 1.
Stress tensors as evaluated at various locations.

Sl. no.	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure, P_{si} (MPa)
1	8.1	40	3.7
2	34.4	40	6
3	65.8	22.3	4
4	68.4	159.0	6.3

Table 2.
Fracture orientation data obtained from BH-1 and BH-2 with high flow rate-15 l/min (location: Powerhouse upstream; Teesta stage-IV Hydroelectric project West Bengal).

Sl. no.	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure, P_{si} (MPa)
1	72	175	4
2	39	139	5
3	23	26	5
4	64	54	4
5	60	57	6

Table 3.
Fracture orientation data obtained from BH-5 and BH-6 with high flow rate-15 l/min (location: intake drift; Teesta stage-IV Hydroelectric project West Bengal).

Sl. no.	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure, P_{si} (MPa)
1	44.2	171	7.6
2	63.2	136.1	6.5
3	77.4	52.6	7.8
4	58.5	156.8	6.0

Table 4.
Fracture orientation data obtained from BH-3 and BH-4 with high flow rate-15 l/min (location: powerhouse downstream; Teesta stage-IV Hydroelectric project West Bengal).

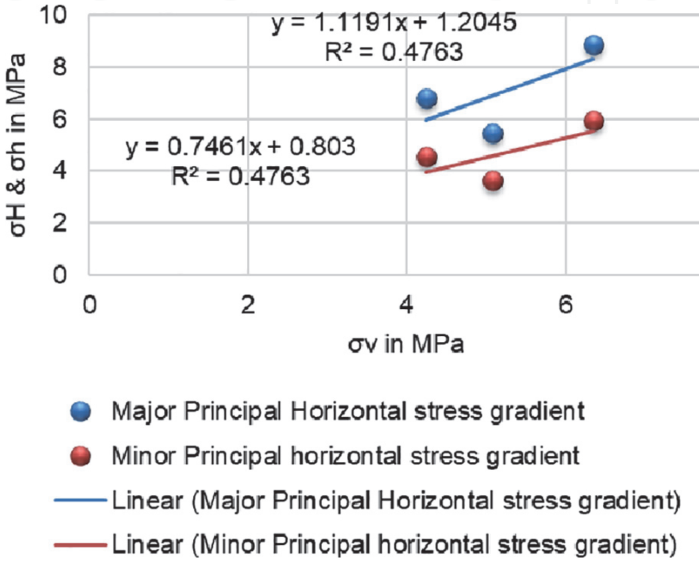

Figure 16.
Stress gradient in fracture rock mass.

4. Hydraulic fracturing test with normal flow rate in same test section of good rock mass

It is necessary to find out or validate the results obtained by using high flow rate technique to measure the maximum principal horizontal stress and its direction in highly fractured rock mass. Hence two to three zones of good rock mass area where the rock mass is not highly fractured were identified in the same borehole and conducted the experiments with normal flow rate method of 4–6 l/min to create the new fracture for classical hypothesis or to reopen the existing fractures. Tests with normal flow rate in non-fractured rock mass would give nearer result to correlate with the high flow rate technique. Other methods may show some difference

All the experiments were conducted in the EX-size hole at the depth where the rock mass was not fractured. These particular zones were selected after careful observation of core logging data. The injection unit was placed at this depth for the pressurization. The pressure was injected at a rate of 6 l/min for a span of 50–250 sec and the pressure was instantaneously increased up to 90–100 bars. The shut-in pressure could be achieved even though the pump was shut-off at certain peak levels [25]. It clearly indicated that normal stress required for reopening of the pressure could build up across the fracture [21]. The detailed procedure of the experiments, results obtained at different places are given below (**Tables 5–8**).

Sl. no	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure P_{si} (MPa)
1	8.11	40	3.76
2	34.44	40	6
3	65.88	22.36	4
4	68.46	159.07	6.3

Table 5.
Pressure and fracture orientation data derived from BH-1 and BH-2 with normal flow rate (6 l/min) in the powerhouse drift (upstream); Teesta stage-IV HEP West Bengal.

Sl. no	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure P_{si} (MPa)
1	44.28	171	7.7
2	63.27	136.14	6.6
3	77.4	52.65	7.8
4	58.5	156.82	6.0

Table 6.
Pressure and fracture orientation data derived from BH-3 and BH-4 with normal flow rate (6 l/min) in the powerhouse drift (downstream) *Teesta stage-IV HEP West Bengal.*

Sl. no	Fracture inclination/dip (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	Shut-in pressure P_{si} (MPa)
1	72.99	175.99	4.2
2	39.06	139.43	5.6
3	23.33	26.46	5.53
4	64.03	54.66	4.6
5	60.8	57.76	6.77

Table 7.
Pressure and fracture orientation data derived from BH-5 and BH-6 with normal flow rate (6 L/min) at Intake drift Teesta stage-IV HEP West Bengal.

Principal stresses	Intake drift	Powerhouse drift (upstream)	Powerhouse drift (downstream)
Vertical stress (σ_v) in MPa (calculated with a rock cover 160 m and density of rock = 2.7 g/cc)	4.24	5.08	6.35
Maximum horizontal principal stress (σ_H) in MPa	7.575 ± 1.47	7.28 ± 2.23	8.95 ± 0.931
Minimum horizontal principal stress (σ_h) in MPa	5.05 ± 0.9803	4.42 ± 1.04	5.97 ± 0.621
Maximum horizontal principal stress direction	N 20 degrees	N 120 degrees	N 110 degrees
$K = \sigma_H/\sigma_v$	1.78	1.35	1.40

Table 8.
Stress tensors as evaluated at various locations.

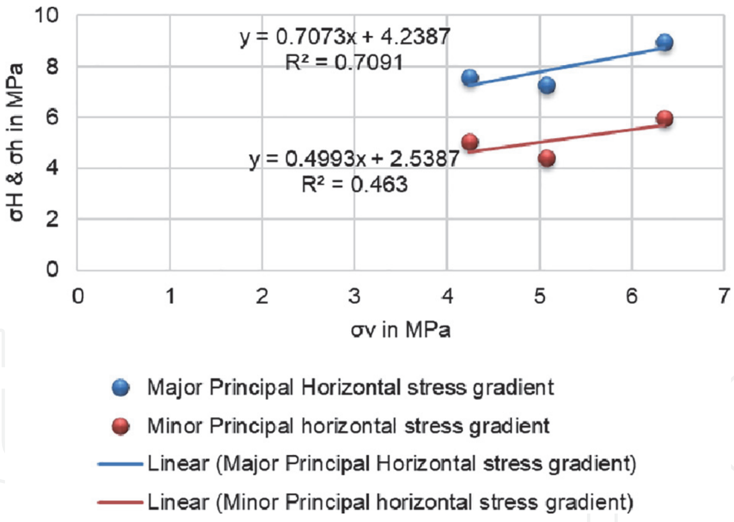

Figure 17.
Stress gradient in non-fractured rock mass.

The stress gradient is plotted to observe any induced stress due to excavation or topography of rock cover (**Figure 17**). It is clearly understood that results determined are free from any influence.

5. Comparison of results obtained from both methods

The hydraulic fracturing tests were conducted by using high flow rate technique in fractured rock mass and normal flow rate technique in good rock mass zones in the same boreholes. A total of 24 hydraulic fracturing tests were attempted in different EX size boreholes inside the tunnels of proposed powerhouse and Intake drift areas. The testing zones selected at the depths between 7 and 27 m. In most hydraulic fracturing testing, at the depth of 7–30 m, pumping rates of 4–6 l/min are sufficient to conduct the entire test. Such pumping rates were sufficient to conduct good hydraulic fracturing tests, but proved to be insufficient for tests in the fractured zones. As this problem became apparent during testing, a high-pressure pump was used in order to achieve higher pumping rates (up to 18 l/min). HTPF method was used for data interpretation and the analysis of the results was done by using PLANE software and GENSIM.

The software PLANE incorporates the impression data with the compass data as input parameters and gives the strike, dip and dip direction as the output known as fracture orientation data.

The software GENSIM computes the stress field based on measured shut in pressure and fracture orientation data. Assumption is that the vertical stress is a principal stress and is equal to the weight of the overburden. The powerful GENSIM program requires only the shut-in pressure and the orientation of an induced or preexisting fracture. As a result, the role of breakdown pressure and fracture reopening pressure are nil as far as stress computation is concerned [17].

After obtaining the results by both methods it is observed that the direction of maximum principal horizontal stress is not changed. The magnitude of maximum and minimum principal horizontal stresses is also almost same with negligible or fraction of difference. The stress gradients are observed in fractured and non-fractured rock mass. No influence found of any induced stress at any location. The results are compared in **Table 9**.

Stresses	Fractured rock mass	Non-fractured rock mass	Remarks
Maximum horizontal principal stress (σH) orientation	N 20 to N 120 degrees	N 20 to N 120 degrees	No change in orientation
Stress gradient (σ_H/σ_v)	1.19.Z + 1.2 R^2 = 0.47	0.7.Z − 4.23 R^2 = 0.7	No change in stress gradient
Stress gradient (σ_h/σ_v)	0.74.Z + 0.8 R^2 = 0.47	0.49.Z − 2.53 R^2 = 0.46	No change in stress gradient

Table 9.
Comparison of results determined in fractured and non-fractured rock mass.

6. Hydraulic fracturing in porous rocks

Hydraulic fracturing method is the accepted technique for measurement of in-situ stresses in hydroelectric projects and in metalliferous mining projects in India and abroad. But its use in coal mines is limited to a few British and Australian coal mines. This is mainly because of the occurrence of porous rocks in coal mines in India and elsewhere.

Scanty literature is available for this type of studies as this procedure of experiments is still in the budding stage. Hence literature references have not been elaborately quoted since it was not an objective of this work to critically compare aspects of our experience with those of other works. The method used in the present study is described here in its near original form in order to place on record the experience gained.

6.1 High viscous fluid technique in porous rocks

The porosity has major effects on hydraulic fracturing technique which results in fracture deviation away from the actual orientation (**Figures 18** and **19**). High viscous liquid instead of water is used for pressurization during hydraulic fracturing, but the influence by using viscous liquid on the stress is not known (**Figures 20** and **21**). The results (**Tables 10–12**) are validated with overcoring technique that is applicable in porous rocks. Over coring method does not get influenced from the presence of porosity in the rock mass [2, 7, 14, 19].

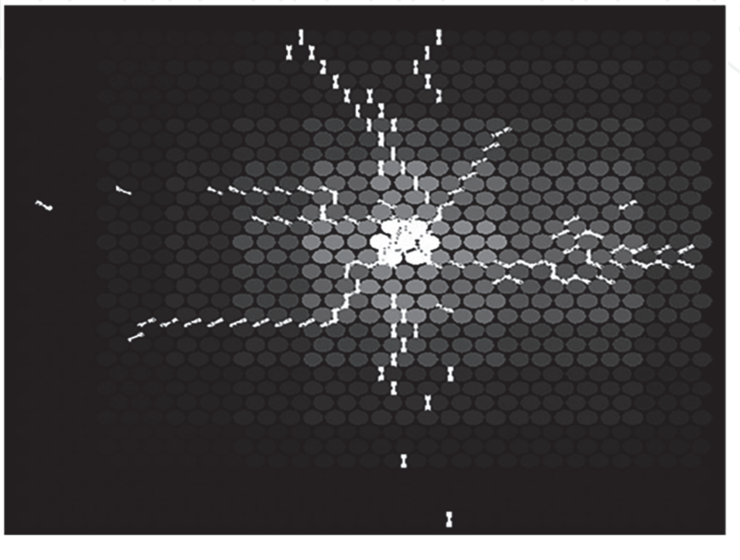

Figure 18.
Hydraulic fracturing test with normal flow rate in porous rocks.

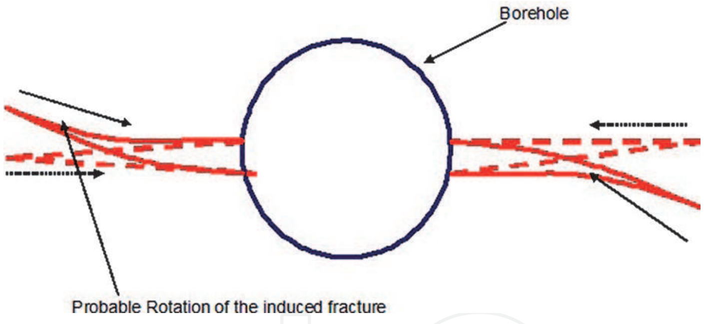

Figure 19.
Hydraulic fracturing test with high flow rate in porous rocks.

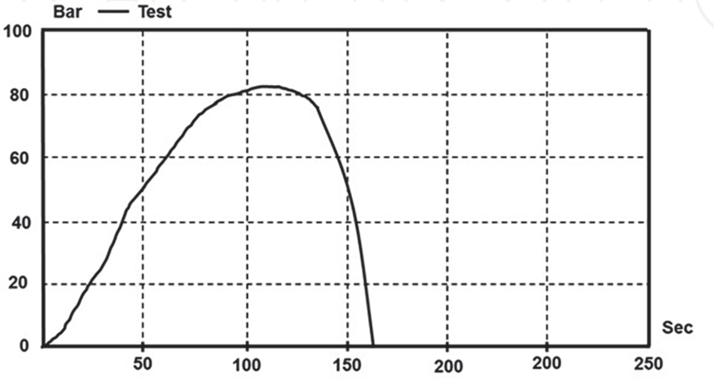

Figure 20.
Pressure drop for low viscous oils (less than 200 cP).

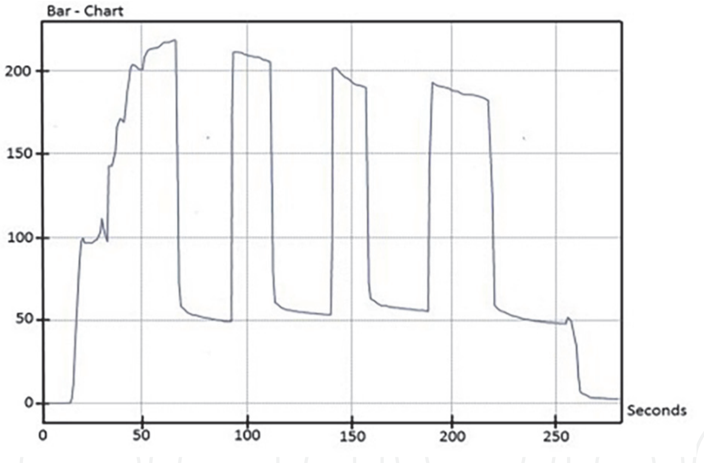

Figure 21.
Pressure-time record obtained using high viscous liquid (ISO VG 320 oil).

Sl. no.	Trace dip (degree) [0–90 degrees]	Trace orientation (degree) [North–East]	Shut-in pressure (Mpi)
1	87	165	9
2	88	160	9.3
3	58	73	8
4	84	128	10.34
5	62	25	5
6	59	84	6

Table 10.
Fracture orientation data obtained from borehole at KTK 8 incline, Singareni Collieries Company Ltd. Telangana.

Sl. No.	Fracture inclination (degrees) [90 degrees = vertical]	Fracture strike (degrees) [N over E]	P_{si} (Mpa)
1	40	25	4.5
2	50	71	8
3	60	20	4.25
4	70	05	5.015

Table 11.
Fracture orientation data obtained from Borehole at Shantikhani longwall mine, Singareni Collieries Company Ltd. Telangana (using high viscous liquid).

Principal stresses	KTK 8	Shantikhani
Vertical stress (σ_v) in Mpa (calculated with a rock cover 160 m and density of rock = 2.7 g/cc)	6.59	11.02
Maximum horizontal principal stress (σ_H) in Mpa	7.31 ± 1.91	11.25 ± 0.4815
Minimum horizontal principal stress (σ_h) in Mpa	3.65 ± 0.957	4.50 ± 0.1926
Maximum horizontal principal stress direction	N 30 degrees	N 20 degrees
$K = \sigma_H / \sigma_v$	1.11	1.02

Table 12.
Stress tensors as evaluated at various locations.

7. In-situ stress measurements by overcoring technique in porous rock mass

Overcoring measurements are common in civil and mining engineering and conducted for design of underground openings [2]. The quality of the measurement depends on quality of drilling, gluing and overcoring, and on the rock characteristics such as anisotropy, discontinuities, and heterogeneity [2, 7, 14, 16, 18, 25].

7.1 Overcoring test procedure

Drilling HX size hole— HX size (150 mm diameter) hole was drilled up to a depth of 7 m in the roof sandstone (**Figure 22**).

Core retrieval—The overcored rock is recovered from the hole using core-breaking chisel that is attached to the rods used for wedging the core off the face (**Figure 23**). An intact length of core (>500 mm) free of fractures and joints is ideal for a satisfactory overcore. The recovered core was considered satisfactory, and free of fractures and voids [16].

Drilling EX size borehole (Pilot hole)—After removal of the HX size core from the borehole, the EX-size Pilot hole (38 mm) was drilled up to 50 cm exactly at the centre of the HX size bore hole from 7 to 7.5 m (**Figure 24**). This hole was collared concentrically with the large diameter hole. To achieve this, the EX-starter barrel is screwed into a stabilizer and about 60 cm of hole drilled. The drill string is then withdrawn and the Pilot hole is drilled with EX twin tube barrel attached to a stabilizer, to a depth of about 60 cm.

When the EX-hole reached the target depth, water was circulated for an additional 10 min so that the drilling sludge and cuttings could be flushed out. The barrel and drill string are then removed and the EX core is recovered for inspection.

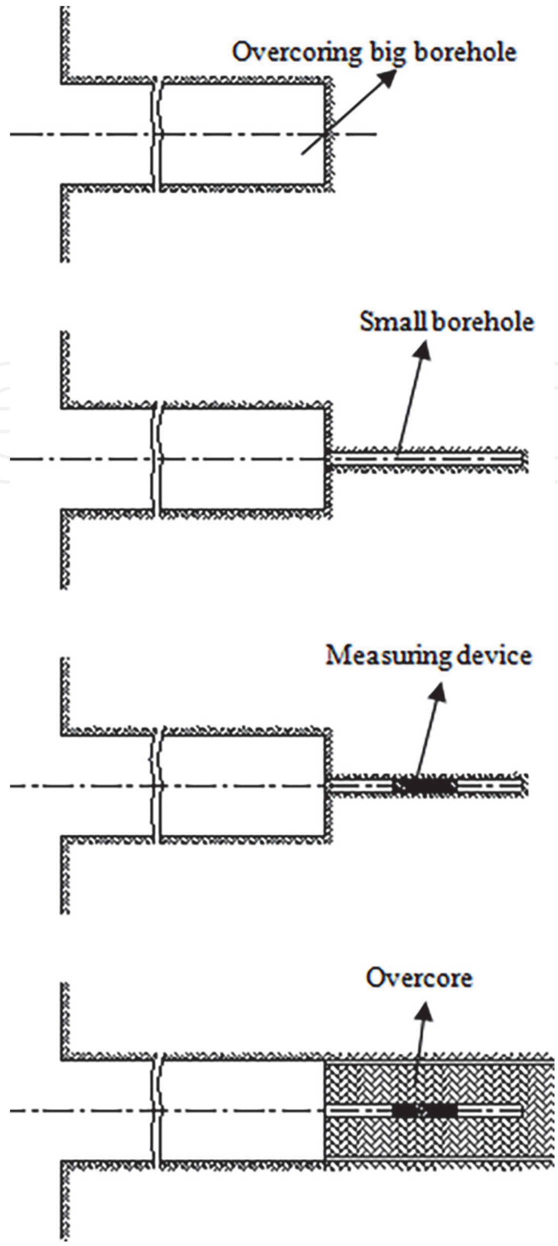

Figure 22.
Overcoming method.

Preparation of glue—It was ensured that the resin and hardener had the correct temperature specification for the expected temperature range. These two were mixed (**Figure 25**) according to the prescribed procedure. Any air pockets remaining were removed by carefully dispersing the glue with a small rod.

Selection of gauge position—The recovered E core was closely inspected to locate the best possible position for the strain gauges. The distance from the strain gauges to any likely weakness planes must be maximized. The other requirement was that the gauges should be at least one diameter away, preferably more, from the other ends of the EX-hole.

Figure 26 show a suitable location of the gauges with respect to the core length. Positioning the gauges too far from the collar can cause problems as the core may break during overcoring and damage the shank and the HI Cell as it tends to rotate in the barrel.

EX size hole measurement—The range of depths that the HI Cell can be placed is limited by the requirement to be approximately beyond two over-core diameters from end of the overcore hole and before the E hole end, minus the stub left when the core is broken out of the hole. Typically, the stub length is up to 150 mm.

Figure 23.
Retrieval of HX size core.

Figure 24.
Drilling of EX size borehole (Pilot hole).

The strain gauge position was measured and it was decided to keep it at 60 cm from the collar of the Pilot hole. The depth to the position where the strain gauges are to be glued, was recorded. The installation rods were marked with tape, which indicates the depth to the end of EX size hole (Pilot hole). The tape was made to coincide with the edge of the collar of the over-core hole.

DOI: http://dx.doi.org/10.5772/intechopen.106552

Figure 25.
Preparation of glue.

Figure 26.
Selection of gauge position and fixing the pin.

Having determined the distance of the gauges from the EX-size hole (Pilot hole) end, the piston was mounted in the shell at the glue extruded position. The piston rod was then cut to length and taped onto the end of the piston.

Piston attachment—The piston was sprayed with a silicon-based releasing agent to prevent it from bonding to the inside of the gauge shell. The piston is aligned in the Cell as indicated by the scribed lines on the piston and upper rim of the Cell (**Figure 27**). Each of the four holes was lined up, and lead shear pins were placed through the cell wall into the piston.

Installation of HI cell—The completed Cell assembly was inserted in the orienting tool, attached to the trolley (**Figure 28**). The installation tool containing the HI Cell was screwed into the installation rods and the whole assembly pushed up the hole.

Each rod and coupling connections were firmly tightened and the cables were also kept taut. The rods were pushed up the hole until the first tape mark was reached. This indicated that the tip of the piston rod was about to enter the E hole. The Cell was then pushed slowly into the EX-size hole (Pilot hole).

When the second tape mark was reached, the piston rod tip was resting against the end of the Pilot hole. Some extra force was required to break the shear pins and then the rods were pushed slowly inwards so that the glue could evenly distribute itself between the rock and the gauge surface.

In this way, the installation was completed. Once the epoxy glue had gelled and curing was reasonably advanced, the installation rods and the trolley were recovered. Overcoring was commenced after 24 h of installation.

Overcoring—The Cell cable was passed through the centre of each rod, and the rod string was held with a slight tension to ensure that it was not cut by the overcoring barrel (**Figure 29**).

Figure 27.
Piston attachment for CISRO HI cell.

Figure 28.
Installation of CISRO HI cell.

Figure 29.
Overcoring test at KTK 8 Incline.

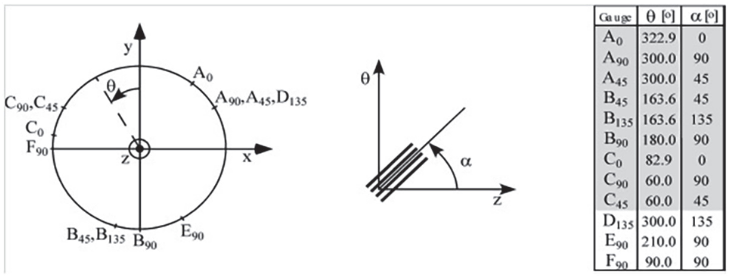

Figure 30.
CISRO HI cell-strain gauge configuration.

The cell contains three strain rosettes 120 degrees apart. The gauge configuration is as follows: two axial, three tangential and four gauges inclined at ±45 degrees in the 9-gauge cell. The 12-gauge cell has one additional 45 degrees and two additional tangential gauges (**Figure 30**). The gauges are 10 mm long and are located 0.5 mm below the outer surface of the cell.

7.2 Determination of strains

Generally, when determining the observed strains from overcoring, a stable value is preferential before overcoring starts and after overcoring stops. The difference among these values is understood to correspond to the strain relief involved in the overcoring process [12]. Generally, each HI-Cell plot shows a peak strain followed by a flat portion that decreases toward the end of the over-core (**Table 13**). The final strains for the site overcore were obtained by averaging the readings in the flat portion of the curve. The recommended overcoring speed of the

Distance (cm)	A0	A90	A45	B45	B135	B90	C0	C90	D135	D135	E90	F90
0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0
15	4	3	0	4	6	6	0	0	0	6	0	0
20	6	10	0	10	45	25	35	0	0	55	0	0
25	9	43	−60	35	89	30	55	−50	−15	120	−20	−60
30	18	140	−35	65	95	55	180	−75	−20	220	−45	−90
35	−75	190	50	95	156	220	300	125	−35	280	−100	−114
40	230	220	280	150	320	380	530	580	350	400	390	180
45	235	250	300	432	330	390	540	590	360	410	403	190
50	130	180	150	350	195	250	320	320	250	350	360	150
55	95	90	102	320	190	240	142	220	222	340	220	135
60	98	65	90	280	185	230	95	175	212	250	120	125
65	96	64	95	275	184	220	92	165	150	230	115	124
70	95	63	94	274	183	219	91	163	149	229	114	124
75	94	62	93	273	182	218	90	162	148	228	113	124
80	93	61	92	272	181	217	89	161	147	227	112	124

Table 13.
Change in strain from overcoring HI-Cell.

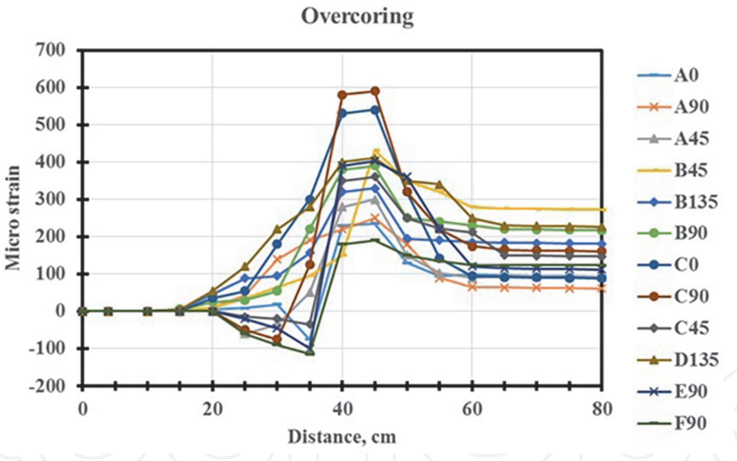

Figure 31.
Overcoring, strains for the KTK 8 incline site.

chuck is 120 rpm to minimize core breakage. **Figure 31** shows overcoring strains as a function of distance for each HI-Cell [14].

7.3 Biaxial testing

In biaxial testing of an overcore section, with no additional instrumentation besides the strain gauges already glued to the inside of the Pilot hole. Measurements can be conducted in the field, in direct unification with overcoring (**Figure 32**). Only radial (biaxial) compression loading is applied to the sample, and there are no restrictions regarding the orientation of the symmetry plane of the rock sample [16]. The biaxial test is exceptionally critical because it establishes the elastic properties of the system, including core, epoxy, and cell, that unlock in situ stress from the over coring strains (**Table 14**). Plots of strain-versus-pressure are shown in **Figure 33**. The calculated elastic properties, namely, Young's modulus and Poisson's ratio are given in **Table 15**.

7.4 Discussion on the results

The calculation of the in-situ state of stress from the measured strains obtained from overcoring measurement in sandstone is based on the theory presented by Amadei [2]. Stress 201X programme allows the user to ascertain stresses and rock properties from raw data output from a CSIRO HI cell and plot overcore (**Figure 31**) and biaxial tests (**Figure 33**). The programme is for genuine ES&S CSIRO HI cells having preset values for alpha and beta angles for the strain gauges. Stress 201X

Figure 32.
Biaxial test.

Pressure, MPa	A0	A90	A45	B45	B135	B90	C0	C90	D135	D135	E90
0	0	0	0	−10	−20	−30	−40	−50	−60	−65	−40
1	10	−10	15	8	−15	−20	−40	−50	−70	−80	−90
3	20	−20	27	15	−17	−26	−50	−80	−100	−110	−120
3.5	25	−30	32	22	−18	−30	−55	−100	−130	−130	−150
4	30	−40	34	28	−19	−40	−60	−110	−130	−140	−160
5.5	40	−50	47	38	−25	−50	−70	−150	−160	−180	−200
7	50	−60	57	48	−35	−60	−90	−200	−220	−240	−260

Table 14.
Change in strain from cyclic biaxial chamber loading to 7 MPa.

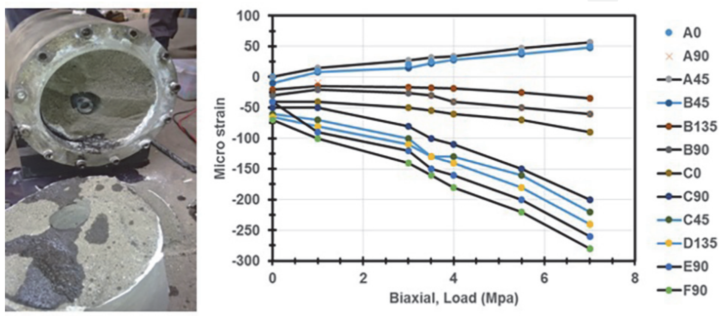

Figure 33.
Plot of micro strain versus pressure.

Sl. No.	Location	Young's modulus, GPa	Poisson's ratio
1	KTK 8	11.6	0.3

Table 15.
Elastic properties calculated from biaxial test.

code was originally developed for CSIR-type triaxial strain cell with a maximum number of 12 strain rosettes with up to 4 strain gauges per rosette. The input values for this program (**Table 16**) are from the averaged circumferential and axial strains for overcore (**Table 13**) and biaxial test results (**Table 15**). The final calculated stress values from Stress 201X program are given in **Table 17**.

Sl.no.	Parameters	Mine data
1	Location	KTK 8–21 incline
2	Hole number	KTK 1
3	Test number	HI 1
4	Hole bearing	—
5	Hole dip	—
6	Date time installed	18-11-2014, 2.00 AM
7	Date time over coring	19-11-2014, 3.00 AM
8	E collar depth	12 cm
9	E hole length	600 m
10	Strain gauge depth	7 m

Sl.no.	Parameters	Mine data
11	Rock temperature	300°C
12	Temperature offset	−0.10°C
13	Drill water temperature	250°C
14	Cell type	CISRO HI Cell
15	Cell number	8069
16	Over core diameter	144.5 mm
17	E hole diameter	38.1 mm
18	Diameter of gauges	35 mm
19	Inner diameter of cell	35 mm
20	Young's modulus of epoxy	2.6 GPa
21	Poison ration of epoxy	0.4
22	K1	1.1258
23	K2	1.2503
24	K3	1.081
25	K4	0.9505
26	Cell gauge factor	2.103
27	Read out gauge factor	2.000
28	Orientation of B90 Gauge (#6)	180
29	Core length	800 mm
30	Maximum biaxial test pressure	15 MPa
31	Rock type	Sandstone
32	Modulus, GPa	11.6
33	Poisons ratio	0.3
34	Maximum temperature change	20°C

Table 16.
Input parameters for software—Stress 201X.

Sl. no	Location	Depth, m	σ_H	σ_h	σ_v	Orientation
1	KTK 8	269	6.8	3.1	6.2	N 30 degrees

Table 17.
Stress values from the ovecoring tests.

7.5 Comparison of the different stress results

The final results of magnitude of the minor and major stress components in the horizontal plane obtained from hydraulic fracturing method show in good agreement with the corresponding stress components obtained from overcoring (**Table 18**).

Two tests were conducted in apparently uniform sandstone by using hydraulic oil as the fracturing fluid. Impression packer images revealed the induced cracks in the reopening pressure. The average horizontal orientation of the cracks obtained at this site shows reasonable agreement with the orientation of the major secondary principal stress component in the horizontal plane obtained from overcoring.

KTK 8–21 incline				
Method of test	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)	**Orientation**
Hydraulic fracturing	6.59	3.65	7.31	N 30
Over coring	6.8	3.1	6.2	N 30

Table 18.
Comparison of results from the two methods.

8. Discussion and inference

The standard long term instantaneous shut-in pressure revealed reasonable pact with the magnitude of the vertical principal stress component obtained from overcoring at the site. In all the sites tested, this was the only instance in which a viscous fracturing fluid had to be employed specifically to enable a crack to be initiated.

There was no indication of crack spin on the impression packer images.

The results indicate the effect of test fluid viscosity on the ability to reliably estimate the magnitude of minor stress component in the horizontal plane from the long term instantaneous shut-in pressure when crack initiation under a seal is suspected. While the agreement was acceptable, for practical purposes, for the tests conducted with water (especially considering the relatively severe influence of experimental errors at the absolute stress levels involved) the discrepancy in the case of the tests conducted with oil was disproportionate. It was also noticed that the relative differences between the tangent intersection and tangent divergence estimates for instantaneous shut-in pressure decreased as the viscosity of the test fluid intensified.

Re-pressurization of the test zones originally tested with oil or hydraulic oil produced instantaneous shut-in pressures and crack reopening pressures consistent with the results obtained using oil as the only test medium. Testing using a combination of fluids such as this may represent a practical means. These results have important implications in the field wherever the hydraulic fracturing stress measurements are required in fractured and porous rock mass. It is suggested to have a re-look at the long-standing view that the hydraulic fracturing method is not suitable for fractured and porous rock mass. But this study has disproved this assumption. In situ stress may vary from point to point, and method to method in a rock mass, and may have different values when measured over different volumes. Such variations are intrinsic and should not always be seen as anomalies or errors in the measurement themselves and cannot be concluded that no comparison or correlation can be drawn from different methods [18].

9. Conclusions

The work described here however represents the results of field evaluation programme, in which a very pragmatic point of view is being taken. The opportunity is taken to evaluate the results obtained from hydraulic fracturing with the results acquired from overcoring at the same site. The results acquired from overcoring are deemed to deliver a trustworthy indication of the in situ stress field.

The in-situ state of stress is measured for two principal reasons

a. To predict rock response to changed loading conditions caused by construction or excavation, including new engineering procedures that require use of the in-situ stress field as part of the design, and

b. To further understand the tectonic processes.

The hydraulic fracturing stress measurements had become a broadly used technique for determining in situ stresses at depth. It is a technique for understanding rock mass behaviour in conjunction with stability of the excavations in rock. Because of the rapidly expanding use of this method, the method is still evolving in certain rock mass conditions [1, 3–11].

Hence the key objective of this project is to develop a proper methodology for in situ stress measurement by hydraulic fracturing method in porous and fractured rock media, encountered in some of the coal mines as well as in some of the underground tunnels of hydroelectric projects in the Himalayas.

To fulfil the objective of the project, it was proposed to conduct in situ stress measurements in fractured and porous rock mass areas by two different methods at the same location. The hydraulic fracturing stress measurements were conducted by adopting both high flow rate and normal flow rate method in fractured rocks and high viscosity liquid method, and overcoring methods in porous rocks. The stress results by the two methods were correlated with already recognized or established technique as a benchmark. The results of hydraulic fracturing stress measurement methods were authenticated, so that this method can be implemented for stress measurement in porous and fractured rocks and use them widely in mining and hydroelectric projects. This will aid in producing a data bank for in situ stress, which will be highly advantageous for both mining and hydropower industries wherever the rock mass is fractured or porous and the stress measurements are indispensable for designing the support systems.

In the first part of the project, two sites were selected inside a proposed powerhouse tunnel of one of the hydroelectric projects in the Himalayas where the rock formations are fractured. Boreholes were drilled 10–30 m deep depending upon the requirement and site conditions. In situ stresses were measured inside these boreholes by hydraulic fracturing method using manipulation of flow rate. The stress evaluation was made using latest software. The stresses evaluated by this method was correlated with normal hydraulic fracturing method at the same locations where the rock mass was not fractured.

A total of 24 hydraulic fracturing tests were attempted in different EX size boreholes inside the tunnels of the proposed powerhouse and intake drift areas where the rock mass was fractured. The testing zones were selected at depths between 10 and 30 m. In normal conditions, and in good rock mass, the pumping rates of 4–6 l/min are sufficient to conduct the hydraulic fracturing test, but such pumping rates proved to be insufficient for tests in the fractured zones. As this problem became apparent during testing, a high-pressure pump was used to achieve higher pumping rates of up to 18 l/min.

It was observed that with increasing or decreasing pressure in each cycle, the pressure also declined automatically after certain increment of pressure. It is interpreted that, since the flow of water is affected by the whole fractured rock mass, the pressure changes were due to the opening of fractures at different spatial positions.

The hydraulic fracturing tests in good rock mass exposed, repeatable pumping pressures, with the same fracture. This indicates that we were creating a new hydraulic fracturing in a formation which had less tensile strength. Data was evaluated from preexisting reopened fractures, and the orientation of these fractures was analysed to understand how the instantaneous shut-in pressures during the test are related to the value of normal stress across the fracture.

The most reasonable explanation, however, is that at the fast-pumping rate the pressure gradient was so large that the tensile strength of the rock near the borehole exceeded before the shear strength of the outer part of the rock mass was reached.

After shut-off of the pump, instantaneous shut-in pressure was obtained to get the normal stress across the fracture and to calculate the minimum principal stress magnitude and direction.

Stress measurements were conducted by using high viscous liquid in porous rocks; in the same rock mass, at about 1 m away, overcoring method using CSIRO Hollow Inclusion Cell was also carried out. The stresses obtained from hydraulic fracturing method using high viscous liquid were correlated with stresses measured by overcoring method. The stress measured by the overcoring method was used as a benchmark as this method does not suffer from the presence of porosity of the rock.

The average long term instantaneous shut-in pressure showed reasonable agreement with the magnitude of the near vertical principal stress component obtained from overcoring at the site. This was the case in which a viscous fluid had to be employed specifically to enable a crack to be initiated, and the shut-in pressure used to make estimates of some stress component magnitudes.

The results indicated the effect of test fluid viscosity on estimation of the magnitude of minor horizontal stress components. It was observed that the relative differences between the tangent intersection and tangent divergence estimates for instantaneous shut-in pressure decreased as the viscosity of the test fluid increased.

10. Summary

Hydraulic fracturing method is the accepted technique for measurement of in situ stresses in hydroelectric projects and in metalliferous mining projects in India and abroad. But its use in coal mines is limited to a few British and Australian coal mines. This is mainly because of the occurrence of porous rocks in coal mines in India and elsewhere.

Despite the extensive theoretical work on the subject of hydraulic fracturing that had been carried out by the mid-1960s, it is for only restricted for fractured rocks. Extensive studies couldn't provide proper solution for conducting hydraulic stress measurements, where it is difficult to conform on the legitimacy of the results [1–15].

The stress measurements in coal mining areas are determined using overcoring methods. Though porosity of the rock mass does not have influence on the stress measured using this method, due to workable limitations, it can be used for shallow depth only. However, the need for the stress measurements at the deeper levels is essential for proper planning of layouts etc. Therefore, it has been widely accepted that hydraulic fracturing technique will be suitable for porous media also provided the practical limitations are overcome.

To fulfil the objective,

a. The hydraulic stress measurement has been conducted by adopting high flow rate method in fractured rocks.

b. The in-situ stress results have been compared with the in-situ stress results by adopting normal flow rate obtained in the same test section of good rock mass condition.

c. The results of hydraulic fracturing can be validated, and the method can be adopted for in-situ stress measurement in fractured rocks.

d. The hydraulic stress measurement has been conducted by using high viscous liquid in porous rocks.

e. The in-situ stress results have been compared with the in-situ stress results by overcoring method obtained in the same test section of porous rock mass condition.

f. The results of hydraulic fracturing can be validated, and the method can be adopted for in-situ stress measurement in porous rocks.

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