REPORT ON IN-SITU STRESS MEASUREMENTS & IN-SITU DEFORMATION MODULUS AT THE POWER HOUSE SITE AND SHEAR STRENGTH PARAMETERS AT THE DAM AND POWER HOUSE SITE OF PROPOSED RATLE H.E. PROJECTM J & K.

SUBMITTED TO:

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REPORT ON IN-SITU STRESS MEASUREMENTS & IN-SITU DEFORMATION MODULUS AT THE POWER HOUSE SITE AND SHEAR STRENGTH PARAMETERS AT THE DAM AND POWER HOUSE SITES OF PROPOSED RATLE H.E. PROJECT, J &K.

1.0 INTRODUCTION

GVK Ratle Hydro Electric Project Pvt. Ltd. has been awarded 780MW (4x195MW) Ratle Hydro Electric Project in Kishtwar district of Jammu & Kashmir State for implementation on BOOT basis for 35 years. To obtain parameters for the purpose of design of proposed structures of the project, M/s GVK Ratle Hydro Electric Pvt. Ltd awarded the work of carrying out rock mechanics tests for the project covering field tests at the dam site and at the power house site and also collection of rock samples from Kuligad quarry area for testing in the laboratory for their suitability for use as coarse aggregate in concrete to M/s AECS Engineering and Geotechnical Services Pvt. Ltd., C-48, Sector-10, Noida, U.P. vide work order No.GVK/RHEPPL/GEO/1 dated 16th November, 2010 followed by Amendment No. 1 dated 22.03.2011 covering geotechnical tests at a bridge site.

This report covers hydro fracturing tests conducted in the drift for measurement of in-situ stresses, Plate Jacking Tests for determination of deformation modulus of rock mass at the power house site and determination of shear strength parameters of rock mass for rock to rock & concrete to rock interfaces by direct shear tests at the dam site and rock to rock interface at power house site of the project, analysis of test results & the recommended parameters for design.

2.0 THE PROJECT

Ratle HE Project is located in Kishtwar District of Jammu & Kashmir and envisages utilization of water of Chenab River harnessing a head of approximately 97m. The Project lies downstream of Dulhasti Hydroelectric project (390MW) and upstream of Baglihar Hydroelectric Project-Stage I (450MW) both of which are in operational stage on the same river. The proposed site for the construction of dam is located near village Drabshala in Distt. Kishtwar. Underground powerhouse is proposed on the right bank of the river near village Drabshala. The water from the reservoir would be taken to powerhouse through pressure shafts/ penstocks and discharged through tail race tunnels (TRT) back into the river.

2.1 Salient Features of the Project

The proposed Ratle H.E. Project in Jammu & Kashmir comprises of the following structures:

- Concrete gravity dam, 133 m high from deepest foundation level across the river Chenab. Length of the dam at top is 189.5m.
- 2 Nos. 11.0m diameter circular shaped diversion tunnels (DT1 & DT2 with lengths of 555m & 460m respectively) for total diversion discharge of 2500m^3/sec (max. observed flood for non-monsoon period).
- Rockfill (with central clay core) coffer dams on u/s and d/s of dam axis.
- Orifice type +Crest Gate Spillway with spillway capacities at FRL of 14700 cumec & 1470 cumec respectively.
- 4 Nos. Quick Acting Gates with sizes of 5.3mx6.6m as Intake structure with trash rack of 6.2mx23.24m size.
- 1 No. restricted orifice type D/S surge chamber having size of 114m x 22 m x 45.4 m.
- 4 Nos. underground circular steel lined penstocks of 6.6 m diameter with lengths of 211 m, 197 m, 184 m & 172 m respectively.
- An underground power house cavern of 24.5m (W) x 48m (H) x 168m (L) size to accommodate 4 Nos. Francis turbines, with installed capacity of 195 MW per unit with total installed capacity of 780MW.
- 4 Nos. 8.6 m diameter circular shaped concrete lined tail race tunnels having lengths of 394m, 369m, 349m & 330m.

2.2 Location & Accessibility of the Project Site

The project site is accessible from Jammu, the nearest town. Project is around 215 km from Jammu .The nearest operational airport is at Jammu and nearest Railhead is Udhampur about 155 km. from project site. The proposed dam site is located at 33^0 10’ 43”N and 75^0 48’ 26”E. The left bank of dam site is connected with a foot track from Jammu-Kishtwar motor road near Drabshala. The motor road is along the left bank of the river. Approximate distances between project components and nearby places are as follows: -

Distance between Delhi to Jammu : 600 kms

Distance from Jammu to Udhampur : 60 kms

Distance from Udhampur to Project Site: 155 kms

3.0 PHYSIOGRAPHY & GEOLOGICAL FEATURES OF THE PROJECT AREA

3.1 Physiography of the Project Area

The area forming the southerly slopes of NW-SE trending Pir Panjal Range constitutes transverse deeply dissected, medium drainage textured, sharp crested ridges with extremely rugged mountainous terrain characterized by high snow covered peaks and deep ravines. The Panjal range forms the water shed of Chenab and Kal Nal Rivers. Chenab is the major drainage in the area. It flows in N-S direction and then in westerly direction after taking a sharp band near Thatri. Chenab River is joined by the westerly flowing streams, Kagune Gad, Gahan Nadi, Kuli Nadi and southerly flowing Marsudar Nadi. There are a number of peaks having altitudes exceeding 4,000m which are snow-clad throughout the year. The river Chenab in general flows in North West direction with minor loops in between up to Bhandarkot (confluence of Chenab and Marusudar rivers at EL. 1100M). However, the river takes a sharp bend from this location and follows almost a southerly course upto Thatri (El 915M). The river again takes a sharp turn at Thatri and flows in westerly direction upto the confluence of Bichlari nala (El. 640M). Both these major kinks of river at Bhandarkot & Thatri are 24 & 4 km upstream and downstream respectively of Drabshala (Ratle Project location). The river during its course is joined by many tributaries; most of the important tributaries are on the right bank and are snow / glacial fed. Out of the total catchment area (22,662 km^2), almost 10,000 km^2 lies above snow line.

3.2 Lithostratigraphy

The Ratle hydro-electric project area comes under Salkhala Formation within the Kishtwar Group of rocks and is located southwest of MCT and geologically this site is represented by gneissose granite and quartzitic-phyllite sequence. Kishtwar area has been classified into older Kishtwar Goup and Younger Sinthan Group. Contact between these two Formations is marked by Chhattru Thrust. Salkhala Formation is the oldest Formation in this area. The Salkhala Formation is delimited by the Vaikrita Group of the Central Crystalline Zone below and Batal Formation of Haimanta Group in the Lahaul-Spiti-Kinnaur area of the Himachal Pradesh of Proterozoic age. The Salkhala Group comprises of laminated chloritic slate with metagreywacke and intercalated carbonaceous slate and quartzite, greyish green phyllite and schists, garnetiferous biotite schist, micaceous quartzites, psammitic gneiss and interbedded with sporadic crystalline limestone. The important feature of the Salkhala Group is the presence of a linear belt of concordant granitoid plutons. The region in the north of the project area is dominated by Higher Himalayan Precambrian high-grade metamorphic rocks and plutonic rocks. The site is closely located to southwest of MCT and west of prominent N-S trending Kishtwar Fault.

3.3 Seismicity

The Himalaya is the product of the collision of the Indian plate with the Eurasian plate, where Indian plate is underthrusting beneath the Eurasian plate. The collision tectonics resulted in progressive progradation of thrust sheets like the Main Central Thrust (MCT), Main Boundary Thrust (MBT) and the Himalayan Frontal Thrust (HFT). The contemporary deformation styles and the seismicity in the Himalaya are related to the continued collision tectonics resulting in strain build up along discrete tectonic surfaces, the transverse features causing segmented blocks.

The Himalayan Seismic Belt generated several large and great earthquakes, some of which are:

- The 1905 Kangra earthquake (M~8.0)
- The 1934 Bihar earthquake (M~8.4)
- The 1897 Shillong earthquake (M~8.7)
- The strike slip motion of the great Assam earthquake (M 8.7)

The Ratle project, located in the Kishtwar district of J&K state, essentially lies in the central crystalline sequences of the Higher Himalaya. This area has been subjected to intense tectonic deformation and has experienced seismic activity. The project area falls under seismic zone – IV of seismic zoning map of India (IS: 1893-2002).

4.0 SCOPE OF WORK

The scope of work awarded to M/s AECS Engineering & Geotechnical Services Pvt. Ltd. as per work order comprises of the following:

- Mobilisation and arrangement of Equipment, Personnel, Power Supply, Explosives-if required at the dam and power house sites.

- Sample collection from Kuligad quarry area, marking, packing & transportation to AECS laboratory in Noida (U.P.) for conducting various laboratory tests for assessing their suitability for use as coarse aggregate in concrete.
- In-situ tests at the Power house site as below:
  - Plate Load Tests in vertical & horizontal directions including drilling of 2 Nos. 6m deep NX size holes per test.
  - Shear Test-Rock to Rock Interface (1 set of 5 blocks)
  - Hydro Fracturing Test for measurement of in-situ stresses including drilling 3 Nos. 30m deep EX size holes in 3 mutually perpendicular directions for the test.
- In-situ tests at the dam site as below:
  - Shear Test-Rock to Rock (2 sets of 5 blocks each)
  - Shear Test- Concrete to Rock (2 sets of 5 blocks each)
  - Deformability test using 80 Bar capacity Dilatometer, 6 sets in each borehole of 20m depth each (2 boreholes) including drilling of 2 NX size holes to a depth of 20m each.
- In-situ tests at the bridge site covering:
  - Drilling of NX size boreholes at 4 locations to a depth of 30m each.
  - Conducting permeability tests using single packer at every 3 meter interval.
  - Conducting SPT at every 1.5m interval in overburden.
  - Conducting UCS in the laboratory on rock core samples.

As indicated in Paragraph 1.0, this report covers the Hydraulic Fracturing Tests conducted at different depths in three mutually orthogonal EX size drill holes in drift of power house area, Plate Jacking Tests in vertical & horizontal directions at two locations in the drift of power house and Direct Shear Tests on 1 set of 5 blocks each in the left bank drift and right bank drift for both rock to rock and concrete to rock interfaces at dam site and one set of 5 blocks for rock to rock interface in the drift of power house site. The report also includes the methodology followed, discussion of the test results and the recommended parameters for design purpose.

Report on determination of deformation modulus of rock mass using pressure meter tests has already been submitted. As regards report on other items of scope of work including laboratory investigations, the same shall be submitted on completion of field work.

Locations of various tests in dam drift have been marked in Drawing Nos. 1(A) & 1 (B) while Drawing No. 2 presents location of various tests in the power house drift.

5.0 HYDRAULIC FRACTURING TEST

5.1 Field Investigations

Field investigations for conducting Hydraulic Fracturing Tests in the Power House drift involved:

1. Drilling of 3 Nos. of EX size holes in the power house area drift and examining the borelogs and cores for deciding/selecting test locations in each bore hole. One No. EX size bore hole 30m deep each at RDs 201.00m (vertical), 202.00m (horizontal, U/S) and 202.00m (horizontal, D/S) was drilled in the Power House area drift.
2. Identifying and preserving cores, including geological logging of boreholes.
3. Performing Hydraulic Fracturing Tests at three different test sections in each of the 3 boreholes.
4. Recording pressure versus time for plotting Pressure-time plots to observe fracture initiation, shut in pressure and fracture reopening pressure in each of the tests performed.

Fig. 1 shows the location plan of the Hydraulic Fracturing Tests in the power house area drift whereas Fig. 2 presents the schematic view depicting locations of boreholes in which HF tests were carried out.

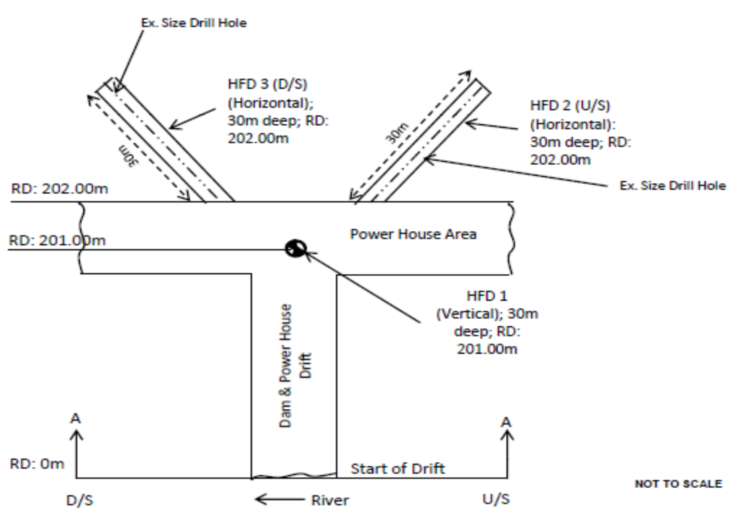


Fig. 1: Schematic Showing Details of Borehole Nos. HFD-1 (Vertical), RD 201.00m; HFD-2 (U/S) (Horizontal) & HFD-3 (D/S) (Horizontal) at RD 202.00m; Power House Area

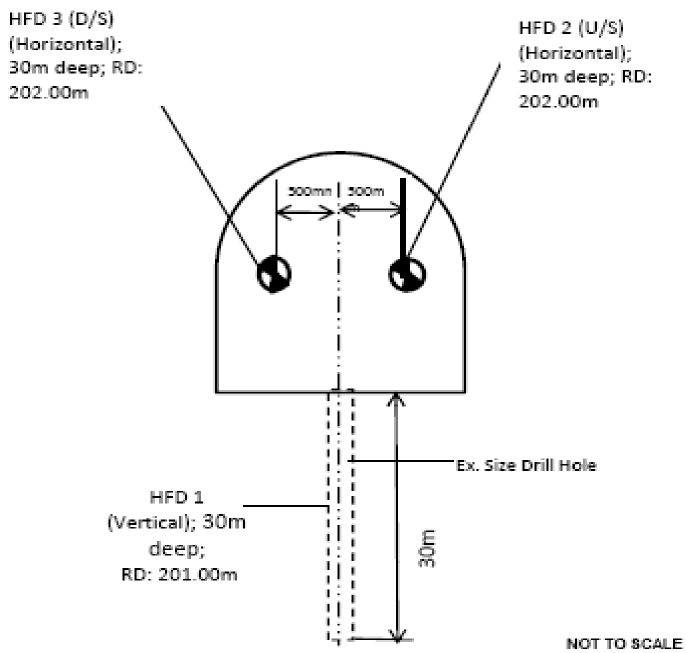


Fig. 2: Schematic View AA-Hydraulic Fracturing Test Locations

5.2 Need for Assessment of In-Situ Stresses

Vertical stress in a rock mass varies in a more predictable fashion than horizontal stress because the former is primarily affected by the weight of overburden. When an opening is introduced in the rock mass, the natural state of stress is disturbed locally as the rock mass attains a new state of equilibrium. The stress around an opening resulting from various man-made activities is termed 'induced stress' as opposed to virgin stress or absolute stress which describes the original, undisturbed state of stress. The natural state of stress is often termed as in-situ stress.

Underground in-situ stress is sometimes sufficiently high (relative to the rock mass strength) to cause rock bursting, spalling, buckling, heaving, or other ground control problems. In such cases, knowledge of the state of in-situ stress is of critical importance to the design and construction of engineering structures in a rock mass. Even in cases where the effects of stress are less dramatic, the optimum shape, orientation and layout of underground structures, as well as the effectiveness and ultimate cost of rock support systems, can be significantly influenced by the in-situ stress.

Factors affecting the magnitudes and orientation of in-situ stresses include the weight of overlying materials, geologic structures (on local and regional scales), tectonic forces within the earth's crust, residual stress and the thermal stress. The complexity of the relations between these factors and the in-situ stress usually prohibits reliable estimation of rock stress.

In-situ stresses in a rock mass control the distribution and magnitude of stresses around underground openings such as tunnels, shafts, caverns etc. and play an important role in the design of and in deciding the shape, location, orientation, method of excavation and type and extent of support for underground excavations. For instance, a circular excavation is better suited in a hydrostatic stress field than in any other stress field. One criterion used for positioning unlined pressure tunnels and shafts in valley sides is that nowhere along the opening alignment should the internal water pressure exceed the minimum in-situ principal stress in the surrounding rock mass (Selmer-Olson, 1974). Large enough in-situ stresses can help in reducing the need for liners in pressure tunnels and shafts, thus creating substantial savings. Also, when choosing the orientation for a cavern, one hopes to avoid aligning the long dimension perpendicular to the greatest principal stress. If the initial stresses are very high, the shape will have to be selected largely to minimize stress concentrations. Knowledge of rock stresses also aids in layout of complex underground works. An underground powerhouse, for example, consists of a three-dimensional array of openings including a machine hall, a transformer gallery, low voltage lead shafts, pressure tunnels, surge shafts, rock traps, access tunnels, ventilation tunnels, muck hauling tunnels, penstocks, draft tubes and other openings. Cracks that initiate at one opening must not run into another. Knowledge of the direction of the stresses permits choosing a layout to reduce the risk of cracking.

Many techniques are in vogue for determining the state of stress in a rock mass. These include flat jack test, hydraulic fracturing, USBM-type drill hole deformation gauge (Overcoring method) and CSIR or CSIR-type cell with 9 or 12 strain gauges. Each of the above techniques has its own advantages and limitations. Hydro Fracturing, however, is the most commonly employed technique that is suitable for very deep boreholes with depths greater than 50m, can work in almost any rock formation and large diameter holes and also has the advantage of determining stresses averaged over a few square meters and not just at a point or over grain size area as in case of overcoring technique.

5.3 Hydraulic Fracturing Technique

5.3.1 Brief Description of Test

Hydraulic Fracturing (HF) Test is one of the most commonly employed field tests, that is performed for determination of state of stress in a borehole. It can be performed at any depth (more suitable in deep holes for which other techniques are not feasible) in boreholes of different diameters and in almost any rock formation. The test gives an estimate of both magnitude and direction of in-situ stress in a plane perpendicular to the axis of borehole.

In a Hydraulic Fracturing Test, a section of borehole is sealed off by two inflatable rubber packers that are pressurized to stick to borehole walls and that prevent the borehole fluid from escaping up or down the borehole. Hydraulic fluid (normally water) is pumped in the sealed off portion under a constant flow rate and gradually the fluid pressure is increased thereby increasing the hoop stress on the borehole wall. As the pressure is continuously increased, the hoop stress will become tensile and when the tensile strength of rock is exceeded, a fracture will be formed in the borehole wall. The fracture thus created propogates perpendicular to the least principal stress. As soon as the fracture is initiated, breakdown pressure is reached. Pumping is then stopped without venting the hydraulic line; the pressure will suddenly drop and settle at a lower level called shut in pressure. The test interval is pressurized very rapidly in the beginning of the first cycle so as to obtain a break-down or fracture initiation pressure as quickly as possible. Repeated cycling (normally 2-3 cycles are done) of the pressurisation procedure as above shall give secondary break down pressures or reopening pressures (as venting the pressure closes the created fracture and subsequent pressurisation reopens the fracture) and additional values of shut in pressures. Values of fracture initiation or breakdown pressure, shut-in pressure and fracture reopening pressure are noted for computation of magnitude of in-situ stresses that are in a plane normal to the borehole axis. Continuous record of pressure time as well as flow rate-time is kept for the entire duration of HF test in a section. Fig. 3 shows a typical set up for a hydraulic pressure test whereas Fig.4 presents a typical pressure time plot wherein fracture initiation pressure, instantaneous shut in pressure and fracture reopening pressure are depicted.

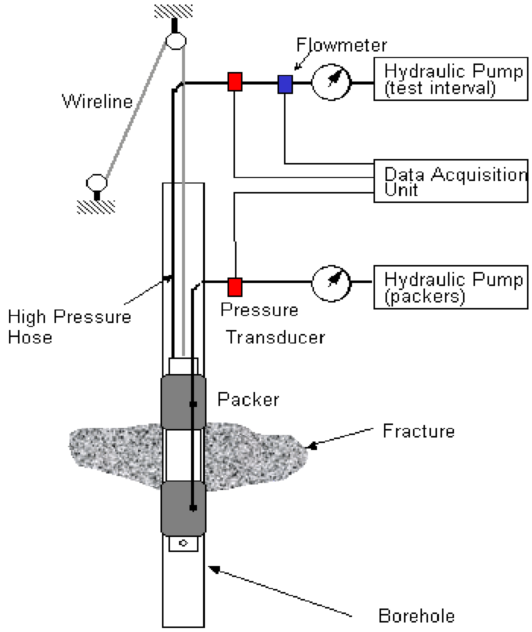


Fig. 3: A Typical Hydraulic Fracturing System Schematic (Haimson and Lee, 1984)

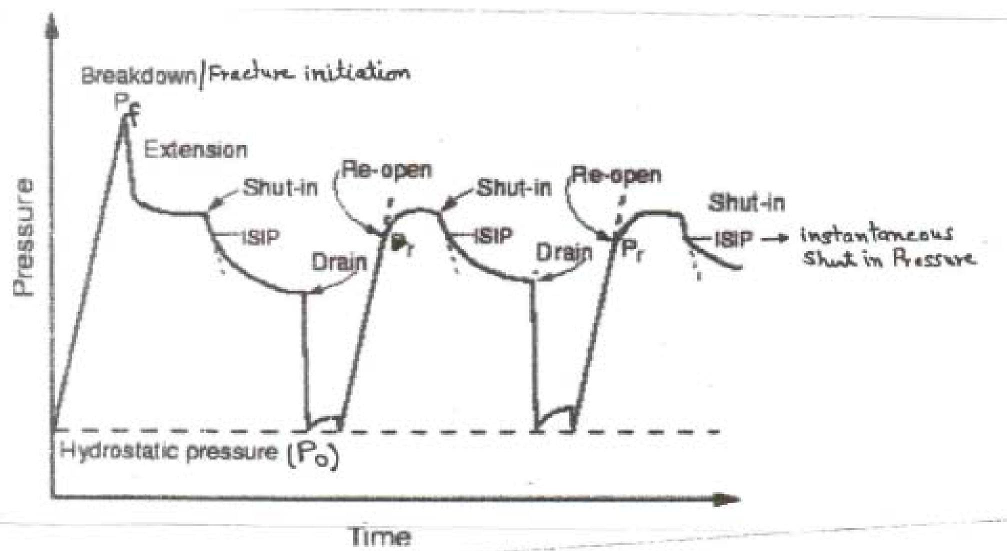


Fig. 4: Pressure-Time plot of typical hydraulic fracturing experiment showing three pressurisation cycles. During shut in phase, the instantaneous shut-in pressure is observed, which is identical to the least secondary principal stress. Modified after Brudy (1995)

The direction of the hydro fracture is obtained by use of oriented impression packer. After the Hydraulic Fracturing Test in a section is complete, the impression packer assembly is lowered precisely to the created fracture test section and pressurised so that the soft rubber covering of the packer impregnates into the fracture and a good imprint of the fracture orientation is obtained. The direction of the fracture is reported with reference to the top of drill hole/North.

5.3.2 Interpretation of Stresses

The Hydraulic Fracturing Test is typically conducted in vertical boreholes and assumes that:

- Rock is linearly elastic, homogeneous and isotropic.
- Borehole axis is parallel to one of the in-situ principal stresses and is contained in the induced fracture plane.
- Fracture attitude persists away from the hole.

The calculations for determining in-situ secondary principal stresses for a vertical/nearly vertical hole (+/- 15^0) wherein vertical stress component acts along a principal direction are performed as under:

Minimum horizontal principal stress magnitude and direction

$$\sigma_h = P_s \qquad \text{(i)}$$

where σ_h is the minimum horizontal stress and Ps is the shut in pressure obtained from the pressure-time plot of HF test and is the pressure needed to equilibrate the fracture-normal stress, which in this case is σ_h.

The direction of σ_h is obtained directly from the azimuth of the HF.

σ_h direction = direction of normal to vertical hydraulic fracture

Maximum horizontal principal stress magnitude and direction

$$\sigma_H = T + 3\,\sigma_h - P_f \qquad \text{(ii)}$$

where

σ_H = Maximum horizontal principal stress;
T = Tensile strength of the tested rock; &
P_f = Fracture initiation pressure or break down pressure

In case of any existing pore water pressure P_0, the above equation becomes:

$$\sigma_H - P_0 = T + 3\,(\sigma_h - P_0)\; - \;(P_f - P_0) \qquad \text{(iii)}$$

The direction of the maximum horizontal stress is perpendicular to σ_h direction.
In case of absence of availability of values of T from laboratory tests, the calculation for σ_H is made using the following equation:

$$\sigma_H - P_0 = 3\,(\sigma_h - P_0)\; - \;(P_r - P_0) \qquad \text{(iv)}$$

where

P_r is the fracture reopening pressure obtained from pressure-time plot of the HF test.

Normally, in case of in-situ rock $P_0 = 0$.
As such, the above equation (iv) reduces to
$\sigma_H = 3\,\sigma_h - P_r$ (v)

Vertical Stress

The vertical stress is estimated from the overburden by:

$$\sigma_v = \sum_{i=1}^{n} \rho_i g D_i \qquad \text{(vi)}$$

Where
ρ_i is the mean density of rock layer I; g is the local gravitational acceleration; D_i is the thickness of the layer i; and n is the number of rock layers overlying the test zone.

5.4 Equipment Used

The equipment used for conducting Hydraulic Fracturing Test comprises of the following main components as shown in Figs. 5 (a) & (b):

- Water Pumps: To supply water under pressure for creating rock fracture or break down pressure.
- Packer Elements: Used to seal off the test section.
- Pressure Transducers: To monitor pressure of fracturing fluid (water) in the test section with respect to time.
- Digital Pressure Recorder
- Borehole impression Packer: To delineate the fracture and to obtain the orientation of the fracture.

Fig. 5 (a): Hydraulic Fracturing Equipment Used

Fig. 5 (b): A Close up of Monitoring System of Hydraulic Fracturing Equipment

5.5 Test Procedure

Three EX size boreholes were drilled in the powerhouse drift (RD 201.00m, RD 202.00m & RD 202.00m) for conducting hydraulic fracturing tests. The drilled boreholes were logged and the cores were preserved properly in core boxes. The locations for hydraulic fracturing (HF) tests were decided based on inspection of cores and borehole logs. The details of the holes drilled are as under:

<u>Bore Hole No. HFD-1(Vertical Hole) at RD 201.00m at the bottom of the drift vertically down (VD), 30m deep</u>

Borehole HFD-1 was drilled at the bottom of the power house drift at RD 201.00m in a vertical direction. 3 Nos. HF tests were performed at three test depths of 24.60m (HF-1), 22.60m (HF-2) & 21.40m (HF-3). The test section depths at these were 24.20m to 25.00m, 22.20m to 23.00m & 21.00m to 21.80m respectively. The test section is the portion of the borehole which includes the zone that is pressurized and also the two inflatable packers.

<u>Bore Hole No. HFD-2 (Horizontal Hole-U/S) at RD 202.00m and on the face of the drift, 30m deep</u>

Borehole HFD-2 was drilled at the right side of the power house drift at RD 202.00m in a horizontal direction on U/S of the HFD-1 location and was at an angle of 45^0 to the drift axis. 3 Nos. HF tests were performed at three test depths of 29.40m (HF-4), 28.40m (HF-5) & 26.40m (HF-6). The test section depths at these were 29.00m to 29.80m, 28.00m to 28.80m & 26.00m to 26.80m respectively. The test section is the portion of the borehole that includes the zone which is pressurized and also the two inflatable packers.

<u>Bore Hole No. HFD-3 (Horizontal Hole-D/S) at RD 202.00m and on the face of the drift, 30m deep</u>

Borehole HFD-3 was drilled at the right side of the power house drift at RD 202.00m in a horizontal direction on D/S of the HFD-1 location and was at right angle to the axis of HFD-2. 3 Nos. HF tests were performed at three test depths of 29.40m (HF-7), 27.40m (HF-8) & 25.40m (HF-9). The test section depths at these were 29.00m to 29.80m, 27.00m to 27.80m & 25.00m to 25.80m respectively. The test section is the portion of the borehole that includes the zone which is pressurized and also the two inflatable packers.

Figs. 6 to 10 show photographs of Hydraulic Fracturing Test conducted at the project site and view of core box containing cores of EX size holes drilled at one of the test locations (Borehole No. HFD-3 (21.50m to 30.00m)).

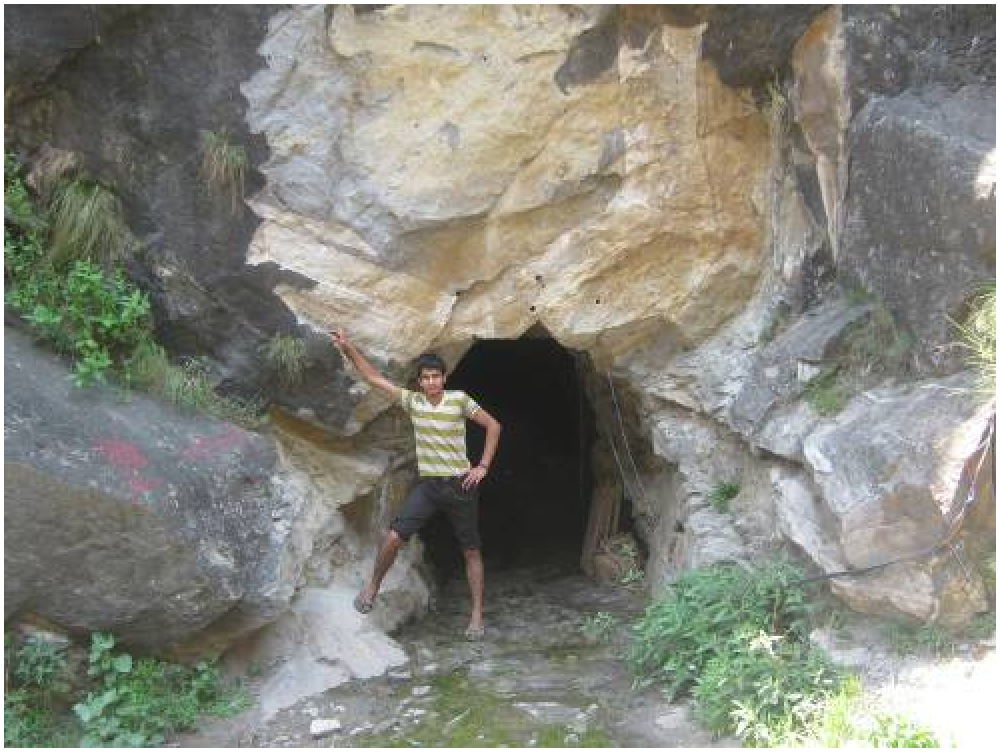


Fig. 6: A View of the Drift for Hydraulic Fracturing Test

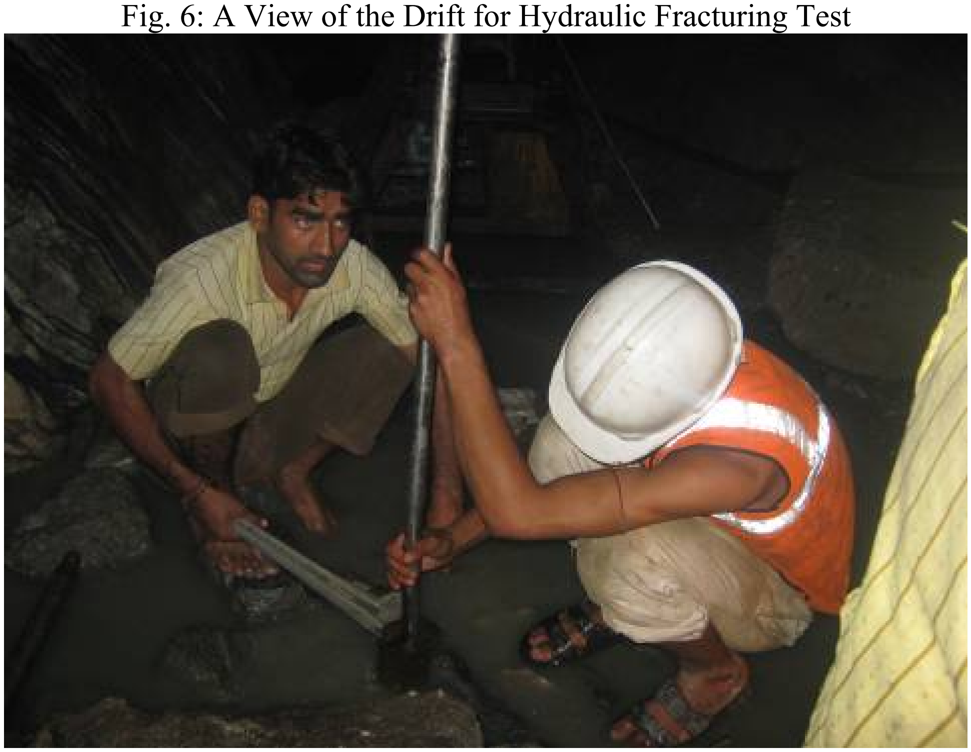


Fig. 7: Rods Being Lowered to Test Depth in Vertical Hole for Hydraulic Fracturing Test

Fig. 8: Preparatory Work for Hydraulic Fracturing Test in Horizontal Bore Hole

Fig. 9: Hydraulic Fracturing Test-Examining and Tracing Fracture Impression

Fig. 10: A View of the Core Box of Borehole No. HFD-3 (21.50m to 30.00m)

The test interval was selected based on inspection of the rock cores obtained from drilling. The straddle packer assembly (unit with two inflatable rubber packers connected mechanically as well as hydraulically and spaced 24cm apart) was lowered to the pre-decided depth of testing in such a manner that the centre of straddle packer assembly coincided with the test depth. Hydraulic pressure was then applied to inflate the packers on to the walls of the borehole. Water was used as pressurising fluid. High pressure hydraulic pumps were used to inflate the packers through tubing. Typically a pressure of 2.0MPa was used to anchor the packers to the borehole wall. The chosen test section was then pressurized. It was ensured that packer inflating pressure was more than the test section pressure. The pressure was employed at a fast rate so as to initiate fracture. As soon as the rock hydro fractures, fracture initiation or breakdown pressure P_f is achieved. The pumping was stopped at this point but hydraulic line was not vented. The pressure drops at this level and equilibrates at a lower level called shut in pressure P_s. The pressure in the line was released and another cycle of pressurisation was started. The breakdown pressure in this cycle is referred to as fracture reopening pressure (P_r) i.e. pressure required to reopen the fracture created during cycle 1 and an additional value of shut in pressure is obtained. It is presumed that after releasing pressure at the end of cycle1 the fracture closes. The same procedure of pressurisation was repeated for third cycle and another set of values of P_r and P_s were obtained. During the entire duration of the test covering 3 cycles of pressurisation process, record of pressure with respect to time was noted (Figs. 11 to 19). The data of 9 Hydraulic Fracturing Tests performed in each test section of boreholes No. HFD-1 to HFD-3 are presented Tables No. 1 to 9.

The direction of the fracture was obtained using an inflatable impression packer with an outer layer of soft rubber. The impression packer was lowered to exactly the same depth as test section and was inflated resulting in an impression on the packer of the borehole wall and any fracture traversing the wall. The orientation of the packer was noted with reference to N direction.

At the conclusion of the test, the packer pressure was totally released thus deflating the packers and the whole assembly was moved to the next test location to record another set of readings and orientation of the fracture through oriented impression packer. The orientation of each fracture in terms of dip amount and dip direction are indicated in Tables No. 1 to 9 and actual impressions of some of the fractures are presented in Figures No. 20 to 25.

5.6 Test Results

Results of Hydraulic Fracturing Tests (calculated values of maximum horizontal secondary principal stress and minimum secondary principal stress) conducted in the Power House drift in 3 boreholes and at three test sections in each borehole are presented in Tables No. 10 to 12. Pressure-Time plots for each of the nine HF tests conducted in the three boreholes in the Power House drift at RD 201.00m, 202.00m & 202.00m are presented in Figs. 11 to 19. The orientation of the maximum horizontal secondary principal stress is also indicated in Tables No. 10 to 12. The direction of minimum secondary principal stress is orthogonal to that of σ_H. . Actual impression packer test records showing the HF trace on the bore hole walls for some tests are shown in Figures No. 20 to 25.

5.7 Calculation of Stresses

5.7.1 Vertical Stress

The vertical stress σ_v is the stress caused by the local stratigraphy overlying the test location. In this case, the overburden is rock to the order of 300m and taking average γ of 2.75 T/m^3,

$\sigma_v = \gamma$ x H, where H is the height of overburden column over the test section and γ is the density of rock material.

Therefore, the estimated vertical stress at the test locations in Power House drift is:

σ_v = 300 x 2.75/100 MPa

σ_v = 8.25 MPa

5.7.2 Maximum and Minimum Secondary Principal Stresses

Computed values of σ_H and σ_h for each of the test location for the initial pressurisation cycle and subsequent cycles are indicated in Tables No. 10 to 12 for all the 6 tests. Typical calculations for vertical borehole No. HFD-1 (Vertical Hole), Test No. HF-3 for maximum and minimum stresses are shown below:

B.H. No. HFD-1 (Vertical Hole); Test No. HF-3, Test Depth: 21.40m, Test Section: 21.00m-21.80m; Observed values of different parameters from Pressure Vs Time plot for 2nd cycle are: (Refer Table 11, Sl. No. 3 and Borehole No. HFD-1)

P_f = 13.24 MPa
P_r = 11.74 MPa

P_s = 9.81 MPa
P_0 = 0 (As there is no hydrostatic Pressure)
$\sigma_h = P_s$ = 9.81 MPa
$\sigma_H - P_0 = 3 (\sigma_h - P_0) - (P_r - P_0)$
or $\sigma_H = 3\sigma_h - P_r$, considering $P_0 = 0$
or σ_H =3x9.80 – 11.74 = 17.69 MPa
σ_H = 17.69 MPa

In case of 1st fracture initiation cycle, σ_H is calculated taking into consideration fracture initiation or breakdown pressure and rock rupture strength T. The tensile strength of rock has been assumed as 7.50MPa based on the actual UCS laboratory test values of the rock type encountered at the power house test location adopted from an earlier laboratory test report of Ratle H.E. Project, J&K. The calculated values of maximum and minimum stresses and other parameters for the initial pressurization cycle are listed in Table No. 10.

The magnitude and directions of the maximum and minimum secondary principal stresses for all the tests for 2^{nd} and 3^{rd} cycles are calculated and presented in Tables No. 11 & 12.

5.8 Discussion of Test Results

5.8.1 Initial Pressurisation Cycle

Based on the data obtained for the initial pressurization cycle in all the three boreholes, it is observed that maximum secondary principal stress varies from 19.80MPa to 32.28MPa with average value of 25.58MPa whereas minimum secondary principal stress varies from 6.80MPa to 13.18MPa with average value of 10.10MPa. In case of 3 Nos. tests in borehole No.HFD-1 viz. vertically downward borehole, σ_H values are found to vary from 25.98MPa to 32.28MPa with average value of 28.66MPa whereas σ_h is observed to vary from 10.21MPa to 13.18MPa with average value of 11.51MPa. In respect of 6 Nos. tests in 2 Nos. horizontal boreholes HFD-2 & HFD-3, maximum principal stress varies from 19.80MPa to 28.40MPa with average value of 24.04MPa and minimum principal stress is found to vary between 6.80MPa to 11.20MPa with average value of 9.39MPa. The orientations of σ_H (dip and dip direction) for 3 Nos. tests in vertical borehole No. HFD-1 are found out to be 52^0 & N 40 W, 54^0 & N 30 W and 67^0 & S 40 E respectively. The corresponding values for horizontal borehole Nos. HFD-2 & HFD-3 are observed to be 58^0 & N 50 W, 56^0 & N 40 W and 56^0 & N 40W and 62^0 & N 30 W, 64^0 & S 10 W and 60^0 & N 20 W respectively.

5.8.2 2^{nd} Pressurisation Cycle

Shut-in pressure is the pressure at which a hydro fracture stops propagating and closes following pump shut off. The determination of the shut-in pressure is straightforward when a sharp break is observed in pressure-time curve after the initial fast pressure decline following pump shut-off. However, in many situations, the pressure decay is gradual with no obvious breaks or kinks and shut-in pressure cannot be readily defined. Over the years a number of methods (mostly graphical) have been proposed. When using these methods, it is a common practice to determine the shut-in pressure after two or three pressurization cycles (Hickman & Zoback, 1983). In general, the shut-in pressure determined in the first cycle is usually higher and decreases somewhat in repeated pressurization cycles as the hydro fracture propagates away from the hole. As such, the 2^{nd} pressurization cycle data is used for calculations and adoption of test results.

(Reference: "Rock Stress and its Measurement" Bernard Amadei and Ove Stephansson ; Chapman & Hall Publishers-1997, PP-163)

In case of vertical borehole No. HFD-1, maximum horizontal stress is found to vary from 14.55MPa to 21.06MPa with average value of 17.77MPa whereas minimum horizontal stress is varying from 8.14MPa to 11.47MPa with average value of 9.81MPa.

Maximum horizontal stress from horizontal borehole Nos. HFD-2 & HFD-3 varies from 9.50MPa to 16.80MPa (average 12.76MPa) whereas minimum horizontal stress is found to vary from 6.20MPa to 9.20MPA (average 7.57MPa).

5.8.3 Vertical Stress σ_v

Vertical stress based on an overburden of 300m and average rock density of 2.75T/m^3 works out to be 8.25MPa which matches well as per Hoek & Brown (1980a) relation for world data for 0-3000m depth range.

5.8.4 Ratio of σ_H / σ_v

Ratio of maximum horizontal stress to vertical stress is found to be varying from 2.40 to 3.91 with average value of 3.10 for all the tests for initial pressurization cycle, 1.15 to 2.55 with average value of 1.75 for 2nd cycle and 0.67 to 1.74 with average value of 1.35 for 3rd cycle.

5.8.5 Ratio of σ_h / σ_v

Ratio of minimum horizontal stress to vertical stress for all the tests is varying from 0.82 to 1.60 with average value of 1.22 for initial pressurization cycle, 0.78 to 1.39 with an average value of 1.00 for 2nd cycle and 0.47 to 0.95 with average value of 0.78 for 3rd cycle.

5.9 Conclusions

Hydraulic Fracturing Tests were conducted at 9 Nos. locations in 2 horizontal and 1 vertical bore holes in the Power House area drift of Ratle H.E. Project, J&K. Tests were performed at three sections in each of the 3 boreholes (1 vertical and 2 horizontal orthogonal boreholes at two RDs of 201.00m and 202.00m) respectively to determine magnitude and direction of principal stresses. Based on all the 9 tests carried out in all the three boreholes, the estimated vertical stress based on average overburden of 300m is found to be 8.25MPa. The values of max. principal stress (σ_H) and min. principal stress (σ_h) for the 2nd cycle in vertical borehole No. HFD-1 are recommended for adoption as explained under para 5.8.2. The maximum principal stress varies from

14.55MPa to 21.06MPa with average value of 17.77MPa and minimum principal stress varies from 8.14MPa to 11.47MPa with average value of 9.81MPa. The corresponding values of dip of σ_H vary from 52^0 to 67^0 with dip directions of N40W to S40E. Average values of σ_H / σ_v and σ_h / σ_v are observed to be 2.15 and 1.19 respectively.

To understand the variation of stresses with depth, which is important for the design of any underground structure, it is customary to find out gradients for principal stresses that are measured to compare with numerous relations that are available in foreign publications. Accordingly, the average gradient, K_{av}= {($\boldsymbol{\sigma_H}$+ $\boldsymbol{\sigma_h}$)/2}/ $\boldsymbol{\sigma_v}$ obtained from test results of hydraulic fracturing test has been plotted in Hoek and Brown envelope where NIRM data for Indian projects and data from others is also plotted and the same is presented in Figure No. 26. It is found that the data falls in close proximity of NIRM data and is within the envelope.

Alaknanda H.E. Project, Uttarakhand

Ratle H.E. Project, J & K

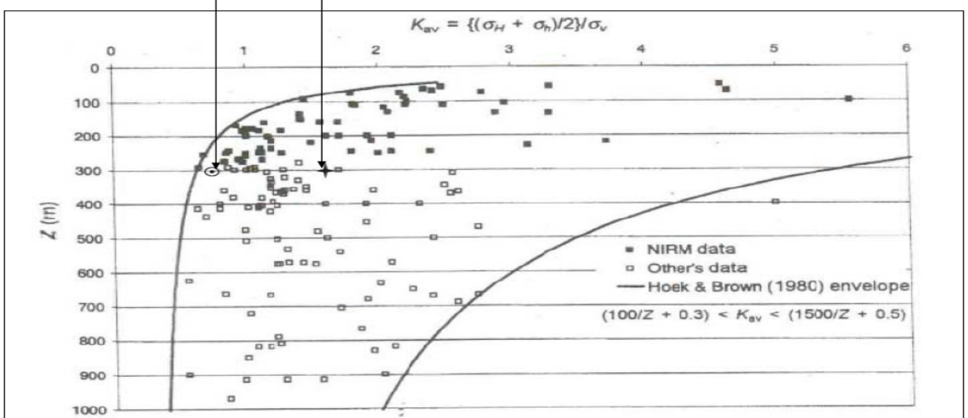


Fig. 26: Hydraulic Fracturing Test data incorporated in Hoek & Brown Envelope

6.0 IN-SITU UNIAXIAL JACKING TEST

(With Deformation Measurements in Boreholes Using Multiple Position Borehole Extensometers)

6.1 In-situ Uniaxial Jacking Test

2 sets comprising of four Nos. uniaxial jacking tests were performed at RDs 192.00m and 195.00m. Tests at RD 192.00m were in horizontal direction with one in the horizontal U/S side and the other in horizontal D/S side while the tests at RD 195.00m were in Vertical direction with one test in vertical upward direction and the

other in vertical downward direction. A schematic diagram depicting locations of Plate Jacking Tests in horizontal and vertical directions is depicted in Figure No. 27.

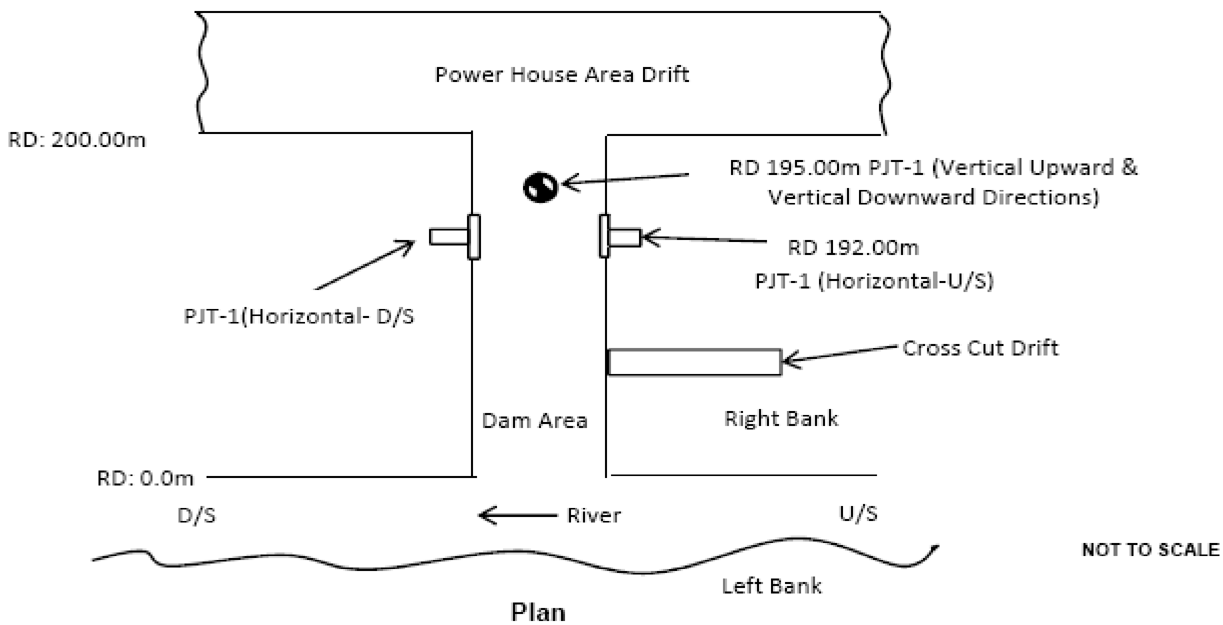


Fig. 27: Schematic Diagram Showing Plate Jacking Test Locations

The test procedure adopted for the in-situ uniaxial test is briefly described below:

Plate jacking test or uniaxial jacking test is normally performed in small tunnels or test adits or drifts to determine the deformability characteristics covering elastic and deformation modulii of a rock mass. Surficial loading is applied on the plate.

The test method given in IS: 7317 - 1993 “Code of Practice for Uniaxial Jacking Test for Modulus of Deformation of Rock” (First Revision) or ISRM “Suggested Methods for Determining In Situ Deformability of Rock” was followed.

6.2 Equipment

Equipment used for accomplishing the test included the following items:

(i) Equipment for test site preparation such as drill & chipping hammers.

(ii) Electrically operated drilling machine for drilling Nx size holes of 6 m depth.

(iii) Instruments for measuring deformations covering data logger and LVDTs with arrangements for different channels measuring deformations at different anchor locations through LVDTs.

(iv) Loading apparatus capable of applying simultaneous uniform pressures to two areas on opposite sides of the tunnel, each approximately 1 m in diameter.

(v) 4 Nos. restraint columns having the capability of sustaining the maximum desired uniform pressure with sufficient factor of safety.

(vi) Hydraulic pump system with necessary fittings, valves, gauges & hoses having sufficient pressure capability & volume to apply & maintain desired pressures to within 3% of a selected value throughout the duration of the test.

6.3 Test Procedure

6.3.1 Site Preparation

The area selected for testing was carefully prepared. All loose rock material was removed by chipping hammers & drill. In order to reduce the restraining influence of adjoining rock, an area with a diameter about 1 ½ times that of the test area was prepared. It was ensured that the two test areas were concentric with & in planes oriented perpendicular to the axis of the restraint column assembly. Rock mass deformations were measured in boreholes through multiple position borehole extensometers behind each loaded area and across the tunnel between each loaded area. A typical test set up (as per ISRM Suggested Method for Determining In-situ Deformability of Rock) is shown in Figure No. 28 below:

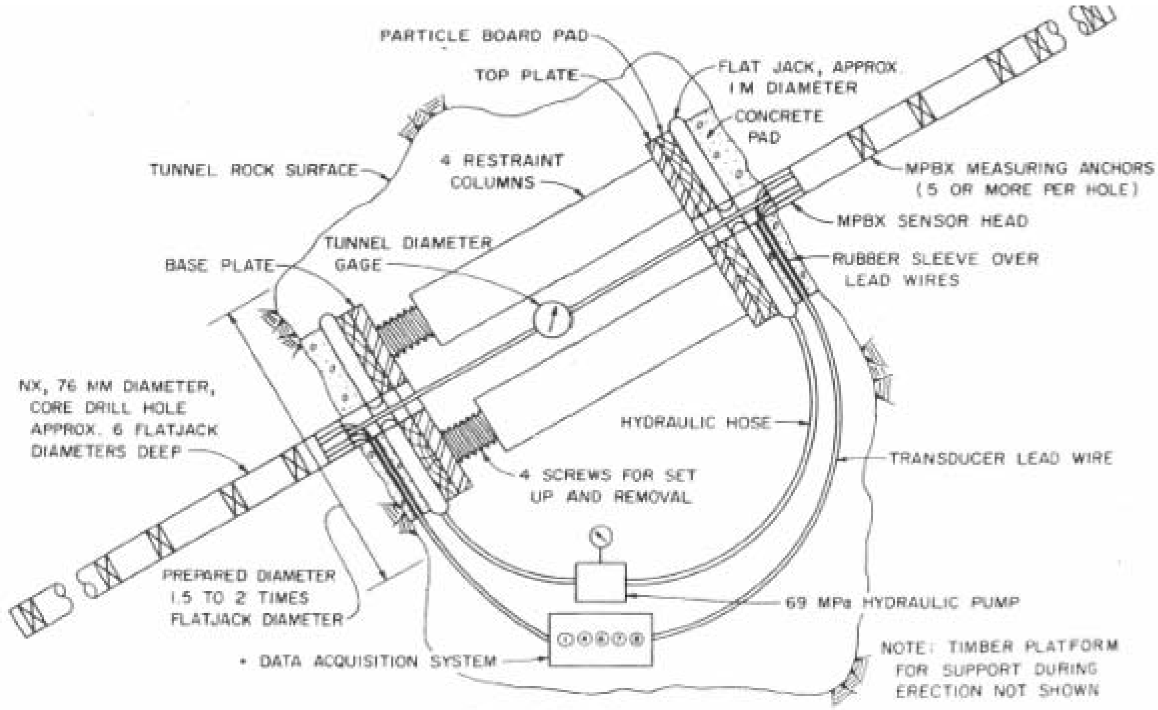


Fig. 1. Uniaxial jacking test.

Fig. 28: Schematic Diagram of Plate Jacking Tests with Deformations Measured Using LVDTs (As per ISRM Suggested Method)

An Nx size hole was core drilled into each prepared test surface taking care to ensure that the two holes were coaxial with each other & with restraint column assembly. Geological logs of all 4 Nos. drill holes along with their locations are presented in Annexure-I.

Based on examination of the retrieved drill core, the deepest anchor was located approximately 6 m below the rock surface in order to provide a fixed point to which the movements of all other anchors could be referenced. The remaining anchors (4 Nos.) were concentrated in the zone of maximum stress between the rock surface & a point approximately 3.0 m back from the surface, they being placed at various depths below the rock surface. Five anchors were similarly installed in the opposite direction.

6.3.2 Equipment Installation

The complete equipment comprising of restraining & load applying set up together with deformation measuring instrumentation comprising of LVDTs and all cables connected to a data logger was then installed. The space between the jack assembly & rock was filled with small aggregate concrete. The concrete was allowed to cure sufficiently to obtain adequate compressive strength prior to commencement of the test. All the readings were taken outside the drift after checking the movement and checking calibration of each LVDT.

6.3.3 Testing

After all components of the instrumentation were installed in the drill holes, they were checked (electronically & mechanically). After the loading & restraining components were installed, another check was made of the instrumentation. A final check of all mechanical, hydraulic & electronic components was made after the concrete pads were placed & again before the first load increment was applied. An initial load of about 20 KN was applied to hold the steel plates in position.

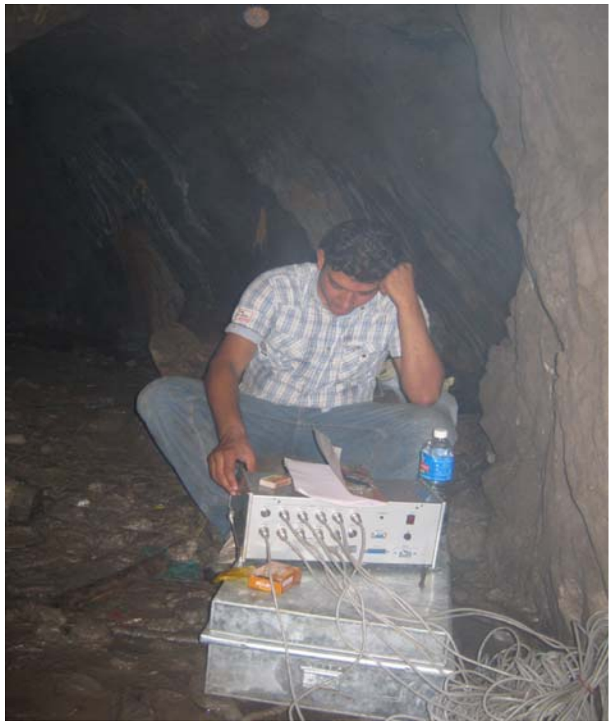
The maximum load corresponding to maximum anticipated stress level was applied in 5 cycles. At least five pressure increments, each followed by a period of zero pressure, were used for each cycle; the unloading was done in 3 to 4 steps. Based on observations during the first pressure increment, the duration of each pressure increment was kept for sufficiently long period & the LVDT readings were continued to be recorded till it was less than 0.001mm over a period of thirty minutes. The jack pressure was maintained within ± 3% of the target value for the duration of each increment.

Data gathered during the test was plotted to provide deformation versus pressure & depth below the surface curves to calculate modulii of deformation/elasticity at various stress levels & the variation of modulii with respect to depth. The test was conducted in vertical as well as horizontal directions.

Photographs 1 & 2 show actual set up of a test in horizontal and vertical directions respectively during the tests at site while Photograph 3 shows the data being recorded.

Photograph 1: Typical Plate Jacking Test Set Up (Horizontal Direction) in Drift

Photograph 2: A Vertical Plate Jacking Test (Vertical Direction) in Progress in Drift at the Project Site



Photograph 3: Data Recording in Progress During a Plate Jacking Test

6.3.4 Calculations

For a uniformly distributed pressure on a circular area, the displacement at any point beneath the center of the area is expressed as follows:

$$w_z = \frac{2q(1-\mu^2)}{E}\left[(a^2+z^2)^{1/2} - z\right] - \frac{qz(1+\mu)}{E}\left[z(a^2+z^2)^{-1/2} - 1\right] \qquad (1)$$

where

w_z = Displacement in the direction of the applied pressure;

z = Distance from the loaded surface to the point where displacement is calculated;

q = Pressure or stress at loaded surface;

a = Radius of loaded area;

μ = Poisson's ratio of the rock;

& E = Modulus of elasticity.

At the surface z = 0 & the expression reduces to:

$$w_{z=0} = \frac{2(1-\mu^2)}{E} qa \qquad \text{--------(2)}$$

When loads are applied with a circular plate with a hole in the center, the effect of the unloaded area in the center must be subtracted. Using the notation:

a_2 = Outer radius of plate; &

a_1 = Radius of hole

$$w_z = \frac{2q(1-\mu^2)}{E} \left[(a_2^2+z^2)^{1/2} - (a_1^2+z^2)^{½}\right]$$

$$+ \frac{z^2 q(1+\mu)}{E} \left[(a_1^2+z^2)^{-1/2} - (a_2^2+z^2)^{-1/2}\right] \text{------------- (3)}$$

After substituting proper values of a_1, a_2, μ & z, equation (3) reduces to:

$$w_z = \frac{q}{E} (kz) \qquad \text{--------- (4)}$$

If displacements wz_1 & wz_2 are measured at points z_1 & z_2, the indicated deformation modulus of the material between depths z_1 & z_2 may be calculated from:

$$E_d = \frac{q.(kz_1 - kz_2)}{(wz_1 - wz_2)} \qquad \text{--------------(5)}$$

Typical sample calculations for evaluating Ed (Deformation Modulus) for rock mass using above equations between depths of 2.00m to 3.00m for Test No. PJT - 1 (Horizontal) (U/S) are presented in Annexure-III

6.4 Plate Load Tests on Rock Mass as per IS: 7317-1993

6.4.1 Power House Site (Right Bank Drift)

Plate Jacking tests were conducted in horizontal U/S (PJT - 1) and horizontal D/S (PJT - 1) directions and vertical upward (PJT – 2 Upward), vertical downward (PLT - 2 Downward) directions at RD 192.00m & RD 195.00m respectively. Deformations were measured in borehole through 5 nos. of anchors connected with LVDTs with a data logger. The results of Plate Jacking Tests conducted on the rock mass in horizontal and vertical directions in the drift at the power house site on the right bank are presented in Tables No. 13 to 20. Stress versus deformation plots for all anchors except the anchor at 6m depth for all cycles of loading and rock deformation versus depth for maximum bearing pressure at each of the two RDs for plate jacking test in vertical and horizontal directions are shown in Figures No. 29 to 48.

6.5 Discussion of Test Results & Recommendations

6.5.1 Rock Mass Deformation Modulii

4 nos. in-situ Plate Jacking Tests, 1 test each in horizontal D/S, horizontal U/S, vertical upward and vertical downward directions, were conducted in the drift in the power house area at two RDs. Values of Modulus of Deformation obtained are summarized below:

S. No.	RD (m)	Direction of Test	Values of E_d, GPa — Zones Between Depth Below Loaded Rock Surface (cm)				Stress Level (MPa)
			50-100	100-200	200-300	300-600	
1	192.00	Horizontal(Down Stream)	10.83	6.80	5.21	6.00	2.56
2	192.00	Horizontal (Up Stream)	10.04	11.85	10.42	11.58	2.56
3	195.00	Vertical (Upward)	9.52	7.54	17.36	8.10	2.56
4	195.00	Vertical (Downward)	12.01	10.37	5.58	8.10	2.56

For each of the tests in vertical (upward & downward) and horizontal (d/s and u/s) directions, the loads were applied in five cycles reaching a maximum value of 2000kN, which corresponded to a stress level of 2.56MPa.

On perusal of test results, it is seen that the values of deformation modulus Ed at the two RDs of 192.00m & 195.00m vary from 9.52GPa to 12.01GPa (50cm-100cm depth) with average value of 10.60GPa; 6.80GPa to 11.85GPa (100cm-200cm depth) with average value of 9.14GPa; 5.21GPa to 17.36GPa (200cm-300cm depth) with average value of 9.64GPa and 6.00GPa to 11.58GPa (300cm-600cm depth) with average value of 8.45GPa respectively. However, the following values of Ed, which are considered to be representative, can be adopted:

Depth below loaded rock surface(cm)	E_d, GPa
50 - 100	9.50
100 - 200	9.00
200 - 300	8.00
300 - 600	8.00

As per Farmer and Kemeny (1992) [Ref: Farmer, I.W. & Kemeny J.M. (1992):"Deficiencies in the rock test data," Int. Conf. Eurock 1992, Thomas Telford, London, pp. 298-303], the modulus of elasticity of intact rock sample is 5 to 20 times of the deformation modulus of the rock mass insitu.
Considering the average value of 12.5, and using the values of E static of 89.30GPa (ignoring one high value of 424.17GPa) obtained in the laboratory (based on an earlier report of the laboratory tests conducted on rock samples from Ratle HEP, J&K) on Gneissic type of rock at the site for the project, the value of deformation modulus works out to be 7.14GPa, which is slightly less than the range of values suggested in the above Table.

Also, in Table 10.7 Page 251 "Engineering in Rocks for Slopes, Foundations & Tunnels" Editor: T. Ramamurthy, the deformation modulus in Biotite Gneiss at stress level of 2.72MPa using Plate Loading Test at Dhauliganga HEP, Uttarakhand is indicated to be 3.20 GPa. The ratio of Plate Jacking Test (PJT) & Plate Loading Test (PLT) i.e. PJT/PLT is suggested to be 2.5 in Table 10.8 –Comparison of test Results, P252 of the above reference. Accordingly, the deformation modulus for PJT corresponding to the value of 3.20 GPa obtained in PLT works out to be 8.00GPa (3.20x2.5). The recommended values of deformation modulus for the rock mass at the project site as indicated in the Table above vary from 8.0GPa to 9.5GPa for marginally lower stress level of 2.56MPa.
Considering the above interrelationships, the recommended values for adoption are, therefore, reasonable.

For use in the designs, the above recommended values may be suitably moderated taking into account the variations in geological conditions, stress levels and their orientations.

7.0 DIRECT SHEAR TESTS

7.1 Test Procedure

The test procedure adopted for the in-situ direct shear tests is briefly described below:

The test method given in IS: 7746 – 1991 (Re-affirmed 1996) "In-situ Shear Test on Rock – Code of Practice" (First Revision) was followed. In this test, peak and residual direct shear strengths are measured as a function of stress normal to the sheared plane, by conducting five tests on the same test horizon, with each specimen tested at a different but constant normal stress. The above procedure was followed for conducting rock-to-rock direct shear tests as well as concrete-to-rock direct shear tests.

7.1.1 Equipment

Equipment used for conducting the test included the following:

- Equipment for cutting and encapsulating the test block. This included rock saws, drills, hammers and chisels, formwork of appropriate dimensions and rigidity, and steel shear box (700 x 700 x 350mm).
- A hydraulic jack of 200 Tonne capacity to apply the required normal load.
- A hydraulic pump capable of maintaining normal load to within ± 2% of a selected value throughout the test.
- A reaction system to transfer normal loads uniformly to the test block, including rollers to ensure that at any given normal load, the resistance to shear displacement was less than 1% of the maximum shear force applied in the test. Wire ties and turnbuckles were used to install and secure the equipment.
- A hydraulic jack of adequate capacity with 150 mm travel for applying shear force.
- A hydraulic pump to pressurize the shear force system.
- A reaction system to transmit the shear force to the test block. The shear force was distributed uniformly along one face of the specimen. The resultant line of applied shear force passed through the centre of the base of the shear plane, with an angular tolerance of ± 5 degrees.
- One system for measuring normal force and another for measuring applied shear force with an accuracy better than 2% of the maximum forces reached in the test. Recently calibrated load gauges were used for the purpose.
- Equipment for measuring shear, normal and lateral displacements. Displacements were measured using dial gauges at 4 locations on the specimen block or encapsulating material. The shear displacement

measuring system was having a travel of 25 mm and an accuracy of 0.01 mm. The normal and lateral displacement measuring systems had travel of 25 mm and accuracy of 0.01 mm.

7.1.2 Procedure

For rock-to rock test, the in-situ rock was cut to the required dimensions (700 x 700 x 350 mm) using methods that avoided disturbance or loosening of the block. The base of the test block coincided with the plane to be sheared. For concrete-to-rock test, first the rock surface was prepared by chiseling so that the maximum trough depth of the undulations did not exceed 10 mm. Concrete block was then cast on the prepared rock surface. The concrete block was adequately cured and allowed to harden before conducting the test.

A layer of 20 mm thick weak material viz. foamed polystyrene was applied around the base of the test block and the remainder of the block was then encapsulated in the steel shear box to prevent collapse or significant distortion of the block during testing. The load bearing faces of the encapsulated block were flat (tolerance ± 1 mm) and at the correct inclination to the shear plane (tolerance ± 2′). Reaction pads, anchors etc. required to carry the thrust from normal and shear load systems to adjacent sound rock were carefully positioned and aligned. All displacement gauges were checked for rigidity, adequate travel and freedom of movement and a preliminary set of load and displacement readings was recorded. Normal load was then raised in increments to the full value specified for the test, recording the corresponding consequent normal displacement of the test block at each increment, when the displacements stabilized.

Shear force was then applied in increments in such a way as to control the rate of shear displacement and the normal and shear displacements of the block for each increment of shear load were recorded. As the applied shear force P_{sa} is inclined at 15^0 to the horizontal plane, the total shear force is calculated as horizontal component of P_{sa} and is equal to P_{sa} x $Cos15^0$.

The total normal load acting on a shear plane P_n is the sum of the normal load applied by the jack resting on the block and normal component of applied inclined force on the base (given by P_{sa} x $Sin15^0$). To account for this normal component of the inclined force and to keep the normal stress approximately constant, the applied normal force was reduced by P_{sa} x $Sin15^0$ each time the applied shear force was increased.

After reaching peak strength, the shear displacement readings were continued to be recorded to adequately define the shear stress – shear displacement curve and to establish the residual shear strength parameters.

7.1.3 Calculations

Shear and normal stresses are computed as follows:

$$\text{Shear Stress, } \tau = \frac{P_S}{A}$$

$$\text{Normal Stress, } \sigma_n = \frac{P_n}{A}$$

Where,

P_S = Total shear force;

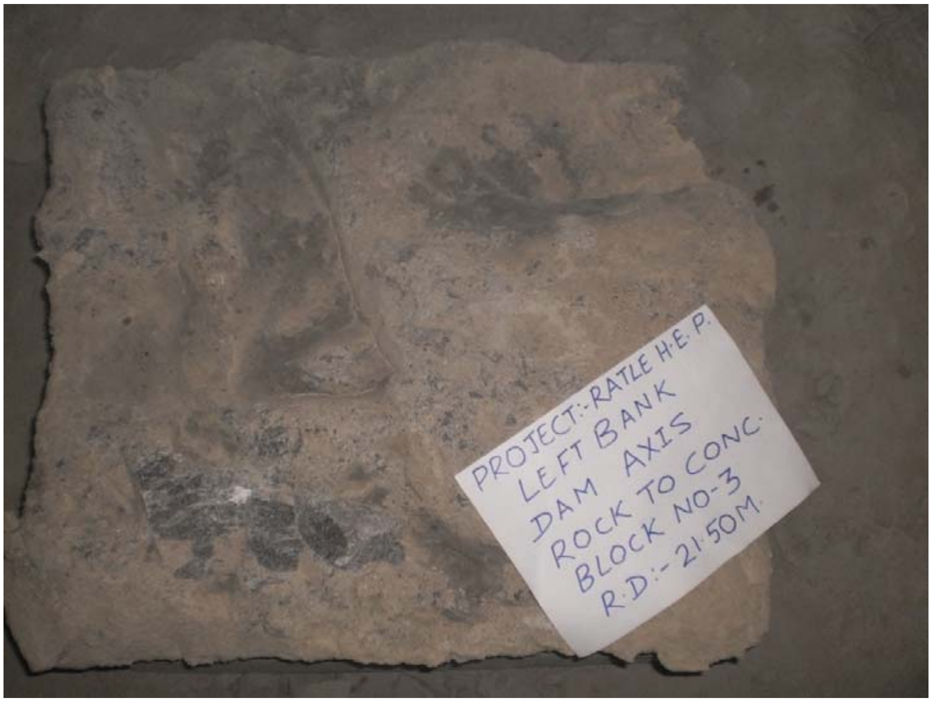
P_n = Total normal force; and

A = Area of shear surface corrected to account for shear displacement.

Graphs of peak and residual shear strength versus normal stress are plotted and from these plots the values of peak and residual shear strength parameters are derived. Photographs 4 to 6 show set up for direct shear test and views of overturned blocks after the in-situ direct shear tests for rock to rock and concrete to rock interfaces in the drifts.

Photograph 4: A View of Direct Shear Test About to be Started- Ratle H.E. Project, J&K

Photograph 5: A View of the Overturned Block After Shear Test (Rock to Rock)- Ratle H.E. Project, J&K



Photograph 6: A View of the Overturned Block After Shear Test (Concrete to Rock)- Ratle H.E. Project, J&K

7.2 Test Results

7.2.1 In-situ Direct Shear Test Results

7.2.2 Rock to Rock Interface

7.2.2.1 Dam Site (Right Bank Drift; RD 28.00 m to 35.70 m)

Results of in-situ direct shear tests for rock-to-rock interface in right bank drift at the dam site are given in Tables 21 to 25. The corresponding shear stress Vs shear displacement curves are shown in Figs. 48 to 52. Based on shear strength versus normal stress plots shown in Figs. 53 &54, the values of peak & residual shear strength parameters for rock-to-rock interface are furnished in Table 26.

7.2.2.2 Dam Site ((Left Bank Drift; RD 8.00 m to 15.00 m)

Results of in-situ direct shear tests for rock-to-rock interface in left bank drift at the dam site are given in Tables 27 to 31. The corresponding shear stress Vs shear displacement curves are shown in Figs. 55 to 59. Based on shear strength versus normal stress plots shown in Figs. 60 & 61, the values of peak & residual shear strength parameters for rock-to-rock interface are furnished in Table 32.

7.2.2.3 Power House Drift (RD 202.00 m + 8.40 m towards U/S to RD 202.00 m)

Results of in-situ direct shear tests for rock-to-rock interface in power house drift are given in Tables 33 to 37. The corresponding shear stress Vs shear displacement curves are shown in Figs. 62 to 66. Based on shear strength versus normal stress plots shown in Figs. 67 & 68, the values of peak & residual shear strength parameters for rock-to-rock interface are furnished in Table 38.

7.2.3 Concrete to Rock Interface

7.2.3.1 Dam Site (Right Bank Drift, RD 19.00 m to 26.50 m)

Results of in-situ direct shear tests for concrete-to-rock interface in right bank drift at the dam site are given in Tables 39 to 43. The corresponding shear stress Vs shear displacement curves are shown in Figs. 69 to 73. Based on shear strength versus normal stress plots shown in Figs. 74 & 75, the values of peak & residual shear strength parameters for concrete-to-rock interface are furnished in Table 44.

7.2.3.2 Dam Site (Left Bank Drift, RD 18.60 m to 23.60 m)

Results of in-situ direct shear tests for concrete-to-rock interface in left bank drift at the dam site are given in Tables 45 to 49. The corresponding shear stress Vs shear displacement curves are shown in Figs. 76 to 80. Based on shear strength versus normal stress plots shown in Figs. 81 & 82, the values of peak & residual shear strength parameters for concrete-to-rock interface are furnished in Table 50.

7.3 Discussion of Test Results & Recommendations

7.3.1 In-situ Shear Strength Parameters

7.3.1.1 Rock-to-Rock Interface

A set of five blocks were tested in the right bank drift from RD 28.00 m to RD 35.70 m at the dam site. The values obtained from these tests for the rock to rock interface were 0.46 MPa & 46° for cohesion & friction angle respectively for the peak shear strength. The corresponding values for the residual shear strength were; cohesion 0.27MPa & friction angle 44.°.

Another set of five blocks were tested in the left bank drift from RD 8.00 m to RD 15.00 m at the dam site. The values obtained from these tests for the rock to rock interface were 0.48 MPa & 50° for cohesion & friction angle respectively for the peak shear strength. The corresponding values for the residual shear strength were; cohesion 0.33 MPa & friction angle 47.5°.

Another set of five blocks were tested in the Power House drift from 202.00 m +8.40 m towards u/s to RD 202.00 m at the power house site. The values obtained from these tests for the rock to rock interface were 0.47 MPa & 46° for cohesion & friction angle respectively for the peak shear strength. The corresponding values for the residual shear strength were; cohesion 0.37 MPa & friction angle 44.5°.

Based on tests conducted in the both drifts at the dam site and drift at the power house site, the following values of rock-to-rock shear strength parameters may be considered as representative values for design purpose:

Dam Site				
Type of Shear Strength	**Shear Strength Parameters (Rock to Rock)**			
	Right Bank Drift		**Left Bank Drift**	
	Cohesion intercept, C (MPa)	Angle of shearing resistance, ϕ (Degree)	Cohesion intercept, C (MPa)	Angle of shearing resistance, ϕ (Degree)
Peak	0.46	46	0.48	50
Residual	0.27	44	0.33	47.5

Power House Site		
Type of Shear Strength	**Shear Strength Parameters (Rock to Rock)**	
	Cohesion intercept, C (MPa)	Angle of shearing resistance, ϕ (Degree)
Peak	0.47	46
Residual	0.37	44.5

The above values may be moderated for use in the designs taking into account the geological conditions and the size effect.

7.3.1.2 Concrete-to-Rock Interface

A set of five blocks were tested in the right bank drift from RD 19.00 m to RD26.50 m at the dam site. The values of cohesion & friction angle obtained from these tests for the concrete to rock interface were 0.31 MPa & 46° respectively for the peak shear strength. The corresponding values for the residual shear strength parameters were; cohesion 0.21 MPa & friction angle 43°.

Another set of five blocks were tested in the left bank drift from RD 18.60 m to RD 23.60 m at the dam site. The values of cohesion & friction angle obtained for the peak shear strength were 0.29 MPa & 48° respectively. The corresponding values for the residual shear strength parameters were; cohesion 0.21 MPa & friction angle 44°.

Based on tests conducted in the both drifts at the dam site, the following shear strength parameters for the rock mass may be considered as representative values for design purpose:

Type of Shear Strength	**Shear Strength Parameters (Concrete to Rock)**			
	Right Bank Drift		**Left Bank Drift**	
	Cohesion intercept, C (MPa)	Angle of shearing resistance, ϕ (Degree)	Cohesion intercept, C (MPa)	Angle of shearing resistance, ϕ (Degree)
Peak	0.31	46	0.29	48
Residual	0.21	43	0.21	44

The above values may be moderated for use in the designs taking into account the geological conditions and the size effect.

A perusal of the shear strength parameters data reveals that the quality of rock on the left bank is superior as compared to that on the right bank.

Annexure-I

PROJECT: GVK RATLE H.E. PROJECT (J. & K.)

Bore Hole No. : PJT-1 (Horizontal)(Downstream)
Sheet-1

GEOLOGICAL LOG OF DRILL HOLE

Location : AT RD 192M
Angle with Horizontal : 0^o
Date of Start : 08/04/2011
Date of completion : 13/04/2011

Type of Bit Used : IMP Diamond Bit 32 carret
Total Depth : 6.00 m

Feature : For PJT Testing (Downstream)
Type of Core Barrel : NWT - Double Tube
Drilling Agency : AECS

Depth (m)	Lithology: Description	Lithology: Legend	Size of core pieces (%): < 10 mm	10 to 25 mm	25 to 75 mm	75 to 150 mm	>150 mm	Structural Condition: Description	Percent Core Recovery (20 / 40 / 60 / 80 / 100)	Type of bit	RQD (%)	Size of hole	Casing	Drill Water Loss: Partial	Drill Water Loss: Complete	Special Observations
0.00–1.00	(0.00-6.00m) Hard, compact, massive and light grey quartzitic gneiss	V V V		4		28	38	Massive Rock Mass	70	IMP	66	Nx-Size	Un Cased	Partial		
1.00–2.00		V V V		10	7	18	63		98		73					
2.00–3.00		V V V			3	34	43		80		77					
3.00–4.50		V V V		7	9	6	55		77		55					
4.50–6.00		V V V		2	3	18	27		50		45					

ORAGANISATION : AECS Engineering & Geotechnical Services Pvt. Ltd.

DEPARTMENT : GEOTECHNICAL

Logged By :MR. RAJIV RANJAN

APPROVED BY : DR. R.C. UPADHYAYA

PROJECT: GVK RATLE H.E. PROJECT (J. & K.)

Bore Hole No. : PJT-1(Horizontal)(Upstream)
Sheet-1

GEOLOGICAL LOG OF DRILL HOLE

Location : AT RD 192M
Angle with Horizontal : 0°
Date of Start : 01/04/2011
Date of completion : 07/04/2011

Type of Bit Used : IMP Diamond Bit 32 carret
Total Depth : 6.00 m

Feature : For PJT Testing (Upstream)
Type of Core Barrel : NWT - Double Tube
Drilling Agency : AECS

Depth (m)	Lithology: Description	Lithology: Legend	Size of core pieces (%): < 10 mm	10 to 25 mm	25 to 75 mm	75 to 150 mm	>150 mm	Structural Condition: Description	Percent Core Recovery (20, 40, 60, 80, 100)	Type of bit	RQD (%)	Size of hole	Casing	Drill Water Loss: Partial	Drill Water Loss: Complete	Special Observations
0.00 0.20 1.00	(0.00-6.00m) Hard, compact, massive and whitish grey alternate bands of quartzite & gneiss.			10	7	8	70	Massive Rock Mass	95	IMP	70	Nx-Size	Un Cased	Partial		
1.20 1.35 2.00				5	6	17	62		90		62					
2.35 2.45 3.00				5	7	9	76		97		76					
4.00				23	29	43			95		25					
4.10 5.00				10	31	39	16		96		39					
6.00				6	22		62		90		62					

ORAGANISATION : AECS Engineering & Geotechnical Services Pvt. Ltd.

DEPARTMENT : GEOTECHNICAL

Logged By :MR. RAJIV RANJAN

APPROVED BY : DR. R.C. UPADHYAYA

PROJECT: GVK RATLE H.E. PROJECT (J. & K.)

Bore Hole No. : PJT -2 (Vertical)(Upward)

Sheet 1

GEOLOGICAL LOG OF DRILL HOLE

Location : AT RD 195M
Angle with Horizontal : 90° (Upward)
Date of Start : 14/04/2011
Date of completion : 19/04/2011

Type of Bit Used : IMP Diamond Bit 32 carret
Total Depth : 6.00 m

Feature : For PJT Testing (Upward)
Type of Core Barrel : NWT - Double Tube
Drilling Agency : AECS

Depth (m)	Lithology: Description	Legend	Size of core pieces (%): < 10 mm	10 to 25 mm	25 to 75 mm	75 to 150 mm	>150 mm	Structural Condition: Description	Percent Core Recovery	Type of bit	RQD (%)	Size of hole	Casing	Drill Water Loss: Partial	Drill Water Loss: Complete	Special Observations
1.00	(0.00-3.00m) Light grey, hard compact and massive quartzitic gneiss					12	83	Massive Rock Mass	95	IMP	95	Nx-Size	Un Cased	No Water Loss		Water drained out of hole due to vertically upward direction of hole. No water loss was observed.
2.00							93		93		93					
3.00				10	11	34	40		95		66					
4.00	(3.00-6.00m) Greyish white hard, compact and massive quartzite with mica			13	6		61		80		61					
5.00				3	17	38	22		80		51					
6.00			12	23	16	47			98		37					

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DEPARTMENT : GEOTECHNICAL

Logged By :MR. RAJIV RANJAN

APPROVED BY : DR. R.C. UPADHYAYA

Annexure-II

Sample Calculations for Calculating Ed (Deformation Modulus) of Rock Mass Between Two Anchor Depths.

(Refer Table 16: Results of Plate Loading Test at Power House Site)

Test No. PJT-1 (Horizontal)(Upstream); Location: Power House Area Drift; RD: 192.00m

Details of anchors:
Anchor 1: 0.50m
Anchor 2: 1.00m
Anchor 3: 2.00m
Anchor 4: 3.00m
Anchor 5: 6.00m
Typical calculations for zone: 2.00m to 3.00m
Data Used:
Z_1=200cm; Z_2=300cm; a_1 (radius of inner hole)=3.9cm; a_2=outer radius of plate=50cm; q=2.56MPa for load of 2000kN, μ (Poisson's ratio)=0.25 (assumed); μ^2 =0.063

$$w_z = \frac{2q(1-\mu^2)}{E}\left[(a_2^2+z^2)^{1/2} - (a_1^2+z^2)^{1/2}\right] + \frac{z^2 q(1+\mu)}{E}\left[(a_1^2+z^2)^{-1/2} - (a_2^2+z^2)^{-1/2}\right].$$

In the above equation, w_z is known for two chosen depths i.e. w_1(200cm) & w_2(300cm) from the test data. In the above equation, except E all other terms are known and are calculated as below and finally E is calculated using:

q*(kz1-kz2)/w1-w2

where Kz_1
$= 2x(1-\mu^2)x((a_2^2+z_1^2)^{1/2}-(a_1^2+z_1^2)^{1/2})+z_1^2x(1+\mu)x((a_1^2+z_1^2)^{-1/2}-(a_2^2+z_1^2)^{-1/2})$

and Kz_2 is same as above except that z_1 is replaced by z_2.

$a_2^2+z_1^2$ =4250000; $\sqrt{a_2^2+z_1^2}$=2061.55

$1/\sqrt{a_2^2+z_1^2}$ or $(a_2^2+z_1^2)^{-1/2}$ =0.000485

$\sqrt{a_1^2+z_1^2}$=2000.38

$1/\sqrt{a_1^2+z_1^2}$=0.000499

$a_2^2+z_2^2=9250000$

$\sqrt{a_2^2+z_2^2}=3041.38$

$1/\sqrt{a_2^2+z_2^2}=0.000329$

$\sqrt{a_1^2+z_2^2}=3000.25$

$1/\sqrt{a_1^2+z_2^2}=0.000333$

$KZ_1=188.87$

$KZ_2=127.82$

$w_1=0.0291cm$
$w_2=0.014cm$

Using the above equation and the corresponding values calculated as above. E is calculated as:
E (2.00m-3.00m) = $q*(kz1-kz2)/w_1-w_2$=10418.63MPa = 10.42 GPa ***(Refer Table 16)***

Fifth cycle where the bearing pressure is maximum, has been chosen for calculation of E.

$1/\sqrt{a_2^2+z_2^2}=0.000329$

$\sqrt{a_1^2+z_2^2}=3000.25$

$1/\sqrt{a_1^2+z_2^2}=0.000333$

$KZ_1=188.87$

$KZ_2=127.82$

$w_1=0.0291cm$
$w_2=0.014cm$

Using the above equation and the corresponding values calculated as above. E is calculated as:
E (2.00m-3.00m) = $q*(kz1-kz2)/w_1-w_2$=10418.63MPa = 10.42 GPa ***(Refer Table 16)***

Fifth cycle where the bearing pressure is maximum, has been chosen for calculation of E.

Table -1

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 1**
Bore Hole Direction : Vertical (Downward) **Bore Hole No. :HFD-1**
Test Depth :24.20 to 25.00 m
RD: 201.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	14.76	510	52^0 / N40^0 W
	Shut in	13.18	570	
Cycle 2	Reopening	13.35	750	
	Shut in	11.47	810	
Cycle 3	Reopening	9.12	980	
	Shut in	7.42	1030	

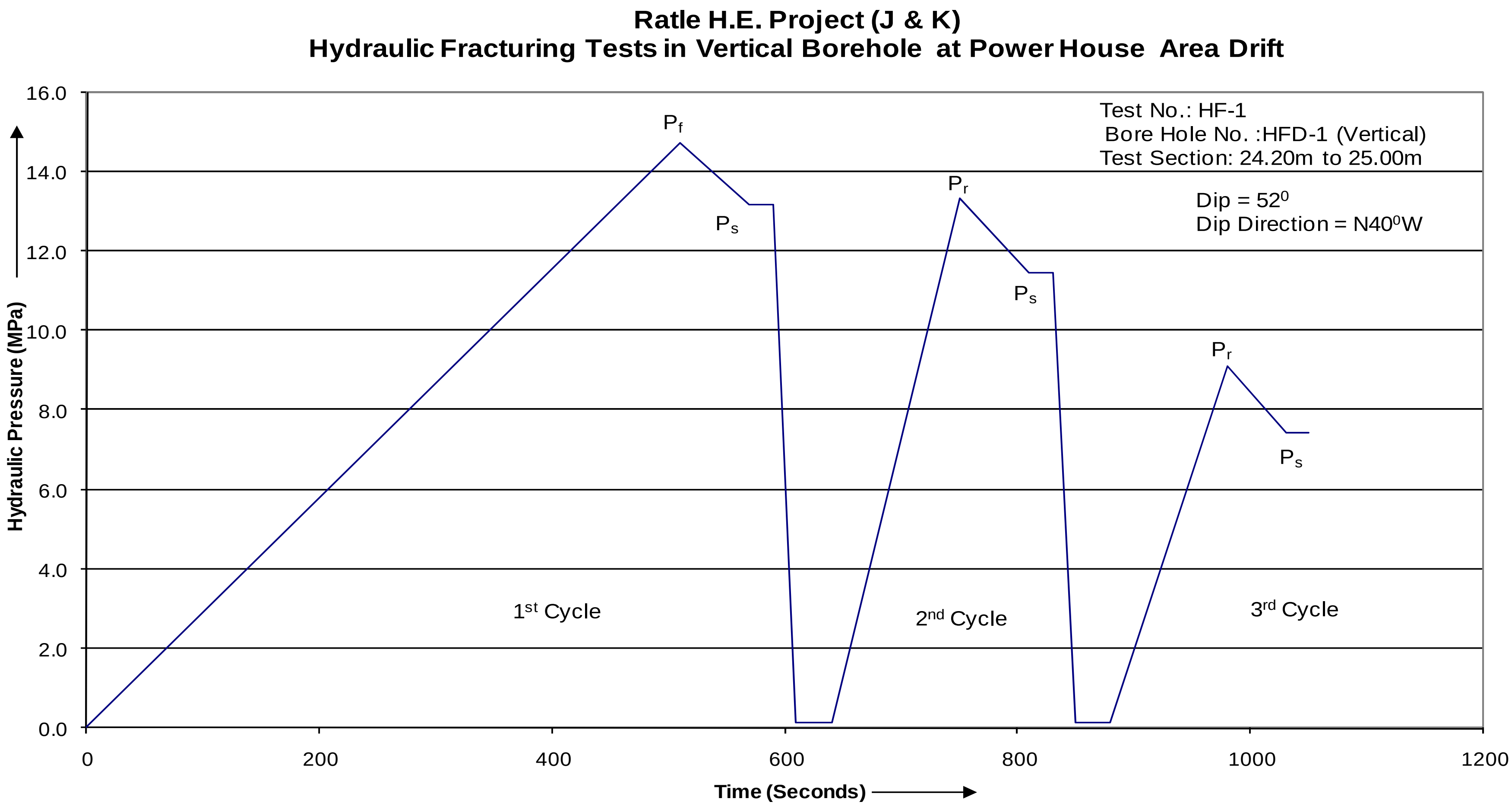

Fig. 11: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF -1

Table -2

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 2**
Bore Hole Direction : Vertical (Downward) **Bore Hole No. :HFD-1**
Test Depth :22.20 to 23.00 m
RD: 201.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	12.15	390	54^0 / N30^0 W
	Shut in	10.21	430	
Cycle 2	Reopening	9.87	680	
	Shut in	8.14	720	
Cycle 3	Reopening	8.48	940	
	Shut in	6.61	980	

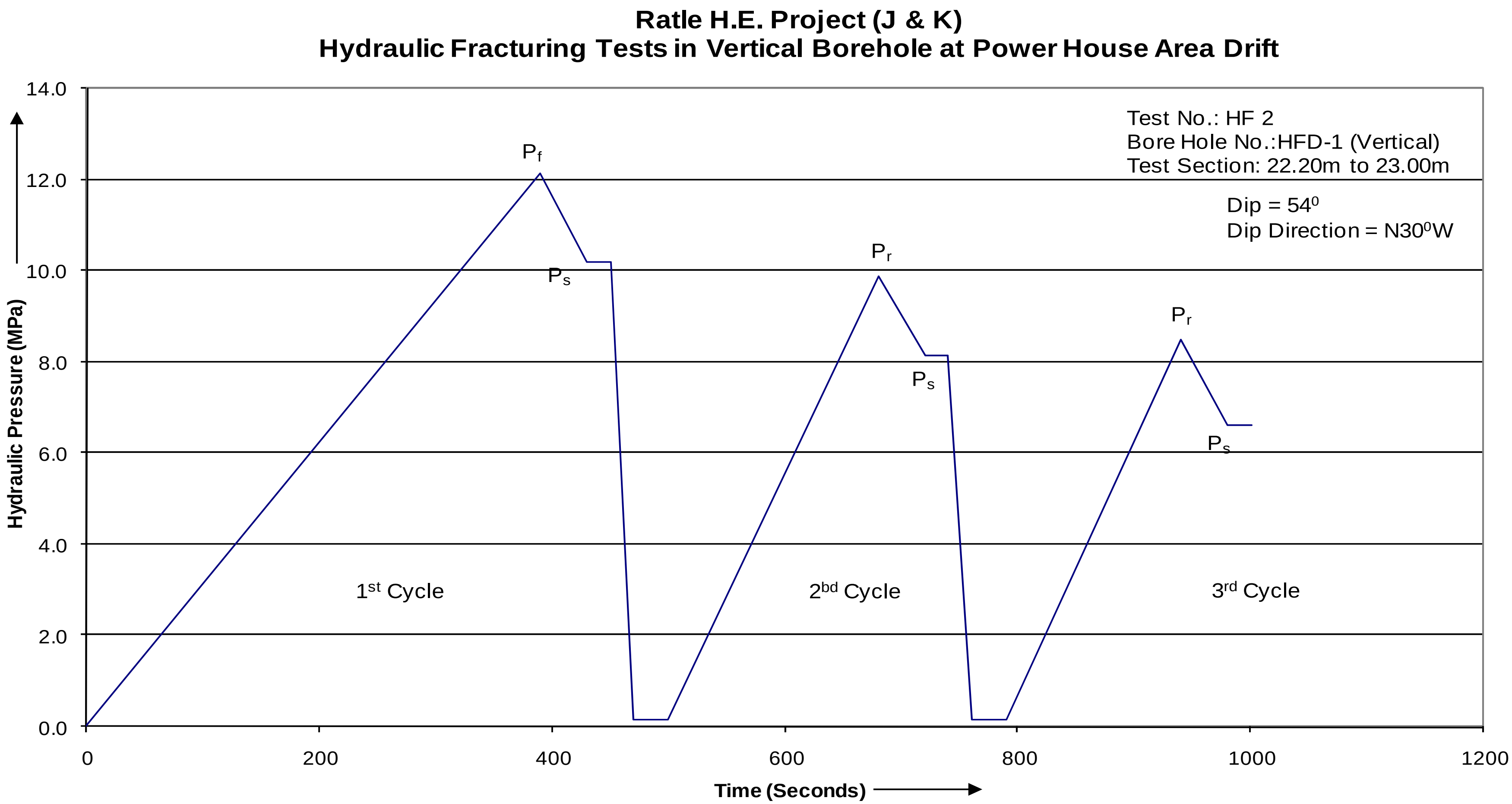

Fig. 12: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF-2

Table -3

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 3**
Bore Hole Direction : Vertical (Downward) **Bore Hole No. :HFD-1**
Test Depth :21.00 to 21.80 m
RD: 201.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	13.24	430	67^0 / S40^0 E
	Shut in	11.15	450	
Cycle 2	Reopening	11.74	670	
	Shut in	9.81	710	
Cycle 3	Reopening	9.18	840	
	Shut in	7.84	860	

Ratle H.E. Project (J & K)
Hydraulic Fracturing Tests in Vertical Borehole at Power House Area Drift

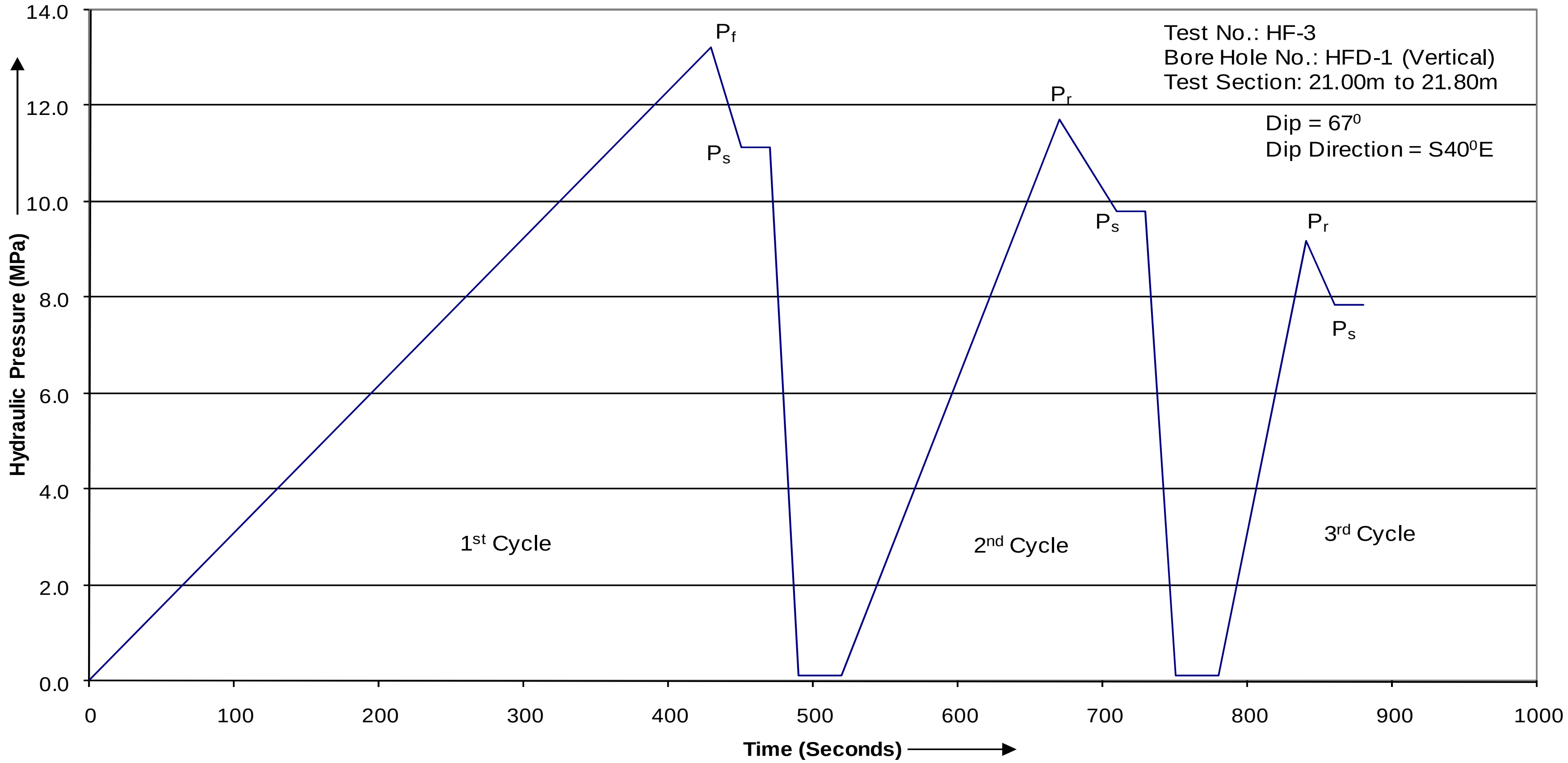

Fig. 13: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF-3.

Table -4

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift
Test No. : HF 4
Bore Hole Direction : Horizontal
Bore Hole No. :HFD-2 (U/S)
Test Depth :29.00 to 29.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	13.80	490	58^0 / N50^0 W
	Shut in	11.20	510	
Cycle 2	Reopening	11.80	670	
	Shut in	8.20	730	
Cycle 3	Reopening	9.10	1080	
	Shut in	6.34	1110	

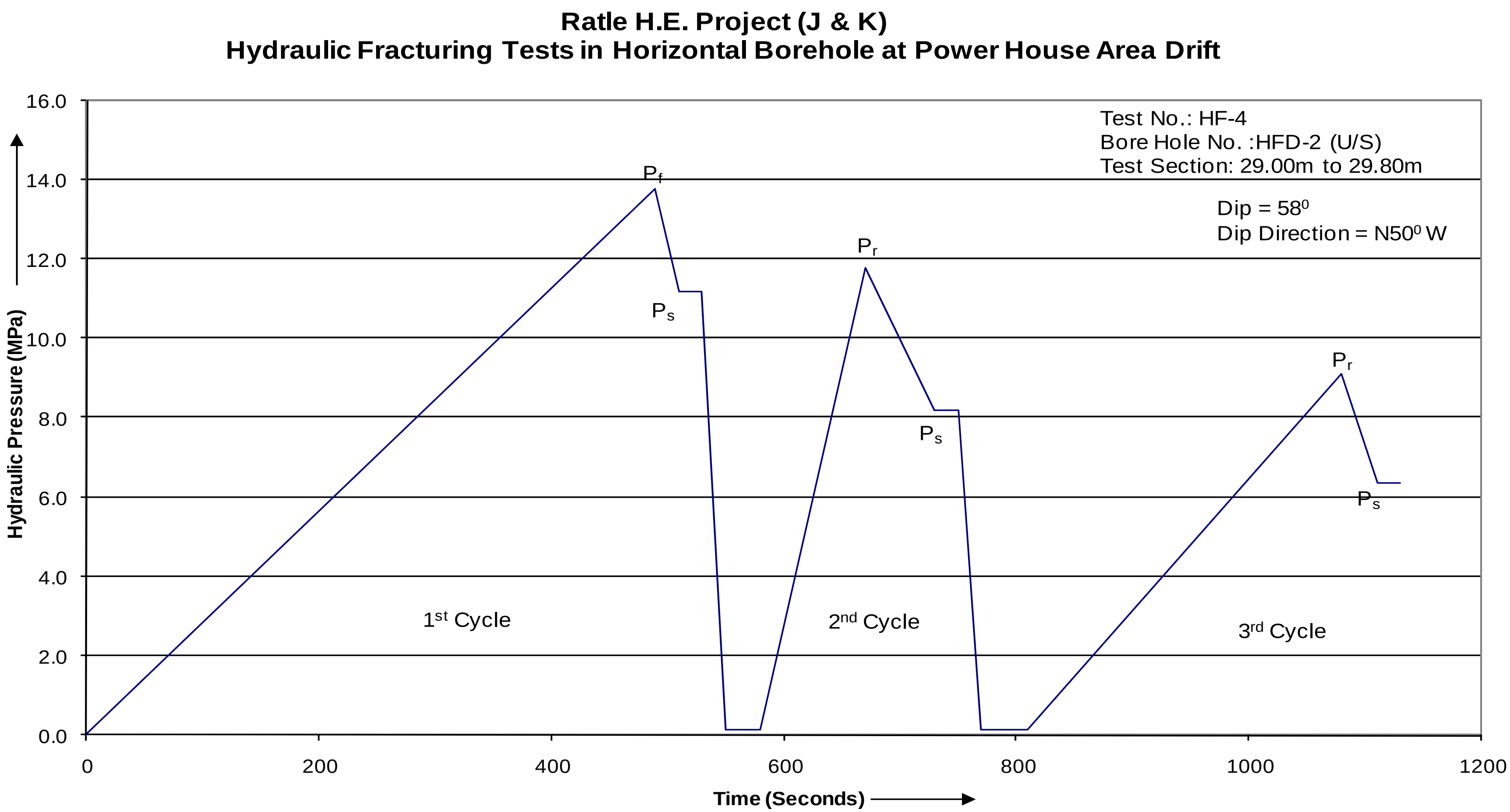

Fig. 14: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF -4.

Table -5

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 5**
Bore Hole Direction : Horizontal **Bore Hole No. :HFD-2 (U/S)**
Test Depth :28.00 to 28.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	11.88	380	56^0 / N40^0 W
	Shut in	9.74	420	
Cycle 2	Reopening	10.26	530	
	Shut in	8.41	610	
Cycle 3	Reopening	8.87	960	
	Shut in	6.74	1060	

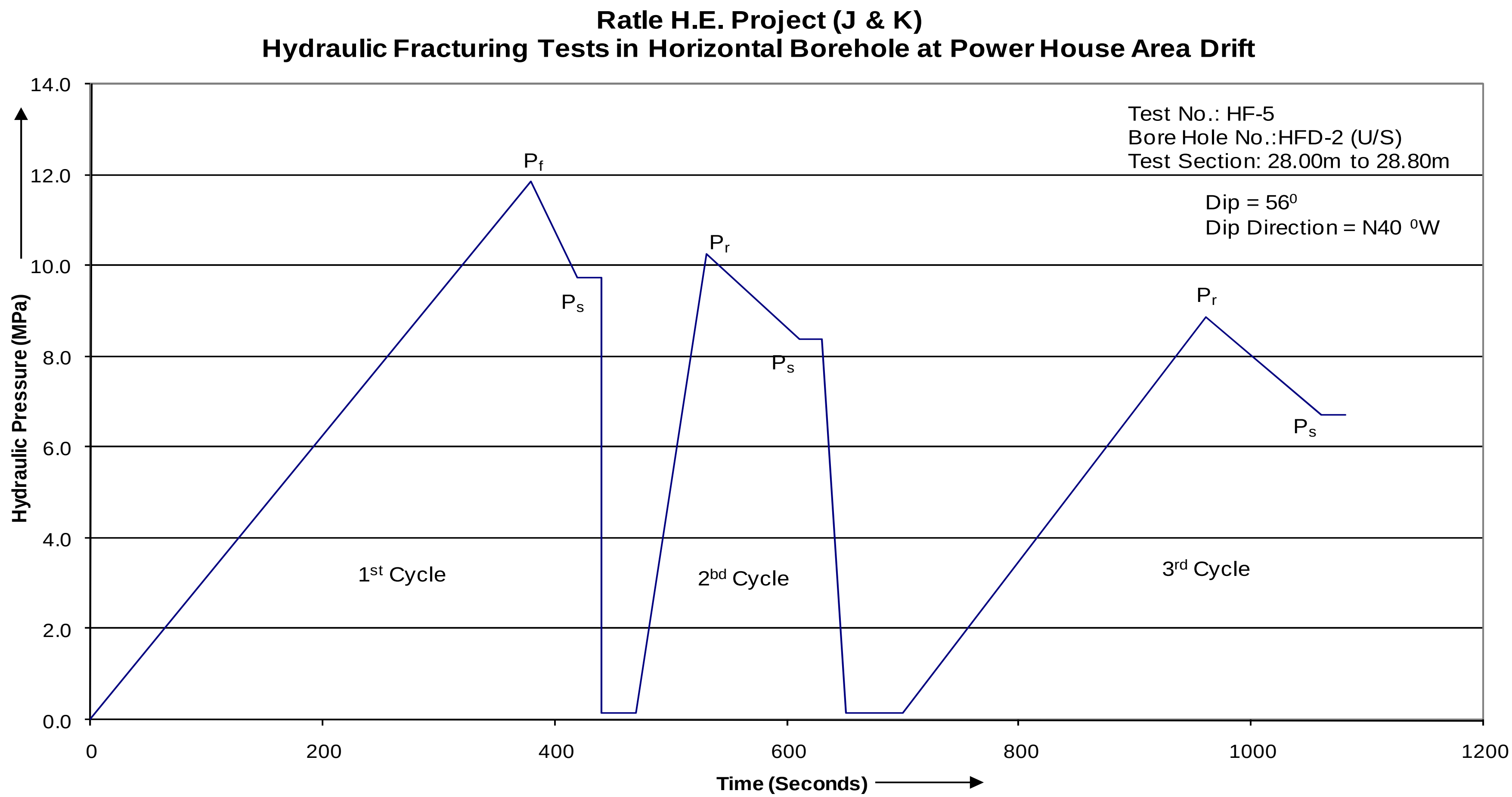

Fig. 15: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF-5.

Table -6

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 6**
Bore Hole Direction : Horizontal **Bore Hole No. :HFD-2 (U/S)**
Test Depth :26.00 to 26.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes- Dip Amount /Dip Direction
Cycle 1	Breakdown	12.70	460	56^0 / N40^0 W
	Shut in	11.20	510	
Cycle 2	Reopening	10.80	740	
	Shut in	9.20	760	
Cycle 3	Reopening	8.70	940	
	Shut in	7.20	990	

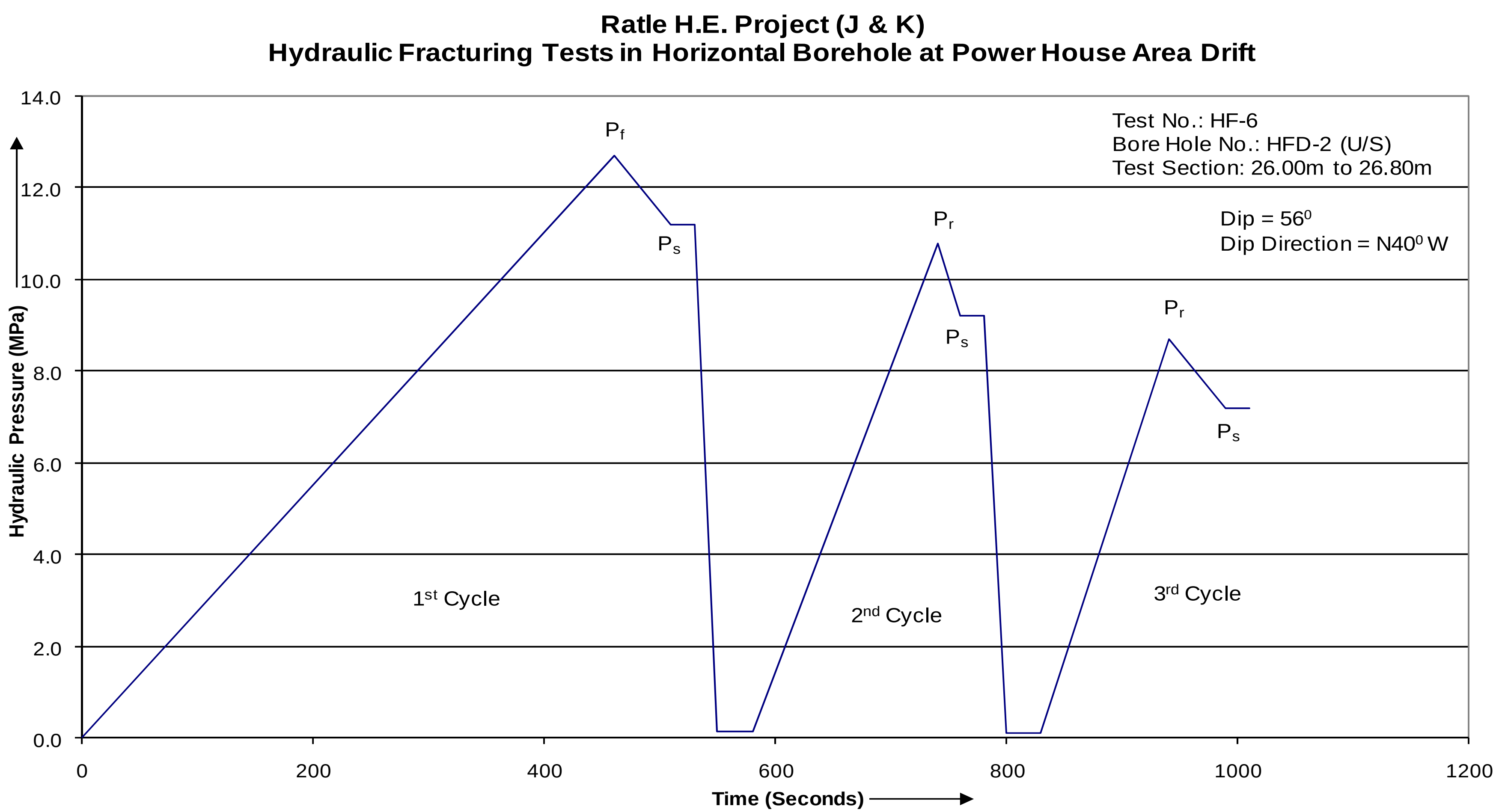

Fig. 16: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF1-6.

Table - 7

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 7**
Bore Hole Direction : Horizontal **Bore Hole No. :HFD-3 (D/S)**
Test Depth :29.00 to 29.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	8.10	360	62^{0} / N30^{0} W
	Shut in	6.80	400	
Cycle 2	Reopening	7.54	510	
	Shut in	6.40	540	
Cycle 3	Reopening	6.00	870	
	Shut in	5.40	930	

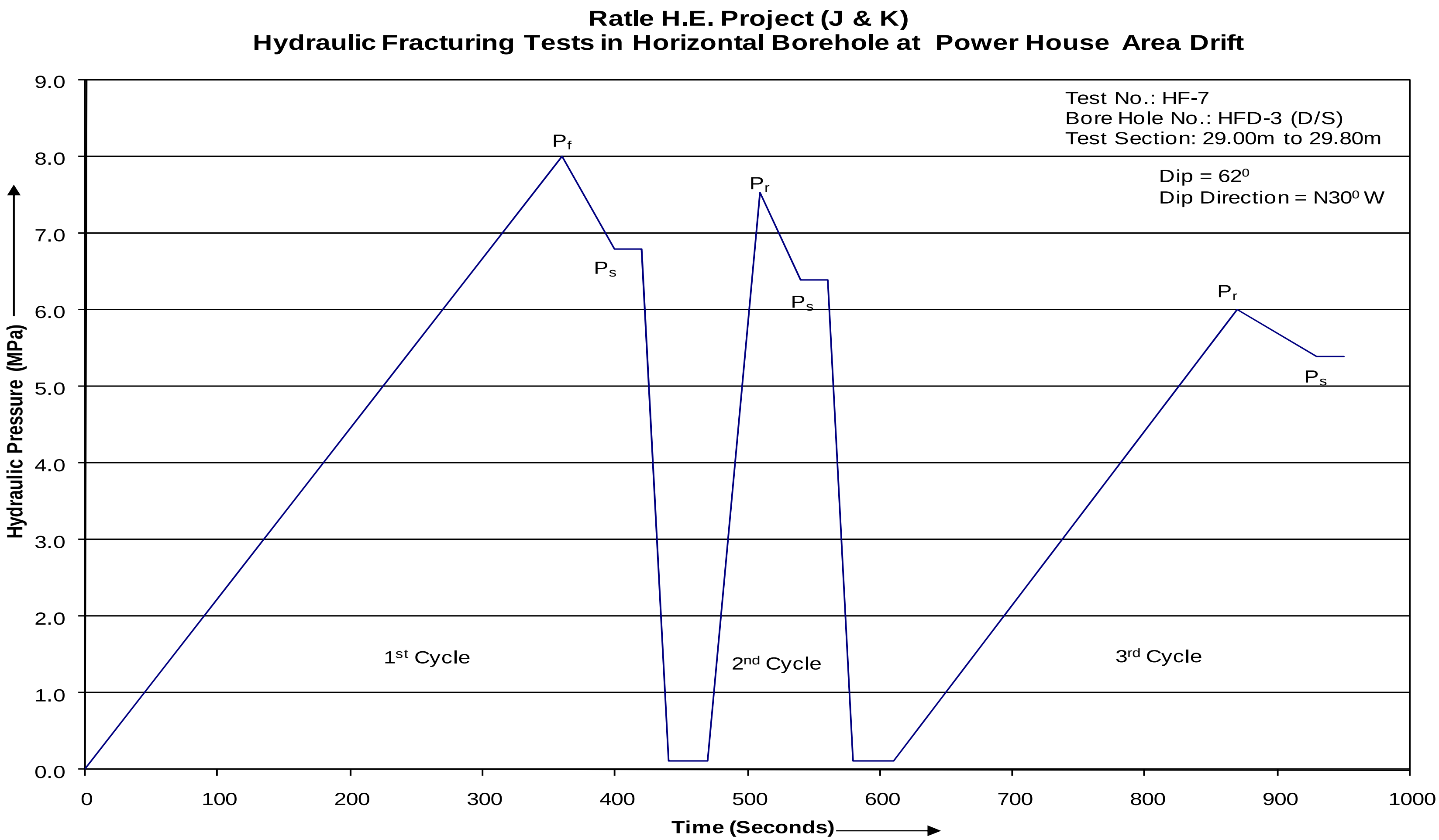

Fig. 17: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF-7.

Table -8

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF-8**
Bore Hole Direction : Horizontal **Bore Hole No. :HFD-3 (D/S)**
Test Depth :27.00 to 27.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	11.80	260	64^0 / S10^0 W
	Shut in	9.20	310	
Cycle 2	Reopening	8.40	620	
	Shut in	6.40	640	
Cycle 3	Reopening	6.90	890	
	Shut in	6.20	920	

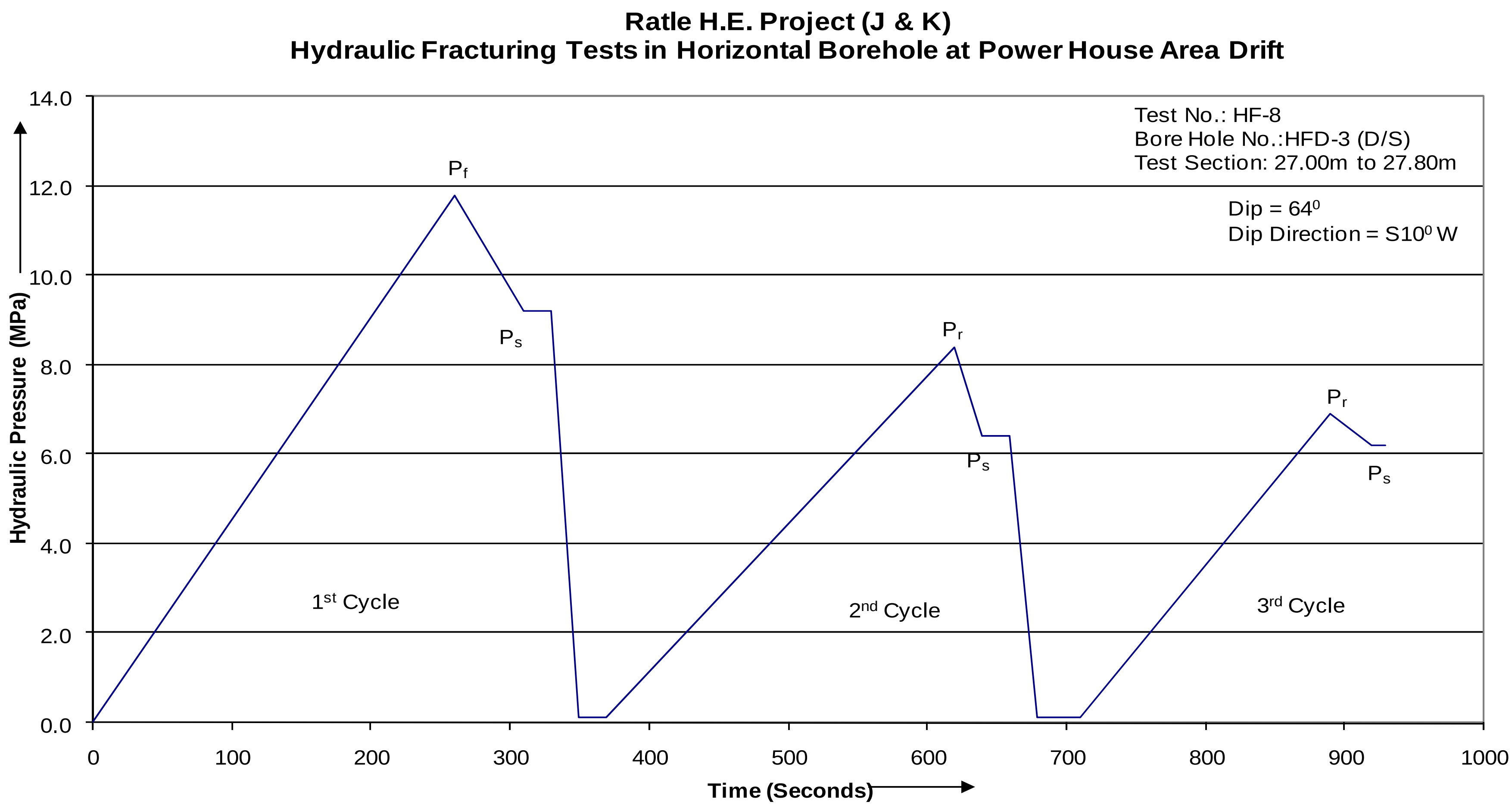

Fig. 18: Hydraulic Pressure-Time Plot of Hydraulic Fracturing for Test No. HF-8.

Table -9

Project : Ratle Hydroelectric Project (J & K)
Test Location : Power House Area Drift **Test No. : HF 9**
Bore Hole Direction : Horizontal **Bore Hole No. :HFD-3 (D/S)**
Test Depth :25.00 to 25.80 m
RD: 202.00m

Cycle No.	Hydraulic Pressure (MPa)		Time (Second)	Fracture Attributes-Dip Amount /Dip Direction
Cycle 1	Breakdown	11.50	450	60^0 / N20^0 W
	Shut in	8.20	490	
Cycle 2	Reopening	9.10	670	
	Shut in	6.20	710	
Cycle 3	Reopening	6.10	920	
	Shut in	3.87	970	

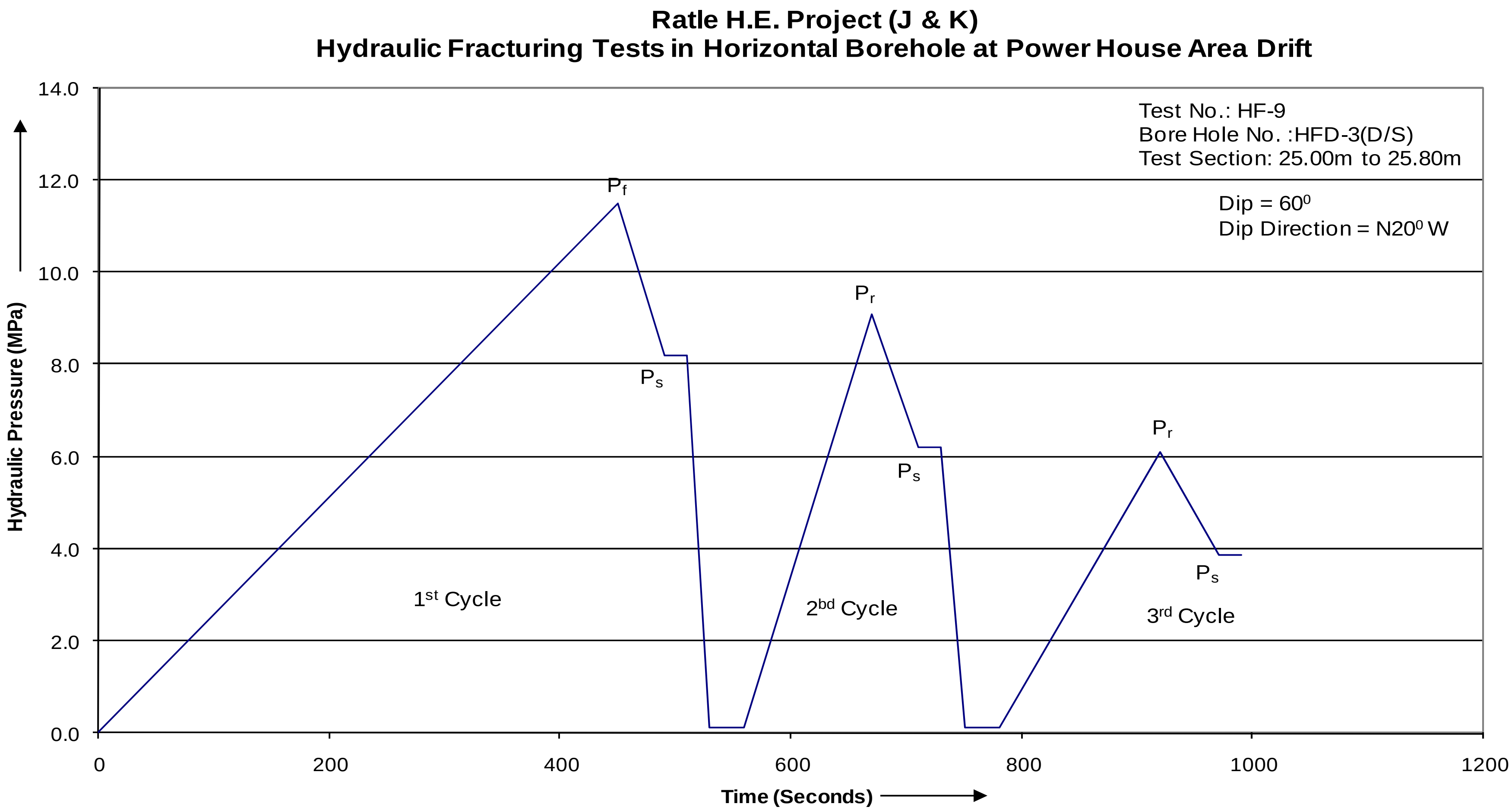

Fig. 19: Hydraulic Pressure-Time Plot of Hydraulic Fracturing Test No. HF -9.

RATLE H.E. PROJECT, J & K

Hydraulic Fracturing Tests in Power House Area Drift

Table 10: Results of Hydraulic Fracturing Tests Carried out in All Boreholes for Initial Pressurisation Cycle

Sl. No.	Test No./ Borehole No. (Type*)	R D (m)	Test Depth (m)	Test Section Depth(m)	Fracture Initiation Pressure P_f (MPa)	Tensile Strength of Rock T (MPa)**	Shut in Pressure P_s (MPa)	Max. Horizontal Stress σ_H (MPa)	Min. Horizontal Stress σ_h (MPa)	σ_v (MPa) Estimated	σ_H / σ_v cal.	σ_h / σ_v cal.	Orientation of σ_H	
													Dip Amount	Dip Direction
1	HF-1/ HFD-1 (VD)	201.00	24.60	24.20-25.00	14.76	7.50	13.18	32.28	13.18	8.25	3.91	1.60	52°	N 40^0 W
2	HF-2/HFD-1(VD)	201.00	22.60	22.20-23.00	12.15	7.50	10.21	25.98	10.21	8.25	3.15	1.24	54°	N 30^0 W
3	HF-3/HFD-1(VD)	201.00	21.40	21.00-21.80	13.24	7.50	11.15	27.71	11.15	8.25	3.36	1.35	67°	S 40^0 E
4	HF-4/HFD-2/ (H, U/S)	202.00	29.40	29.00-29.80	13.80	7.50	11.20	27.30	11.20	8.25	3.31	1.36	58°	N 50^0 W
5	HF-5/HFD-2/ (H, U/S)	202.00	28.40	28.00-28.80	11.88	7.50	9.74	24.84	9.74	8.25	3.01	1.18	56°	N 40^0 W
6	HF-6/HFD-2/ (H, U/S)	202.00	26.40	26.00-26.80	12.70	7.50	11.20	28.40	11.20	8.25	3.44	1.36	56°	N 40^0 W
7	HF7/HFD-3/ (H, D/S)	202.00	29.40	29.00-29.80	8.10	7.50	6.80	19.80	6.80	8.25	2.40	0.82	62°	N 30^0 W
8	HF8/HFD -3/ (H, D/S)	202.00	27.40	27.00-27.80	11.80	7.50	9.20	23.30	9.20	8.25	2.82	1.12	64°	S 10^0 W
9	HF-9/HFD-3/ (H, D/S)	202.00	25.40	25.00-25.80	11.50	7.50	8.20	20.60	8.20	8.25	2.50	0.99	60°	N 20^0 W
							Average	25.58	10.10	8.25	3.10	1.22		

* VD-Vertically Down
* H, U/S- Horizontal Upstream
*H, D/S- Horizontal Down stream

**Assumed tensile strength of rock based on laboratory ucs tests on rock core samples of ratle HEP adopted from an earlier report.

RATLE H.E. PROJECT, J & K

Hydraulic Fracturing Tests in Power House Area Drift

Table 11: Results of Hydraulic Fracturing Tests Carried out in Vertical Borehole No. HFD-1

2nd Cycle

Sl. No.	Test No./ Borehole No. (Type*)	RD (m)	Test Depth (m)	Test Section Depth(m)	Fracture Initiation Pressure P_f (MPa)	Fracture Reopening Pressure P_r (Mpa)	Shut in Pressure P_s (MPa)	Max. Horizontal Stress σ_H (MPa)	Min. Horizontal Stress σ_h (MPa)	σ_v (MPa) Estimated	σ_H / σ_v cal.	σ_h / σ_v cal.	Orientation of σ_H	
													Dip Amount	Dip Direction
1	HF-1/HFD-1 (VD)	201.00	24.60	24.20-25.00	14.76	13.35	11.47	21.06	11.47	8.25	2.55	1.39	52º	N 40⁰ W
2	HF-2/HFD-1 (VD)	201.00	22.60	22.20-23.00	12.15	9.87	8.14	14.55	8.14	8.25	1.76	0.99	54º	N 30⁰ W
3	HF-3/HFD-1 (VD)	201.00	21.40	21.00-21.80	13.24	11.74	9.81	17.69	9.81	8.25	2.14	1.19	67º	S 40⁰ E
							Average	17.77	9.81	8.25	2.15	1.19		

3rd Cycle

Sl. No.	Test No./ Borehole No. (Type*)	RD (m)	Test Depth (m)	Test Section Depth(m)	Fracture Initiation Pressure P_f (MPa)	Fracture Reopening Pressure P_r (Mpa)	Shut in Pressure P_s (MPa)	Max. Horizontal Stress σ_H (MPa)	Min. Horizontal Stress σ_h (MPa)	σ_v (MPa) Estimated	σ_H / σ_v cal.	σ_h / σ_v cal.	Dip Amount	Dip Direction
1	HF-1/HFD-1 (VD)	201.00	24.60	24.20-25.00	14.76	9.12	7.42	13.14	7.42	8.25	1.59	0.90	52º	N 40⁰ W
2	HF-2 /HFD-1 (VD)	201.00	22.60	22.20-23.00	12.15	8.48	6.61	11.35	6.61	8.25	1.38	0.80	54º	N 30⁰ W
3	HF-3/HFD-1 (VD)	201.00	21.40	21.00-21.80	13.24	9.18	7.84	14.34	7.84	8.25	1.74	0.95	67º	S 40⁰ E
							Average	12.94	7.29	8.25	1.57	0.88		

*VD-Vertically down

RATLE H.E. PROJECT, J & K
Hydraulic Fracturing Tests in Power House Area Drift

Table 12: Results of Hydraulic Fracturing Tests Carried out in Borehole Nos. HFD-2 & HFD-3

2nd Cycle

Sl. No.	Test No./ Borehole No. (Type*)	RD (m)	Test Depth (m)	Test Section Depth(m)	Fracture Initiation Pressure P_f (MPa)	Fracture Reopening Pressure P_r (Mpa)	Shut in Pressure P_s (MPa)	Max. Horizontal Stress σ_H (MPa)	Min. Horizontal Stress σ_h (MPa)	σ_v (MPa) estimated	σ_H / σ_v cal.	σ_h / σ_v cal.	Orientation of σ_H	
													Dip Amount	Dip Direction
1	HF-4/HFD-2 (H-U/S)	202.00	29.40	29.00-29.80	13.80	11.80	8.20	12.80	8.20	8.25	1.55	0.99	58°	N 50° W
2	HF-5/HFD-2 (H-U/S)	202.00	28.40	28.00-28.80	11.88	10.26	8.41	14.97	8.41	8.25	1.81	1.02	56°	N 40° W
3	HF-6/HFD-2 (H-U/S)	202.00	26.40	26.00-26.80	12.70	10.80	9.20	16.80	9.20	8.25	2.04	1.12	56°	N 40° W
4	HF-7/HFD-3 (H-D/S)	202.00	29.40	29.00-29.80	8.10	7.54	6.40	11.66	6.40	8.25	1.41	0.78	62°	N 30° W
5	HF-8/HFD-3 (H-D/S)	202.00	27.40	27.00-27.80	11.80	8.40	6.40	10.80	6.40	8.25	1.31	0.78	64°	S 10° W
6	HF-9/HFD-3 (H-D/S)	202.00	25.40	25.00-25.80	11.50	9.10	6.20	9.50	6.20	8.25	1.15	0.75	60°	N 20° W
							Average	12.76	7.47	8.25	1.55	0.91		

3rd Cycle

1	HF-4/HFD-2 (H-U/S)	202.00	29.40	29.00-29.80	13.80	9.10	6.34	9.92	6.34	8.25	1.20	0.77	58°	N 50° W
2	HF-5/HFD-2 (H-U/S)	202.00	28.40	28.00-28.80	11.88	8.87	6.74	11.35	6.74	8.25	1.38	0.82	56°	N 40° W
3	HF-6/HFD-2 (H-U/S)	202.00	26.40	26.00-26.80	12.70	8.70	7.20	12.90	7.20	8.25	1.56	0.87	56°	N 40° W
4	HF-7/HFD-3 (H-D/S)	202.00	29.40	29.00-29.80	8.10	6.00	5.40	10.20	5.40	8.25	1.24	0.65	62°	N 30° W
5	HF-8/HFD-3 (H-D/S)	202.00	27.40	27.00-27.80	11.80	6.90	6.20	11.70	6.20	8.25	1.42	0.75	64°	S 10° W
6	HF-9/HFD-3 (H-D/S)	202.00	25.40	25.00-25.80	11.50	6.10	3.87	5.51	3.87	8.25	0.67	0.47	60°	N 20° W
							Average	10.26	5.96	8.25	1.25	0.72		

*H-U/S- Horizontal Up Stream
*H-D/S- Horizontal Down Stream

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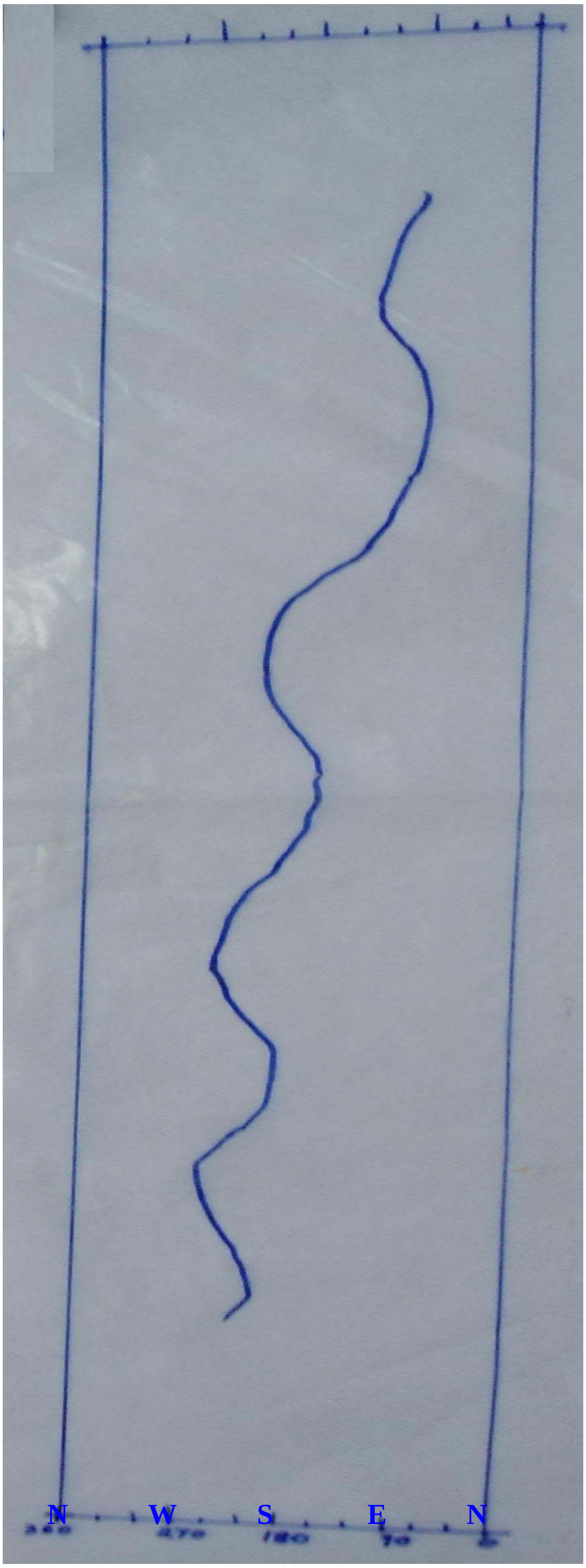
Hydro Fracturing Test

Borehole No.HFD-2; Test No. HF-6; RD: 202.00m; Type: Horizontal (U/S);
Test Section: 26.00m to 26.80m

26.00m



Depth

26.80m

Fig.23: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

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Hydro Fracturing Test

Borehole No.HFD-2; Test No. HF-5; RD: 202.00m; Type: Horizontal (U/S);
Test Section: 28.00m to 28.80m

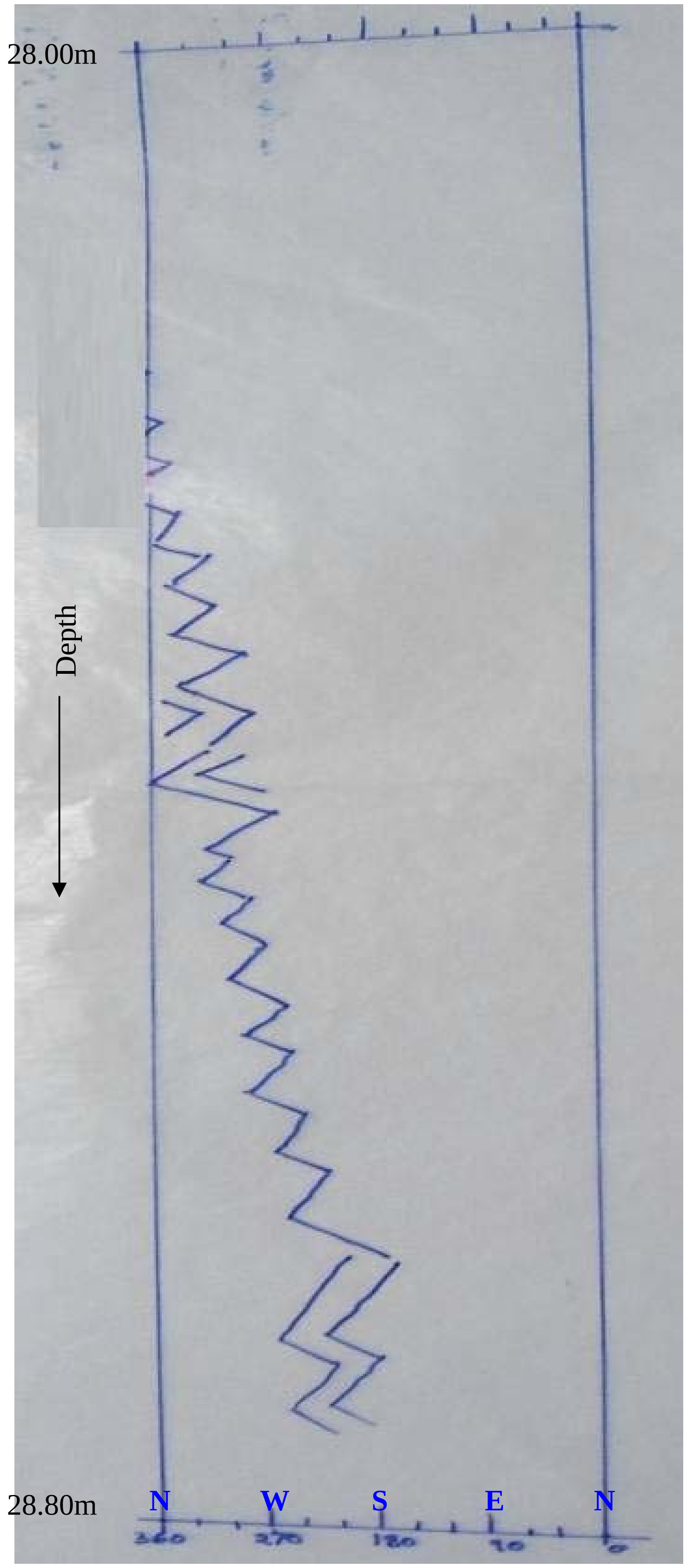


Fig.22: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

Ratle H.E. Project, J & K

Hydro Fracturing Test

Borehole No.HFD-2; Test No. HF-4; RD: 202.00m; Type: Horizontal (U/S);
Test Section: 29.00m to 29.80m

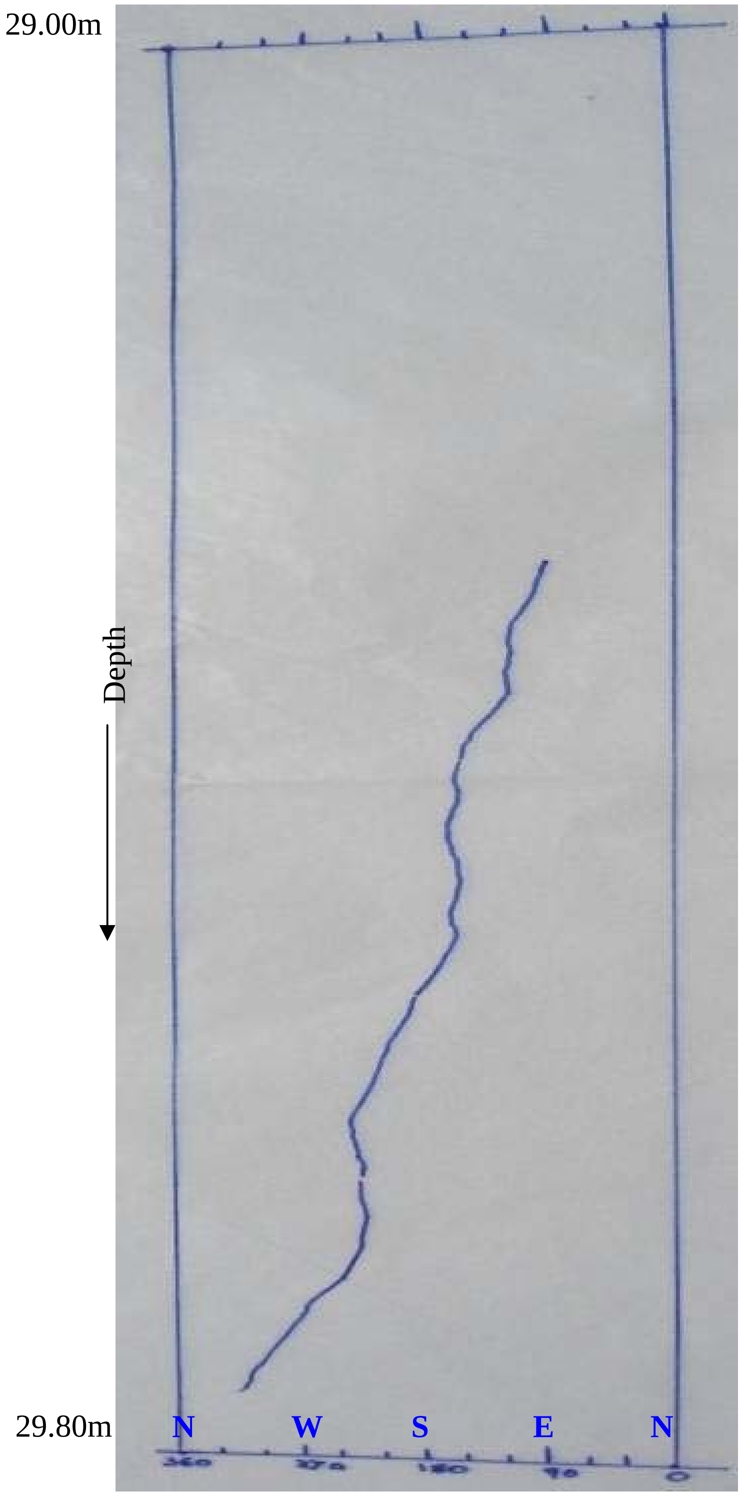


Fig.21: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

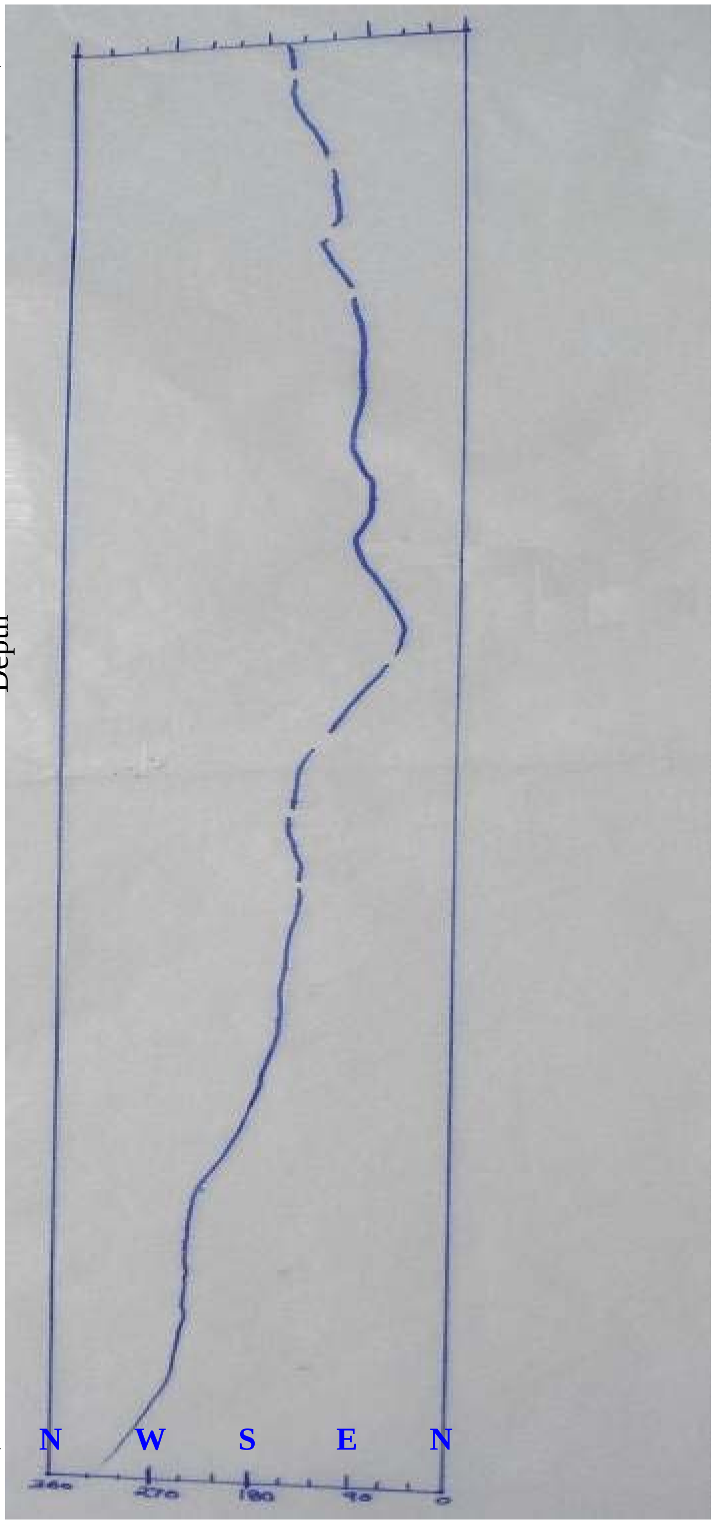
Ratle H.E. Project, J & K

Hydro Fracturing Test

Borehole No.HFD-3; Test No. HF-7; RD: 202.00m; Type: Horizontal (D/S);
Test Section: 29.00m to 29.80m

29.00m

Depth



29.80m

Fig.24: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

Ratle H.E. Project, J & K

Hydro Fracturing Test

Borehole No.HFD-3; Test No. HF-9; RD: 202.00m; Type: Horizontal (D/S);
Test Section: 25.00m to 25.80m

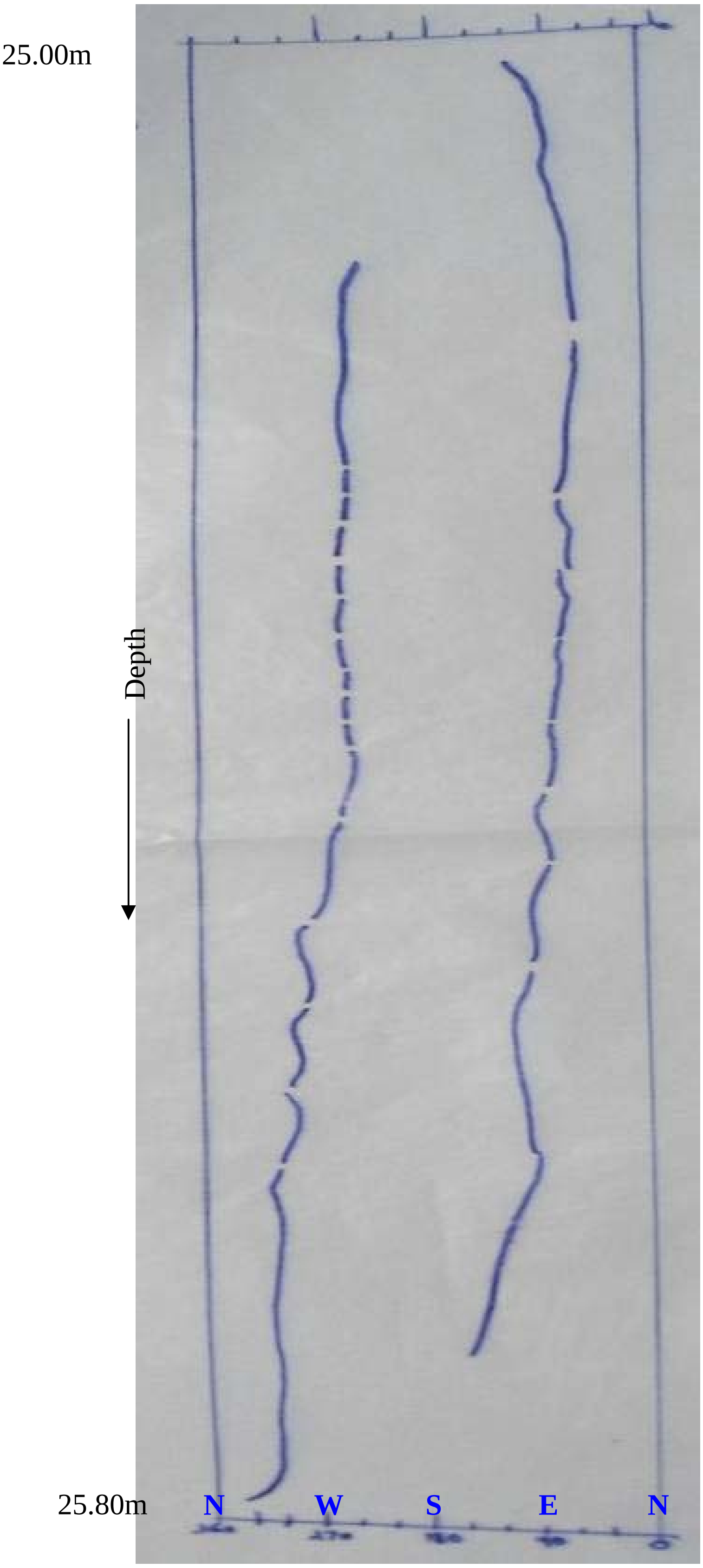


Fig.25: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

Ratle H.E. Project, J & K

Hydro Fracturing Test

Borehole No.HFD-1; Test No. HF-1; RD: 201.00m; Type: Vertical;
Test Section: 24.20m to 25.00m

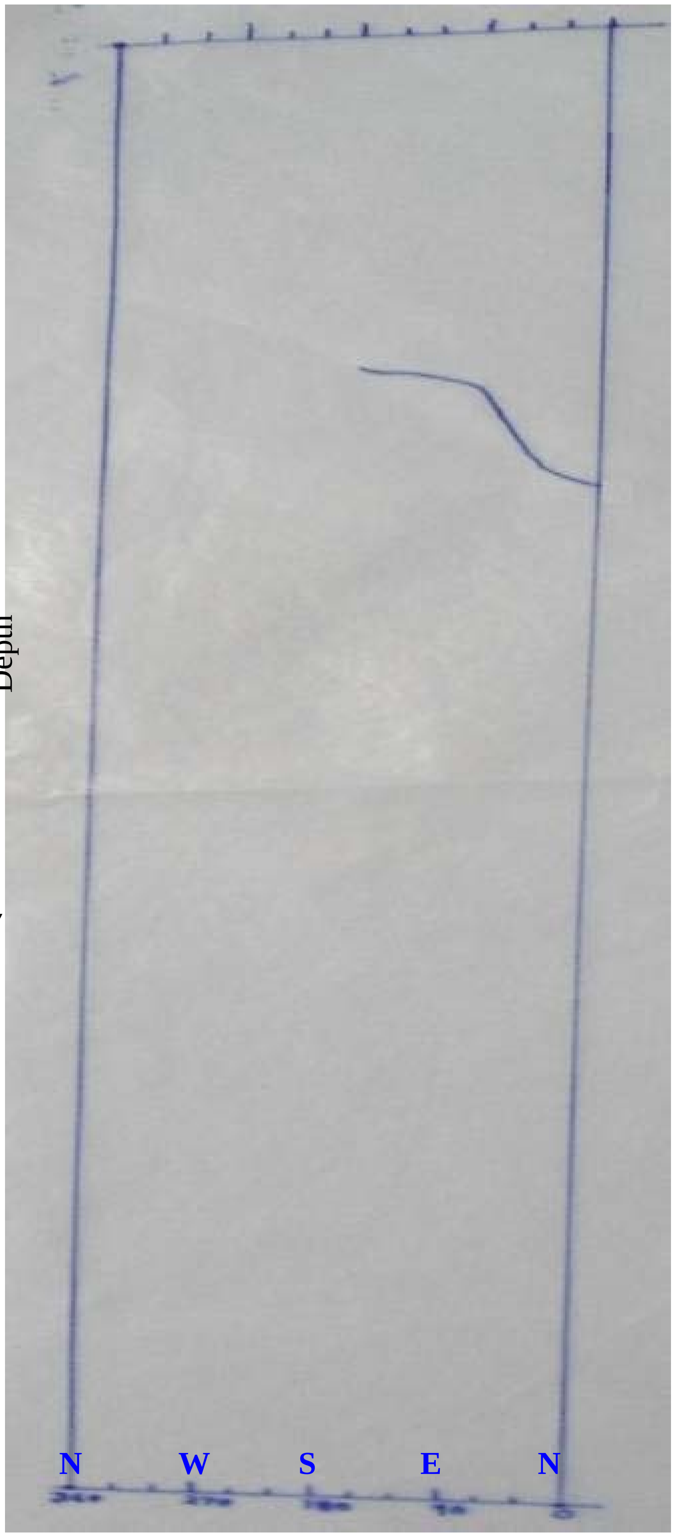


Fig.20: An Actual Impression & Packer Test Record Showing HF Trace on the Borehole Wall.

RATLE H.E. PROJECT (J & K)

TABLE 13: RESULTS OF PLATE LOAD TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift) **RD: 192.00m**
Test No. : PJT-1 (Horizontal) (D/S)
Depth of Anchors

Anchor 1: 0.50 m
Anchor 2: 1.00 m
Anchor 3: 2.00 m
Anchor 4: 3.00 m
Anchor 5: 6.00 m

Load	Stress (MPa)	**Anchor 1**		**Anchor 2**		**Anchor 3**		**Anchor 4**		**Anchor 5**	
(KN)		Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)
0	0.00	0.001	0.000	0.002	0.000	0.009	0.000	0.002	0.000	0.002	0.000
400	0.51	0.005	0.004	0.004	0.002	0.012	0.003	0.004	0.002	0.002	0.000
0	0.00	0.004	0.003	0.003	0.001	0.010	0.001	0.003	0.001	0.002	0.000
800	1.02	0.012	0.011	0.009	0.007	0.015	0.006	0.008	0.006	0.002	0.000
0	0.00	0.006	0.005	0.005	0.003	0.013	0.004	0.005	0.003	0.002	0.000
1200	1.54	0.026	0.025	0.019	0.017	0.024	0.015	0.012	0.010	0.003	0.001
0	0.00	0.018	0.017	0.012	0.010	0.018	0.009	0.008	0.006	0.003	0.001
1600	2.05	0.048	0.047	0.038	0.036	0.033	0.024	0.024	0.022	0.003	0.001
0	0.00	0.040	0.039	0.029	0.027	0.030	0.021	0.015	0.013	0.003	0.001
2000	2.56	0.170	0.169	0.120	0.118	0.066	0.057	0.029	0.027	0.003	0.001
0	0.00	0.102	0.101	0.047	0.045	0.050	0.041	0.019	0.017	0.002	0.000

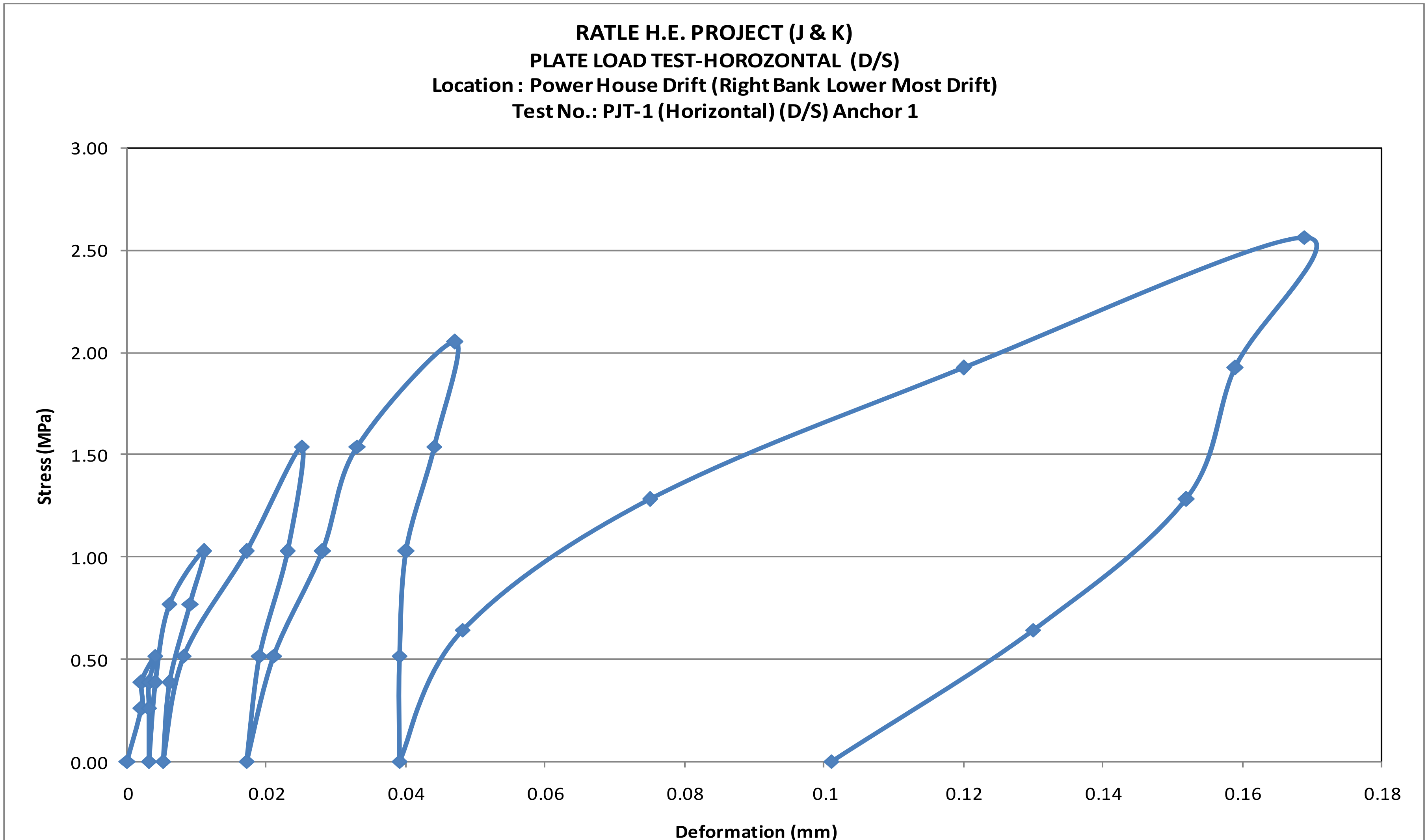


Fig. 29: Stress V/s Deformation Curve

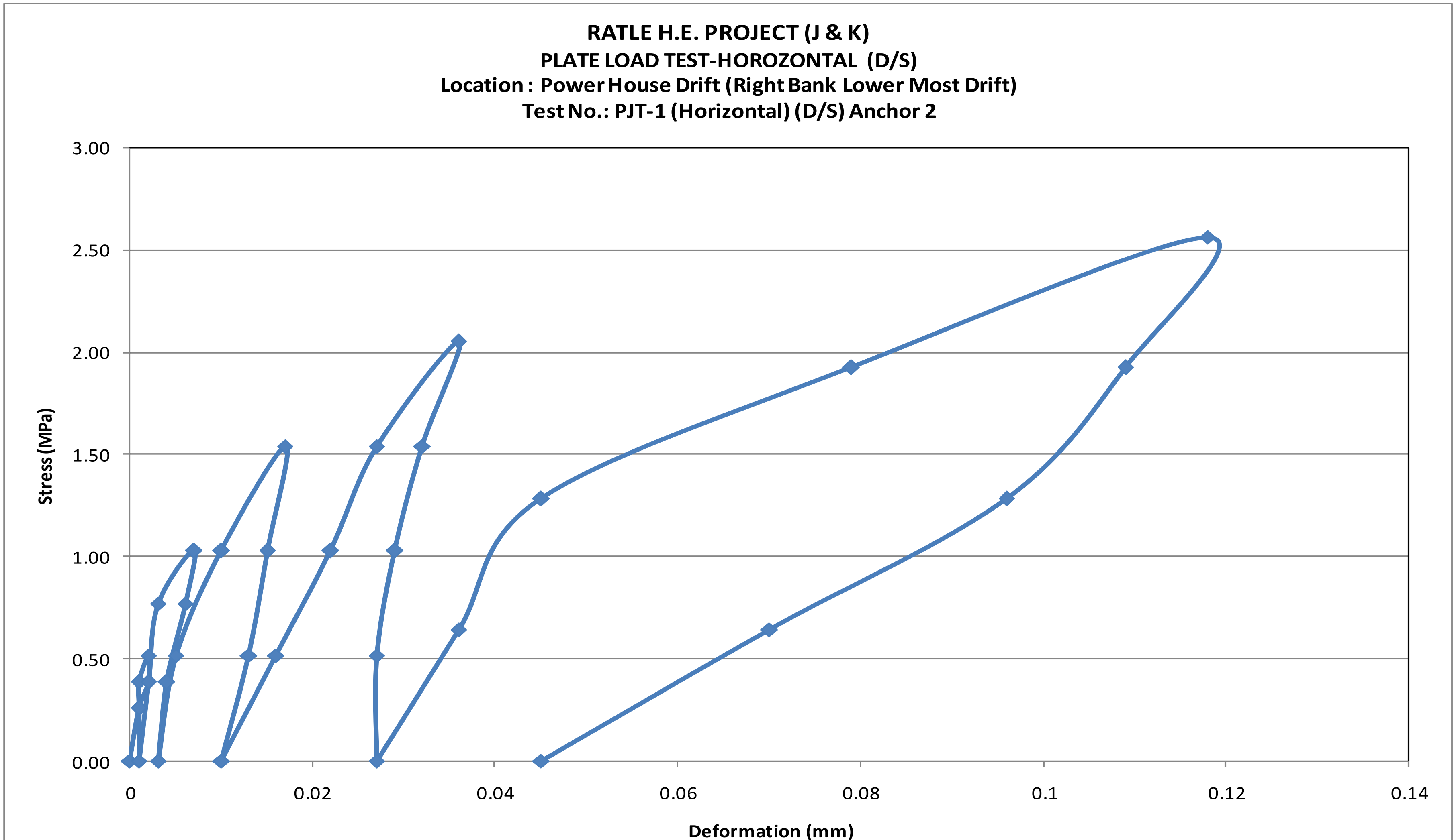


Fig. 30: Stress V/s Deformation Curve

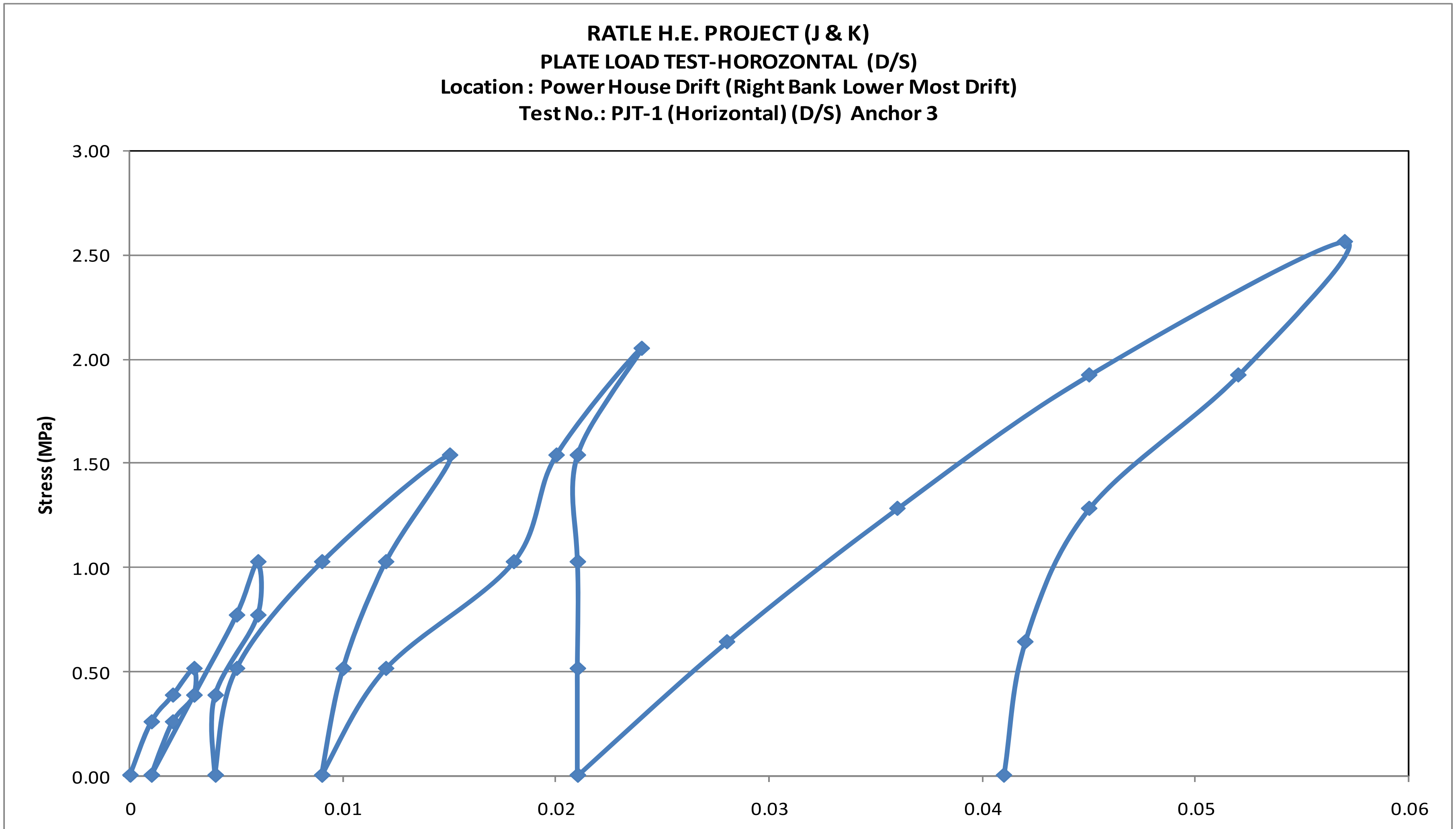


Fig. 31: Stress V/s Deformation Curve

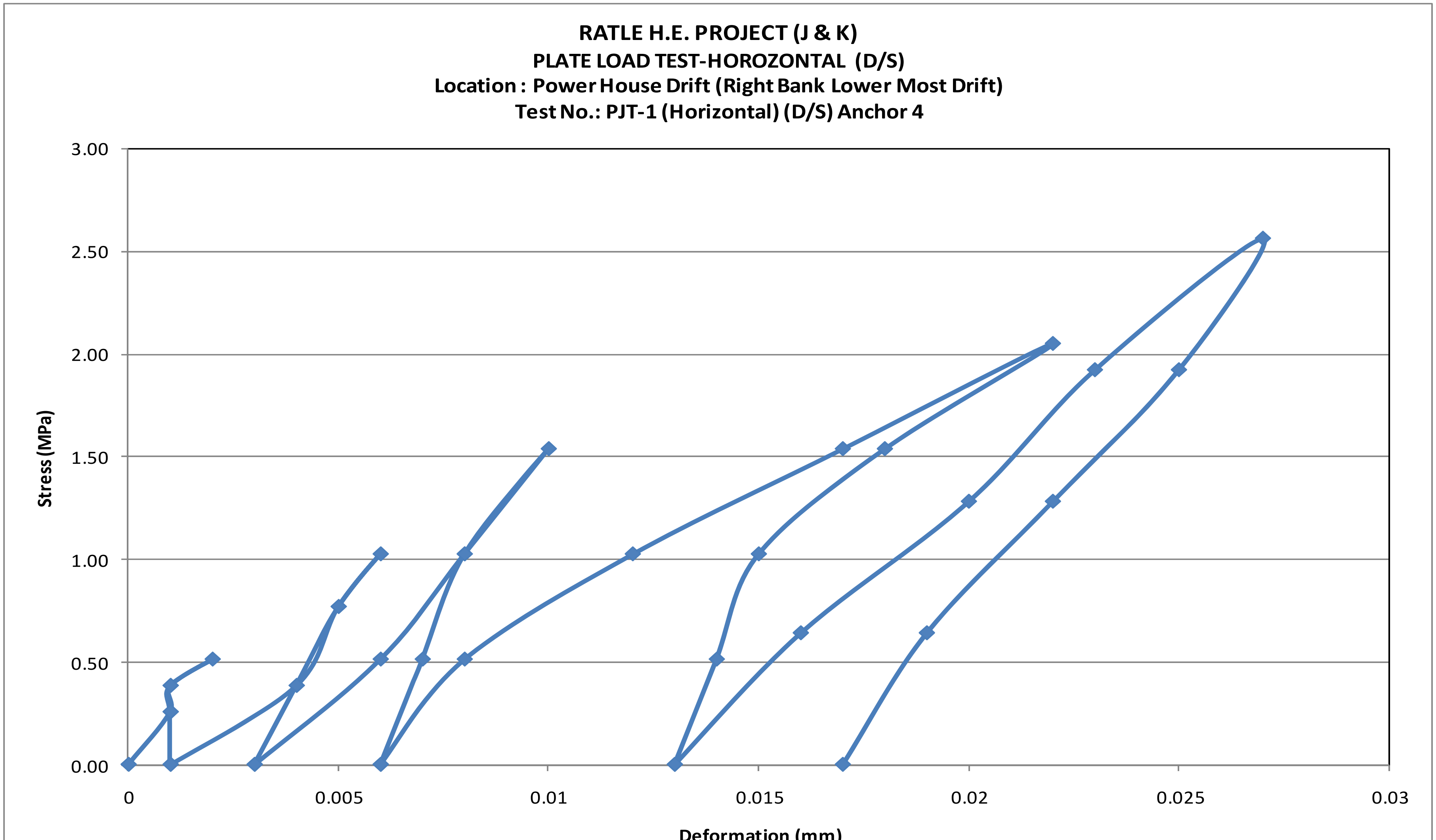


Fig. 32: Stress V/s Deformation Curve

RATLE H.E. PROJECT (J & K)

TABLE 14 : RESULTS OF PLATE LOADING TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift)
RD : 192.00m
Test No.t : PJT-1 (Horizontal) (D/S)
Maximum Bearing Pressure: 2.56MPa
Outer Radius of Circular Plate:500mm; Radius of Central Hole:39mm

Modulii of Deformation Based on Horizontal (D/S) Anchors

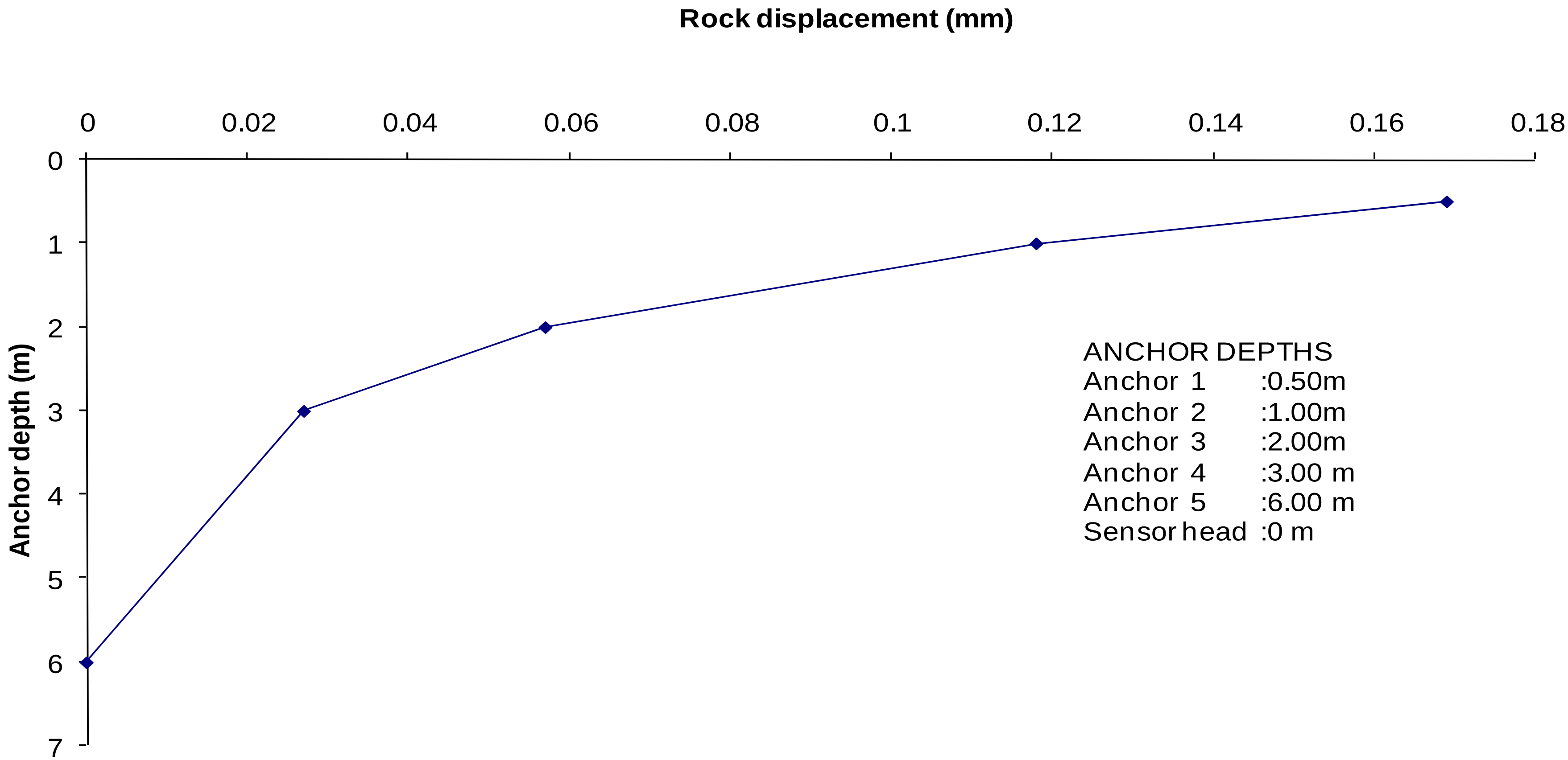
Rock Deformation Data

Designation of Anchor	Depth of Anchor (cm)	Rock Deformation (mm)
1	50	0.169
2	100	0.118
3	200	0.057
4	300	0.027
5	600	-

Modulii of Deformation

Thickness of Layer Below Loaded Rock Surface (cm)	Deformation Modulus (GPa)
50-100	10.83
100-200	6.80
200-300	5.21
300-600	6.00

RATLE H.E. PROJECT (J & K)
Fig. 33: Uniaxial Rock Displacement Vs Depth Referenced to Deepest Anchor at 2.56 Mpa Bearing Pressure for Horizontal (D/S) Anchors
Location: Power House Drift (Right Bank Lower Most Drift); RD: 192.00m



RATLE H.E. PROJECT (J & K)

TABLE 15: RESULTS OF PLATE LOAD TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift) **RD: 192.00m**
Test No. : PJT-1 (Horizontal) (U/S)
Depth of Anchors

Anchor 1: 0.50 m
Anchor 2: 1.00 m
Anchor 3: 2.00 m
Anchor 4: 3.00 m
Anchor 5: 6.00 m

Load	Stress	Anchor 1		Anchor 2		Anchor 3		Anchor 4		Anchor 5	
(KN)	(MPa)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)
0	0.00	0.002	0.000	0.002	0.000	0.002	0.000	0.002	0.000	0.002	0.000
400	0.51	0.003	0.001	0.006	0.004	0.004	0.002	0.003	0.001	0.002	0.000
0	0.00	0.003	0.001	0.003	0.001	0.003	0.001	0.002	0.000	0.002	0.000
800	1.02	0.012	0.010	0.010	0.008	0.008	0.006	0.005	0.003	0.002	0.000
0	0.00	0.008	0.006	0.006	0.004	0.006	0.004	0.003	0.001	0.002	0.000
1200	1.54	0.029	0.027	0.017	0.015	0.012	0.010	0.008	0.006	0.002	0.000
0	0.00	0.020	0.018	0.012	0.010	0.009	0.007	0.005	0.003	0.002	0.000
1600	2.05	0.047	0.045	0.036	0.034	0.023	0.021	0.009	0.007	0.002	0.000
0	0.00	0.035	0.033	0.026	0.024	0.017	0.015	0.007	0.005	0.002	0.000
2000	2.56	0.121	0.119	0.066	0.064	0.031	0.029	0.016	0.014	0.003	0.001
0	0.00	0.102	0.100	0.047	0.045	0.025	0.023	0.010	0.008	0.002	0.000

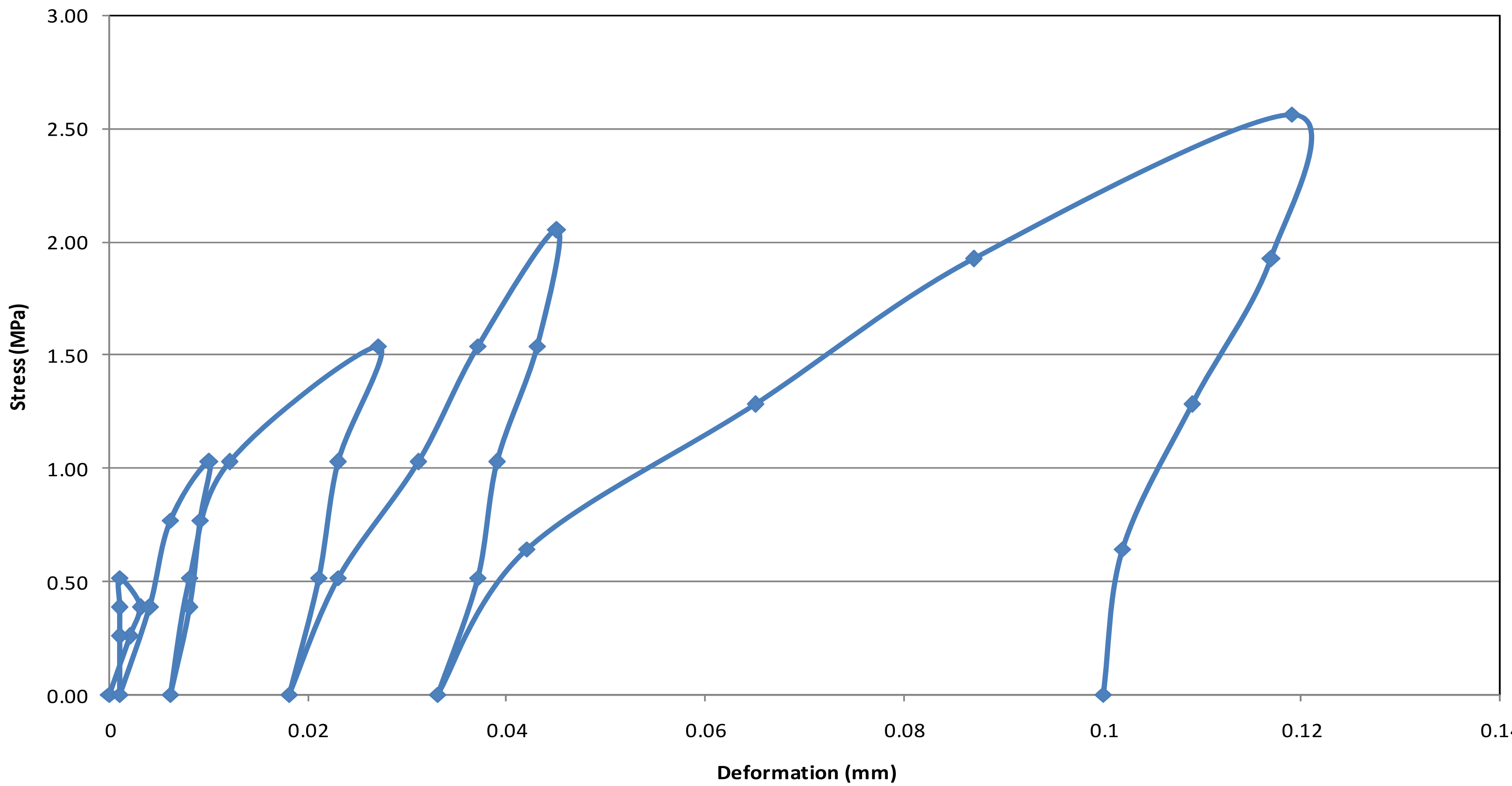


Fig.34: Stress V/s Deformation Curve

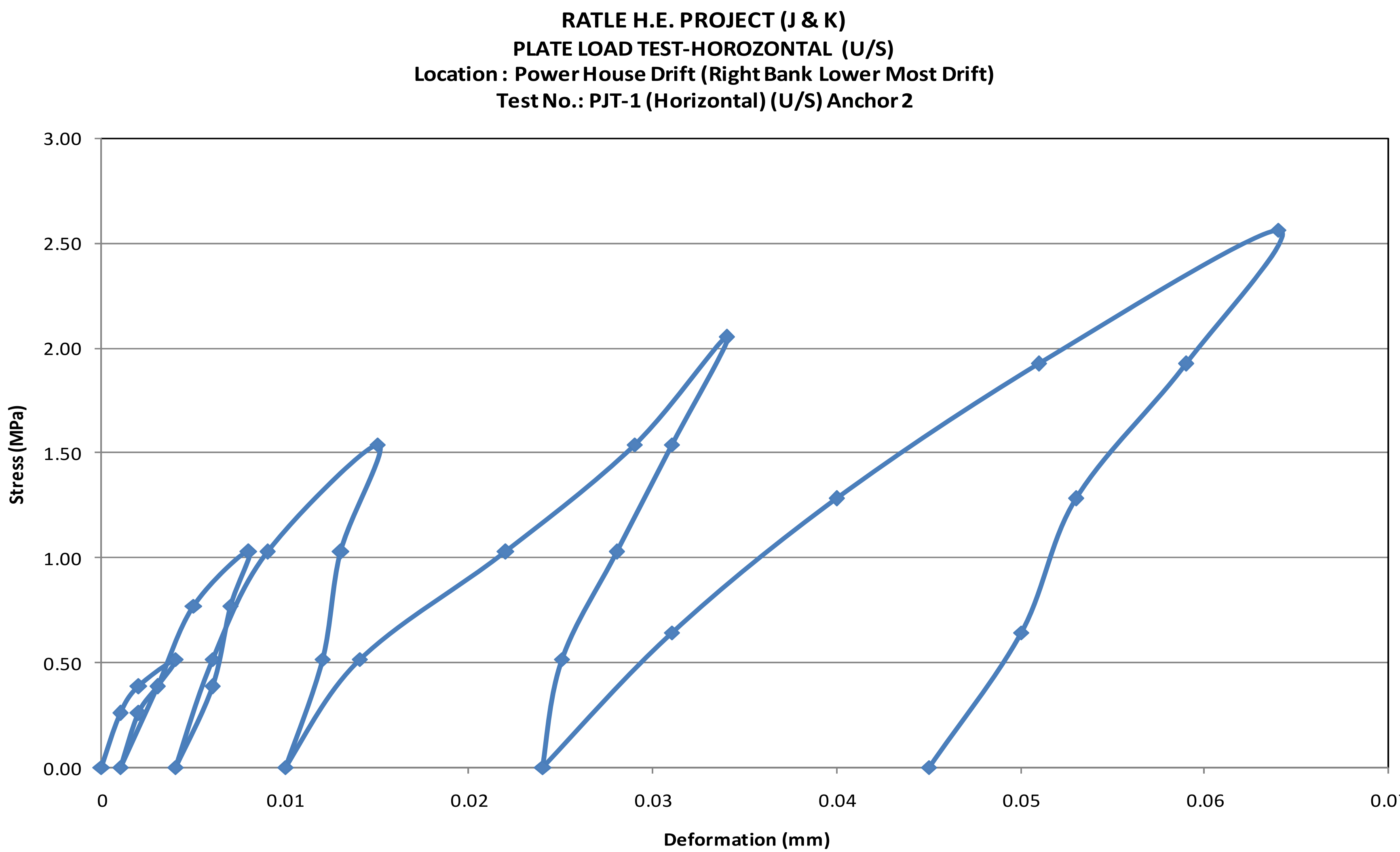


Fig. 35: Stress V/s Deformation Curve

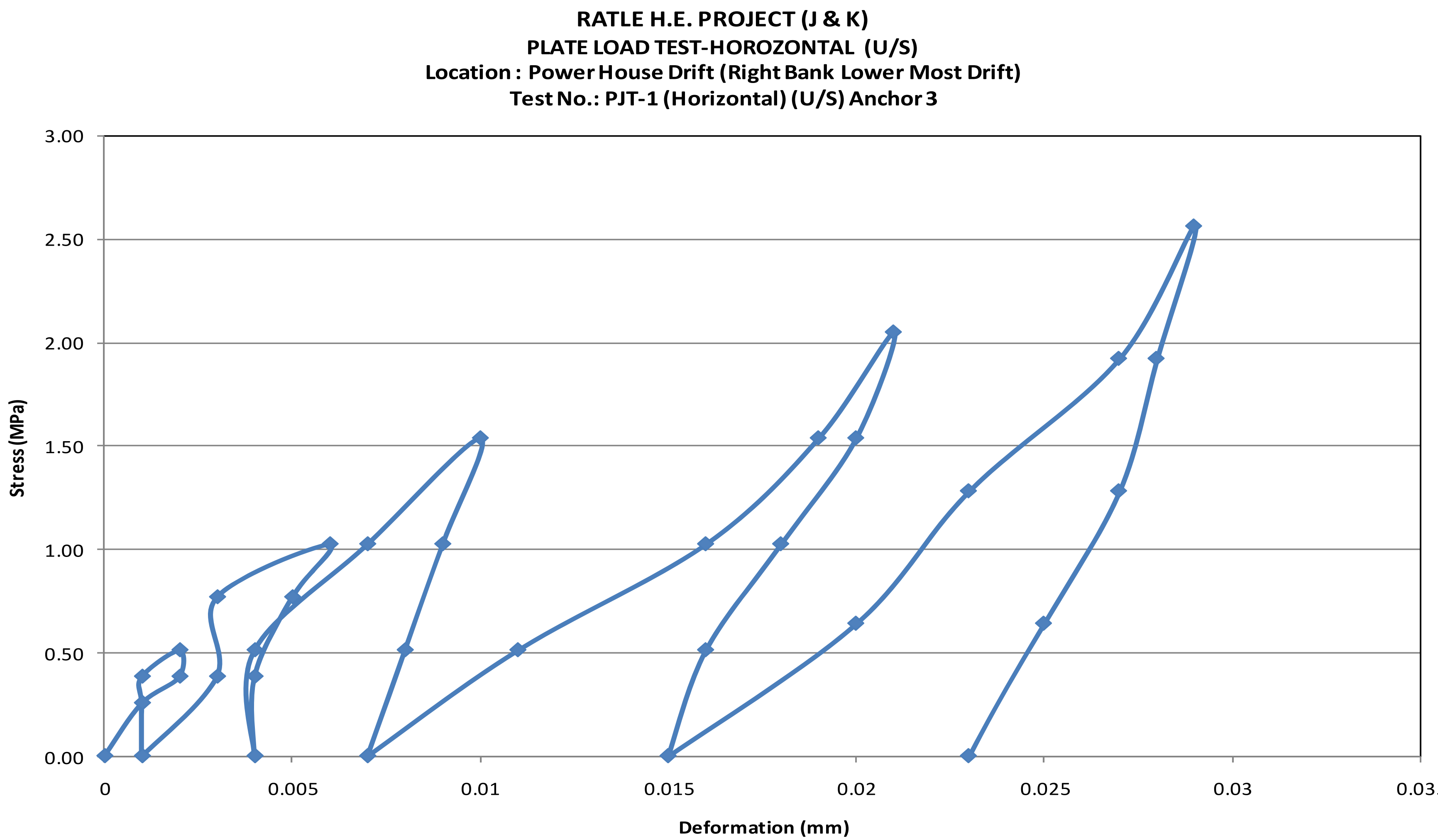


Fig.36: Stress V/s Deformation Curve

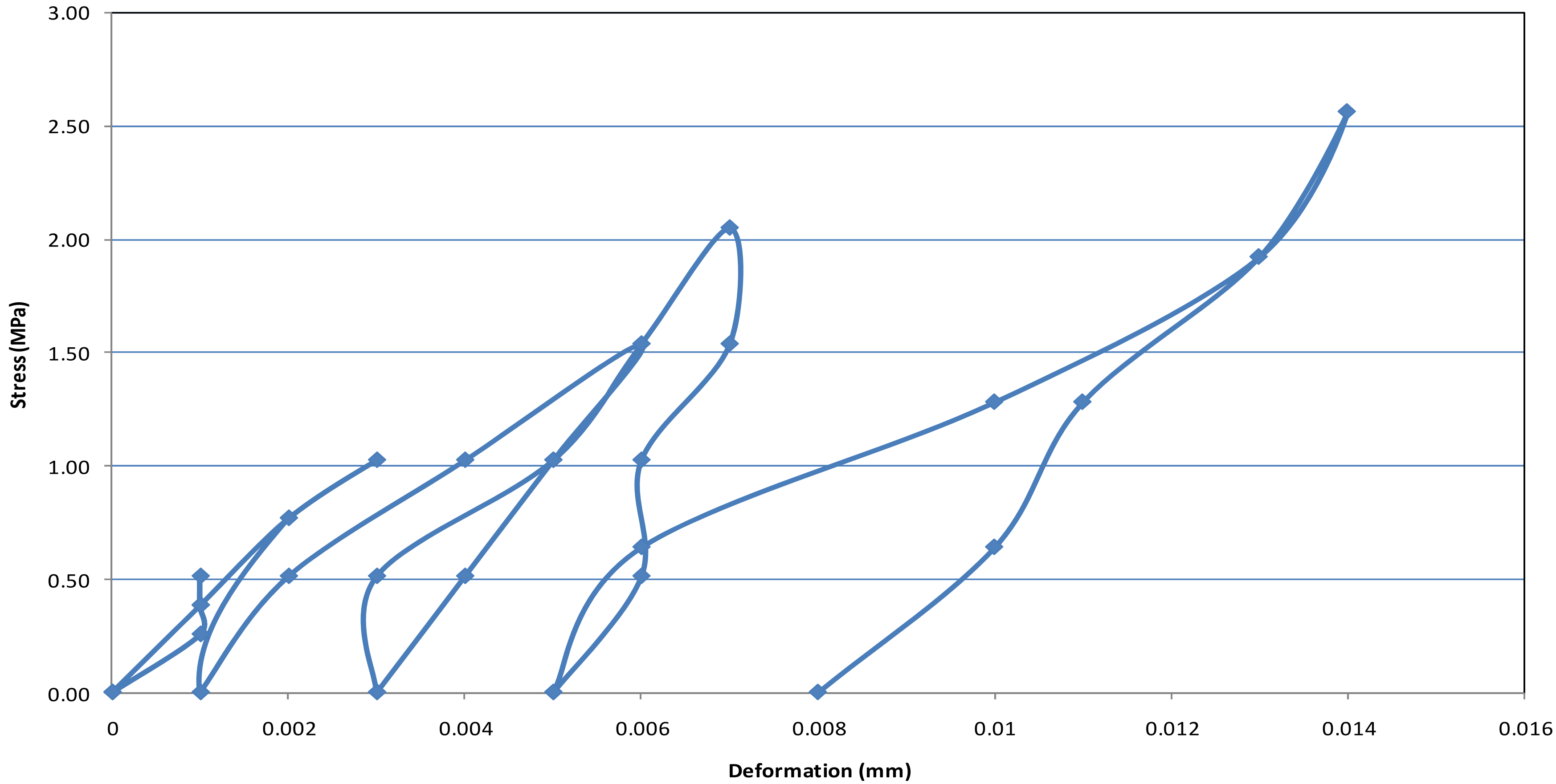


Fig. 37: Stress V/s Deformation Curve

RATLE H.E. PROJECT (J & K)

TABLE 16 : RESULTS OF PLATE LOADING TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift)
RD : 192.00m
Test No.t : PJT-1 (Horizontal) (U/S)
Maximum Bearing Pressure: 2.56MPa
Outer Radius of Circular Plate:500mm; Radius of Central Hole:39mm

<u>Modulii of Deformation Based on Horizontal (U/S) Anchors</u>

Rock Deformation Data

Designation of Anchor	Depth of Anchor (cm)	Rock Deformation (mm)
1	50	0.119
2	100	0.064
3	200	0.029
4	300	0.014
5	600	-

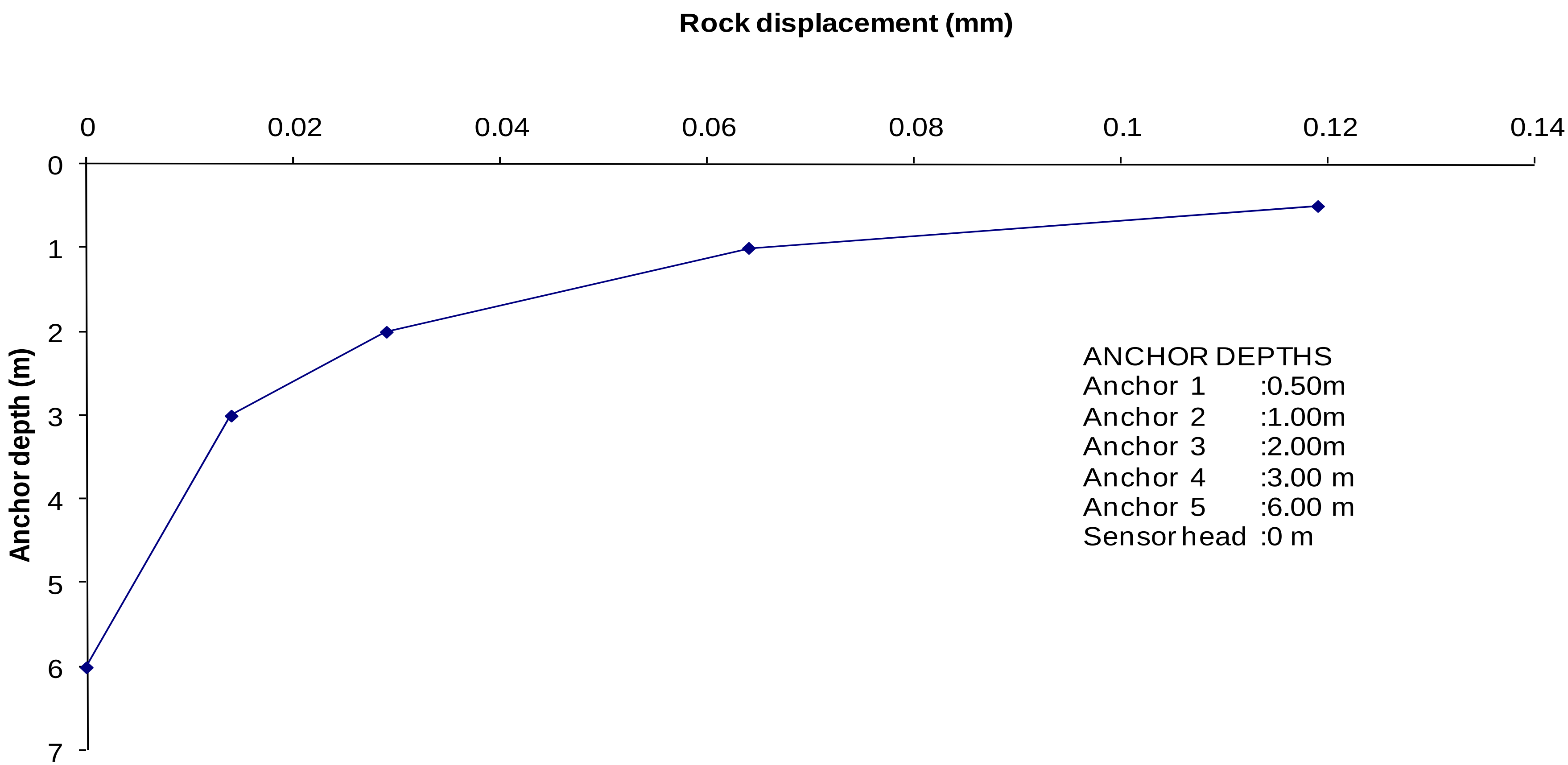
<u>Modulii of Deformation</u>

Thickness of Layer Below Loaded Rock Surface (cm)	Deformation Modulus (GPa)
50-100	10.04
100-200	11.85
200-300	10.42
300-600	11.58

RATLE H.E. PROJECT (J & K)

Fig. 38: Uniaxial Rock Displacement Vs Depth Referenced to Deepest Anchor at 2.56 Mpa Bearing Pressure for Horizontal (U/S) Anchors

Location: Power House Drift (Right Bank Lower Most Drift); RD: 192.00m



RATLE H.E. PROJECT (J & K)

TABLE 17: RESULTS OF PLATE LOAD TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift) **RD: 195.00m**
Test No. : PJT-2 (Vertical) (Upward)
Depth of Anchors

Anchor 1: 0.50 m
Anchor 2: 1.00 m
Anchor 3: 2.00 m
Anchor 4: 3.00 m
Anchor 5: 6.00 m

Load	Stress (MPa)	Anchor 1		Anchor 2		Anchor 3		Anchor 4		Anchor 5	
(KN)		Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)
0	0.00	0.002	0.000	0.002	0.000	0.028	0.000	0.002	0.000	0.012	0.000
400	0.51	0.007	0.005	0.007	0.005	0.031	0.003	0.005	0.003	0.012	0.000
0	0.00	0.004	0.002	0.004	0.002	0.029	0.001	0.004	0.002	0.012	0.000
800	1.02	0.023	0.021	0.019	0.017	0.039	0.011	0.009	0.007	0.012	0.000
0	0.00	0.015	0.013	0.014	0.012	0.035	0.007	0.006	0.004	0.012	0.000
1200	1.54	0.039	0.037	0.027	0.025	0.044	0.016	0.012	0.010	0.012	0.000
0	0.00	0.030	0.028	0.020	0.018	0.037	0.009	0.009	0.007	0.012	0.000
1600	2.05	0.085	0.083	0.057	0.055	0.053	0.025	0.018	0.016	0.012	0.000
0	0.00	0.073	0.071	0.044	0.042	0.041	0.013	0.013	0.011	0.013	0.001
2000	2.56	0.144	0.142	0.086	0.084	0.057	0.029	0.022	0.020	0.013	0.001
0	0.00	0.105	0.103	0.075	0.073	0.046	0.018	0.016	0.014	0.012	0.000

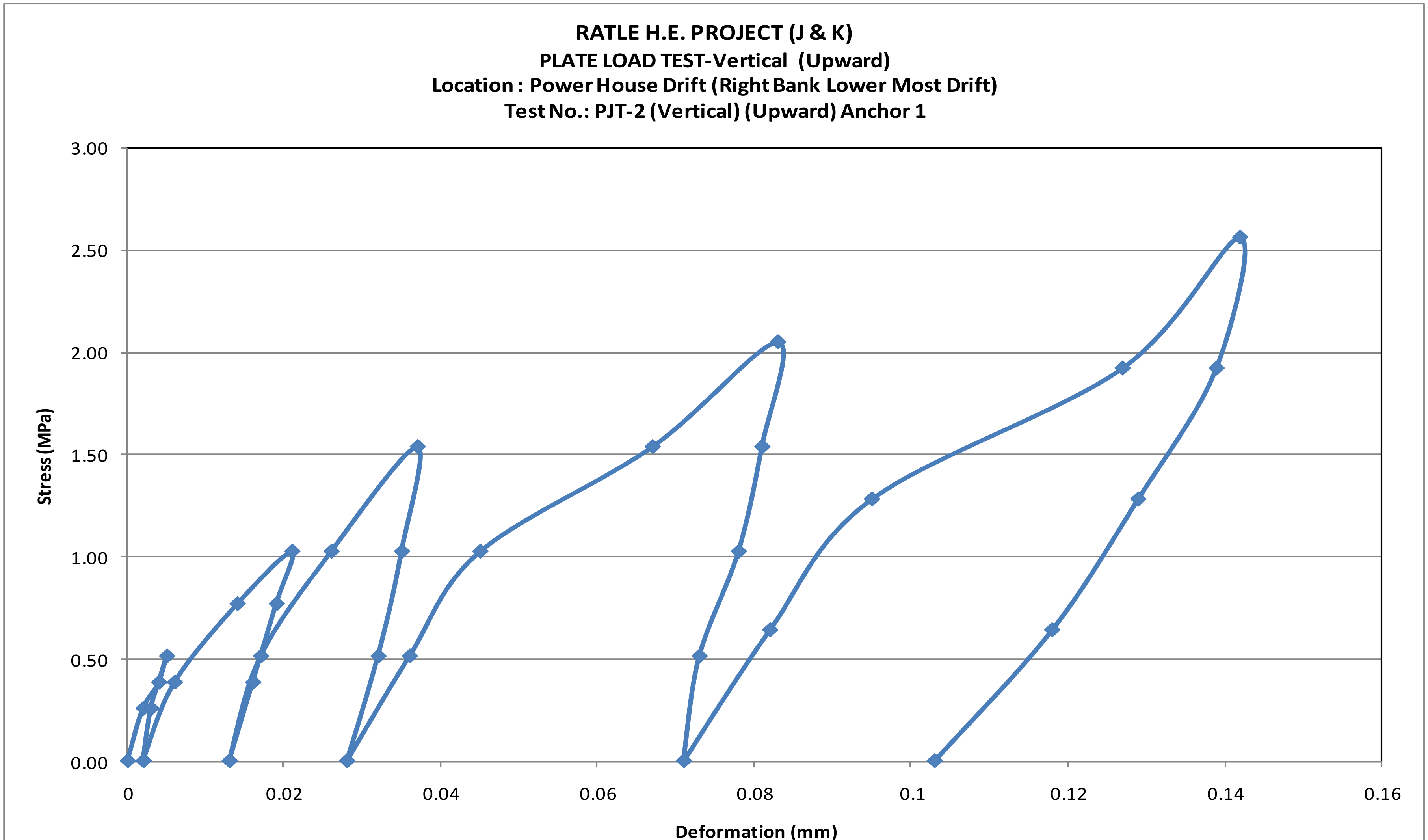


Fig. 39: Stress V/s Deformation Curve

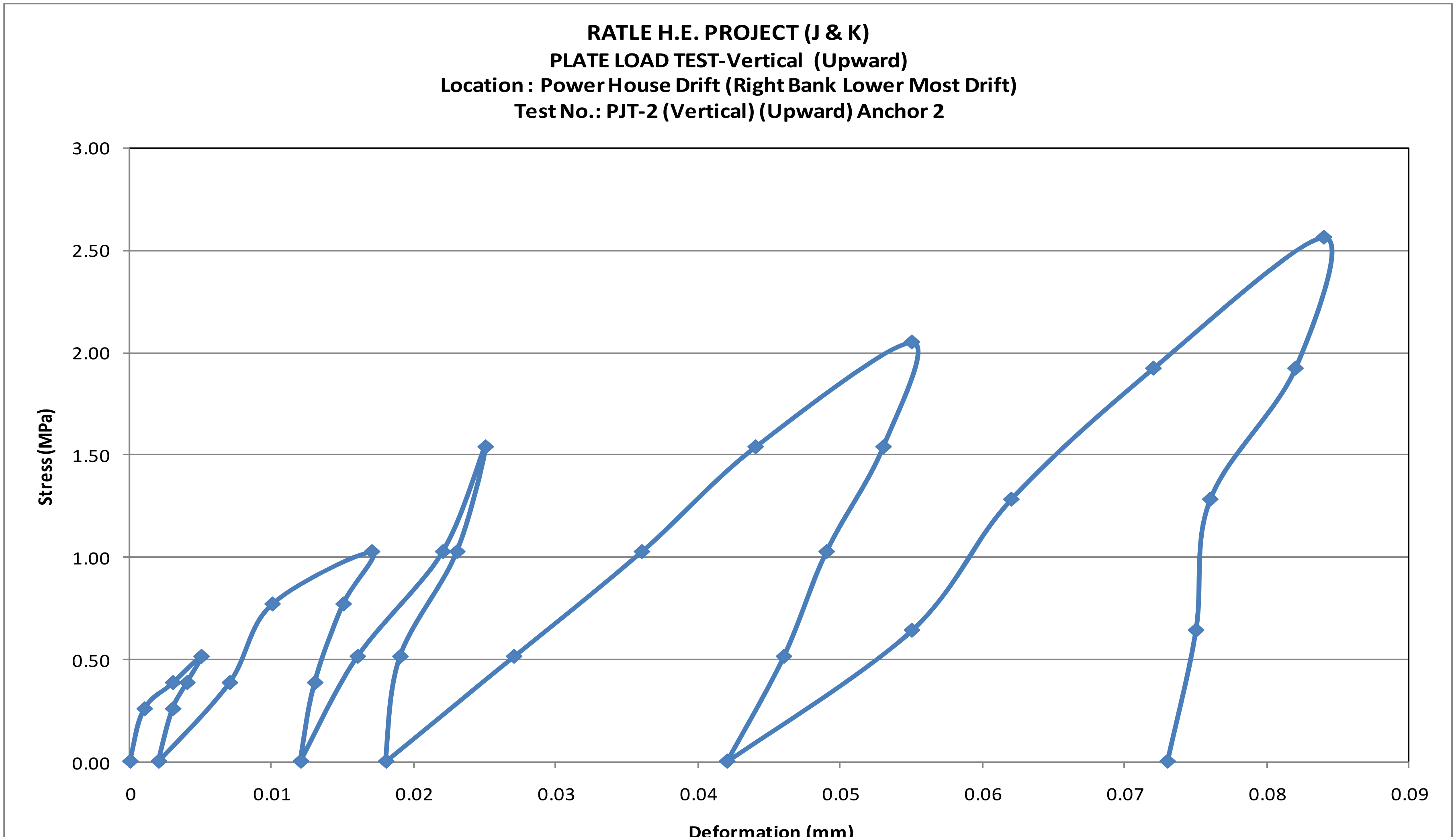


Fig. 40: Stress V/s Deformation Curve

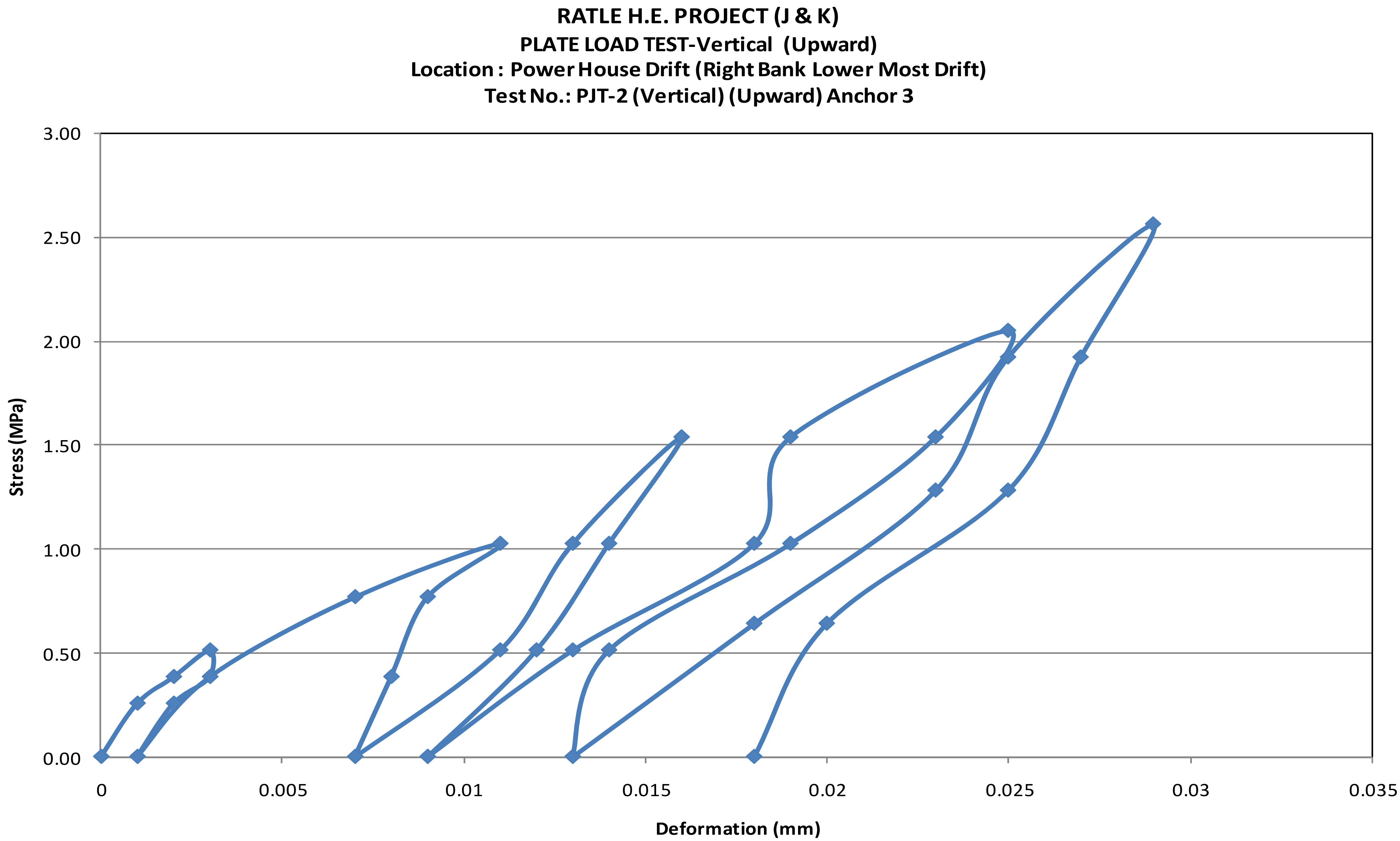


Fig. 41: Stress V/s Deformation Curve

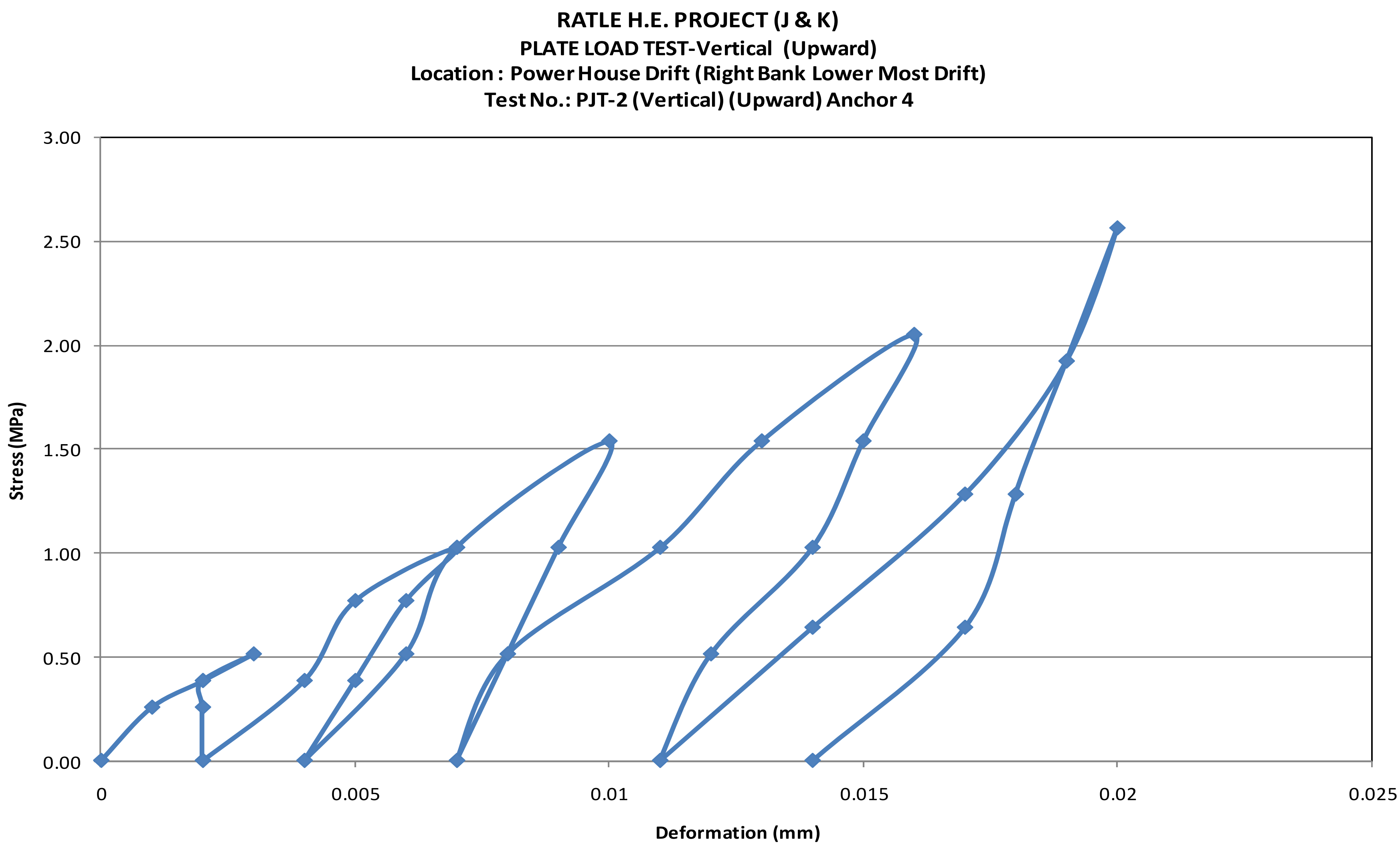


Fig. 42: Stress V/s Deformation Curve

RATLE H.E. PROJECT (J & K)

TABLE 18 : RESULTS OF PLATE LOADING TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift)
RD : 195.00m
Test No.t : PJT-2 (Vertical) (Upward)
Maximum Bearing Pressure: 2.56MPa
Outer Radius of Circular Plate:500mm; Radius of Central Hole:39mm

Modulii of Deformation Based on Vertical (Upward) Anchors

Rock Deformation Data

Designation of Anchor	Depth of Anchor (cm)	Rock Deformation (mm)
1	50	0.142
2	100	0.084
3	200	0.029
4	300	0.020
5	600	-

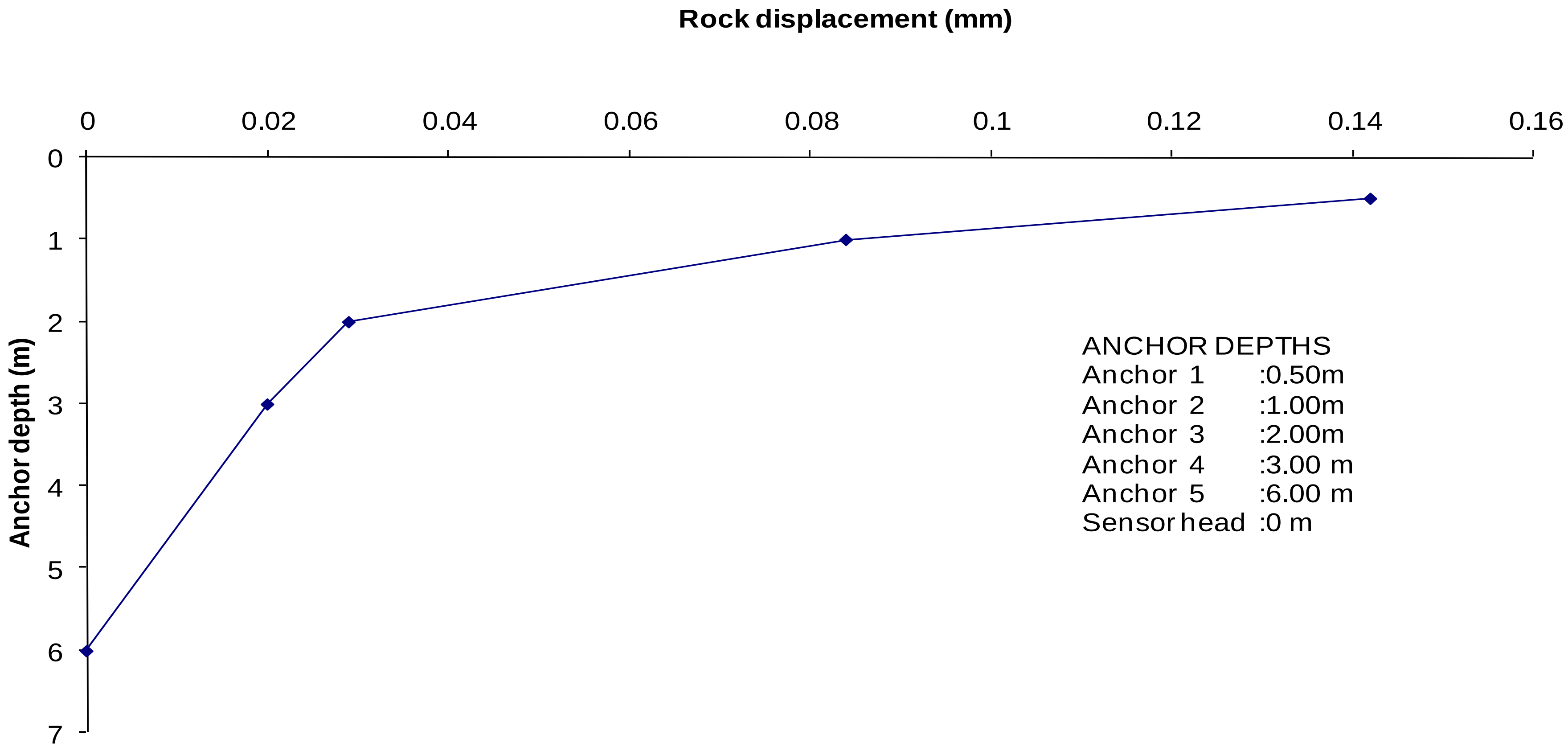
Modulii of Deformation

Thickness of Layer Below Loaded Rock Surface (cm)	Deformation Modulus (GPa)
50-100	9.52
100-200	7.54
200-300	17.36
300-600	8.10

RATLE H.E. PROJECT (J & K)

Fig. 43: Uniaxial Rock Displacement Vs Depth Referenced to Deepest Anchor at 2.56 Mpa Bearing Pressure for Vertical (Upward) Anchors

Location: Power House Drift (Right Bank Lower Most Drift); RD: 195.00m



RATLE H.E. PROJECT (J & K)

TABLE 19: RESULTS OF PLATE LOAD TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift) **RD: 195.00m**
Test No. : PJT-2 (Vertical) (Downward)
Depth of Anchors

Anchor 1: 0.50 m
Anchor 2: 1.00 m
Anchor 3: 2.00 m
Anchor 4: 3.00 m
Anchor 5: 6.00 m

Load	Stress	Anchor 1		Anchor 2		Anchor 3		Anchor 4		Anchor 5	
(KN)	(MPa)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)	Digital Readout	Deformation (mm)
0	0.00	0.002	0.000	0.002	0.000	0.005	0.000	0.002	0.000	0.002	0.000
400	0.51	0.006	0.004	0.007	0.005	0.007	0.002	0.005	0.003	0.002	0.000
0	0.00	0.003	0.001	0.004	0.002	0.006	0.001	0.002	0.000	0.002	0.000
800	1.02	0.011	0.009	0.010	0.008	0.010	0.005	0.008	0.006	0.002	0.000
0	0.00	0.007	0.005	0.007	0.005	0.008	0.003	0.005	0.003	0.002	0.000
1200	1.54	0.024	0.022	0.018	0.016	0.016	0.011	0.011	0.009	0.002	0.000
0	0.00	0.015	0.013	0.013	0.011	0.013	0.008	0.011	0.009	0.002	0.000
1600	2.05	0.054	0.052	0.036	0.034	0.024	0.019	0.015	0.013	0.002	0.000
0	0.00	0.043	0.041	0.027	0.025	0.020	0.015	0.014	0.012	0.002	0.000
2000	2.56	0.136	0.134	0.090	0.088	0.053	0.048	0.022	0.020	0.002	0.000
0	0.00	0.118	0.116	0.068	0.066	0.037	0.032	0.018	0.016	0.002	0.000

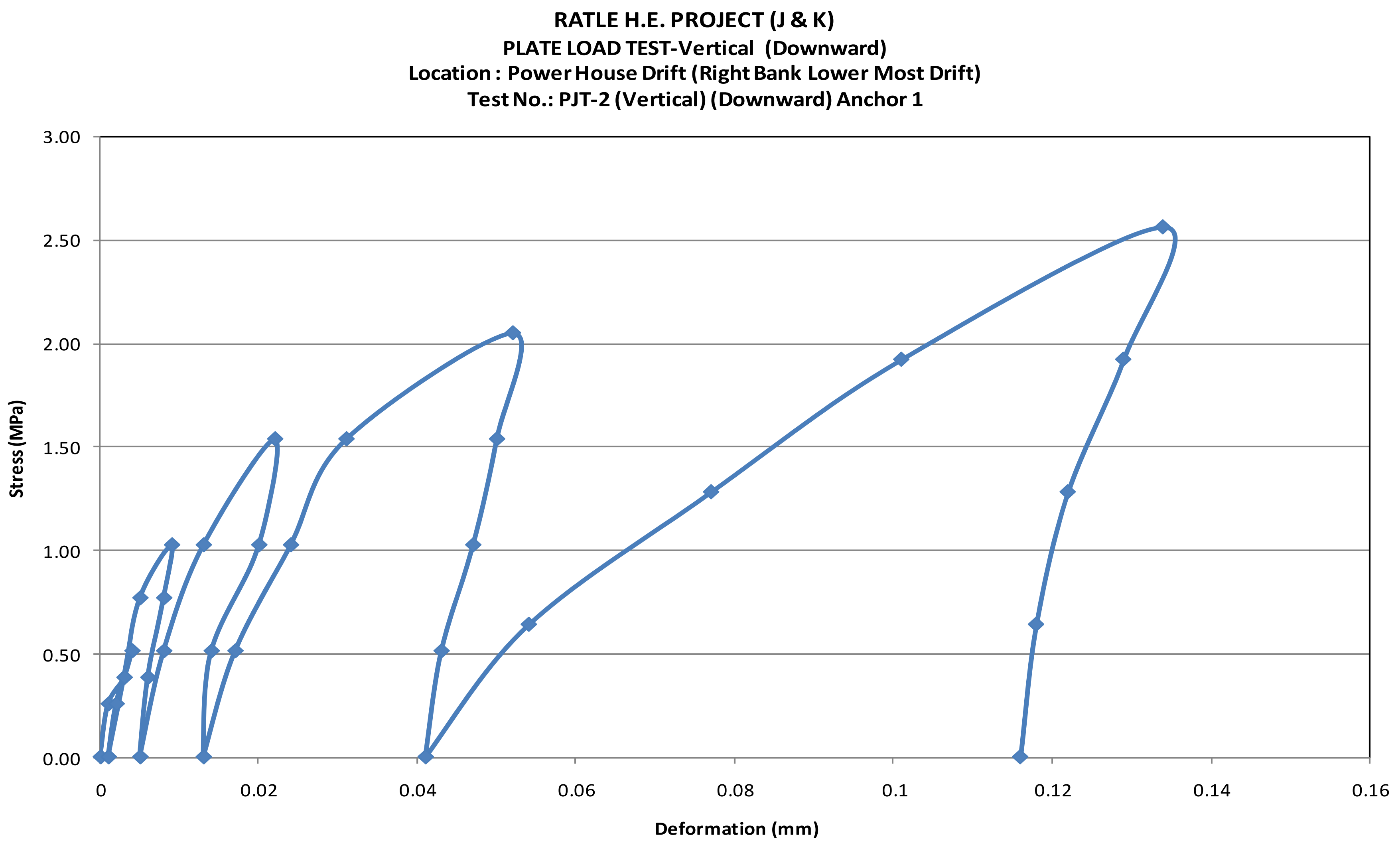


Fig. 44: Stress V/s Deformation Curve

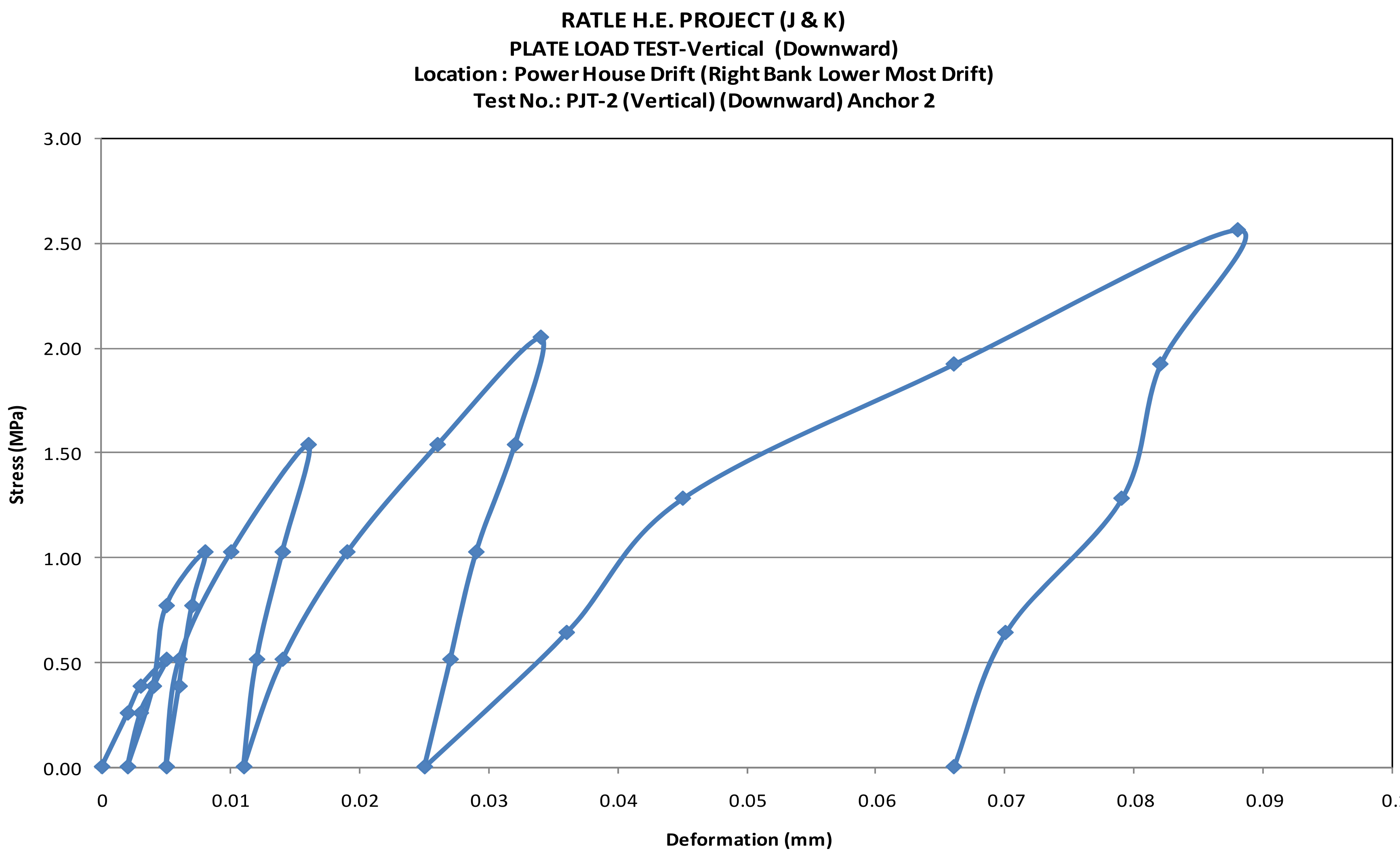


Fig. 45: Stress V/s Deformation Curve

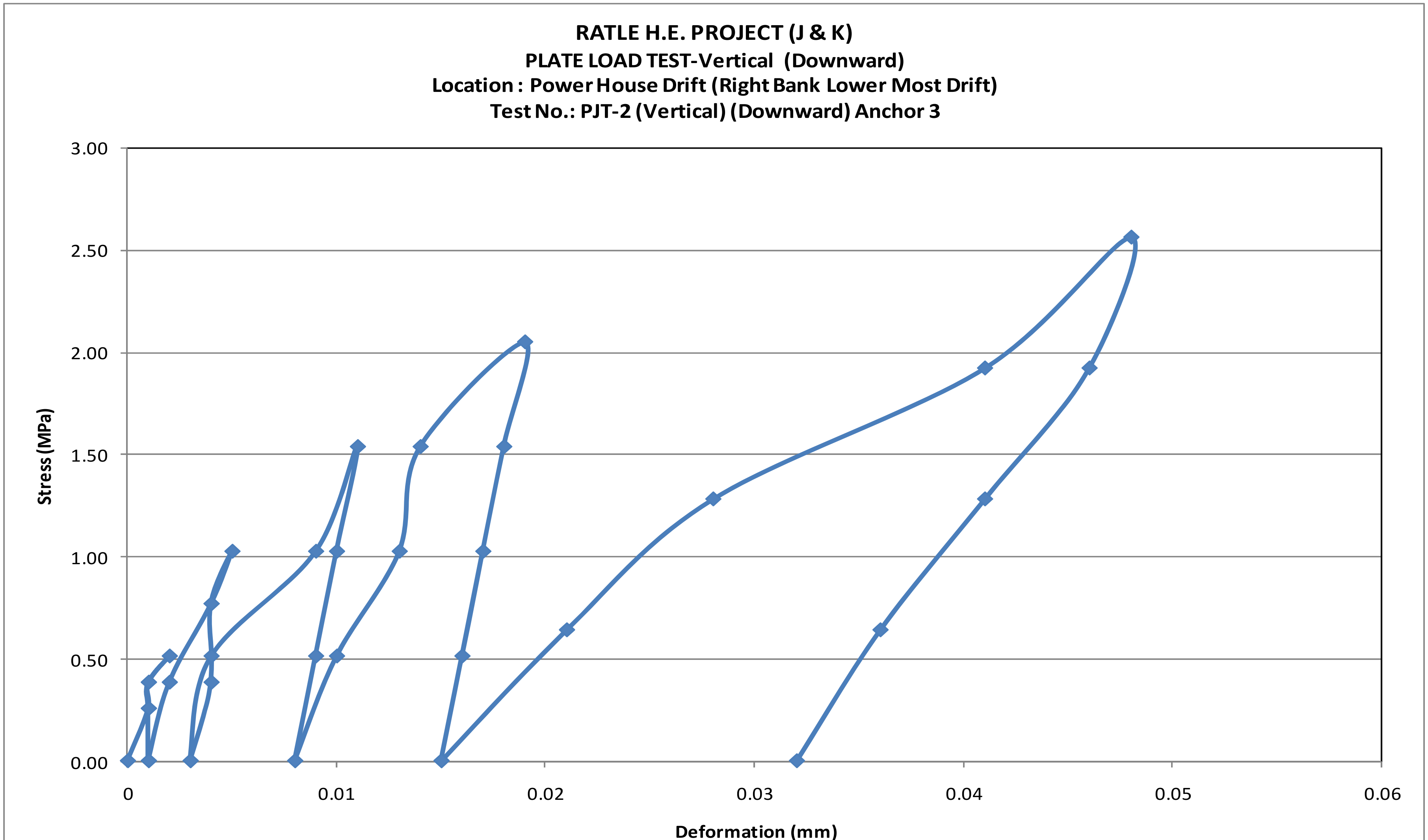


Fig. 46: Stress V/s Deformation Curve

RATLE H.E. PROJECT (J & K)
PLATE LOAD TEST-Vertical (Downward)
Location : Power House Drift (Right Bank Lower Most Drift)
Test No.: PJT-2 (Vertical) (Downward) Anchor 4

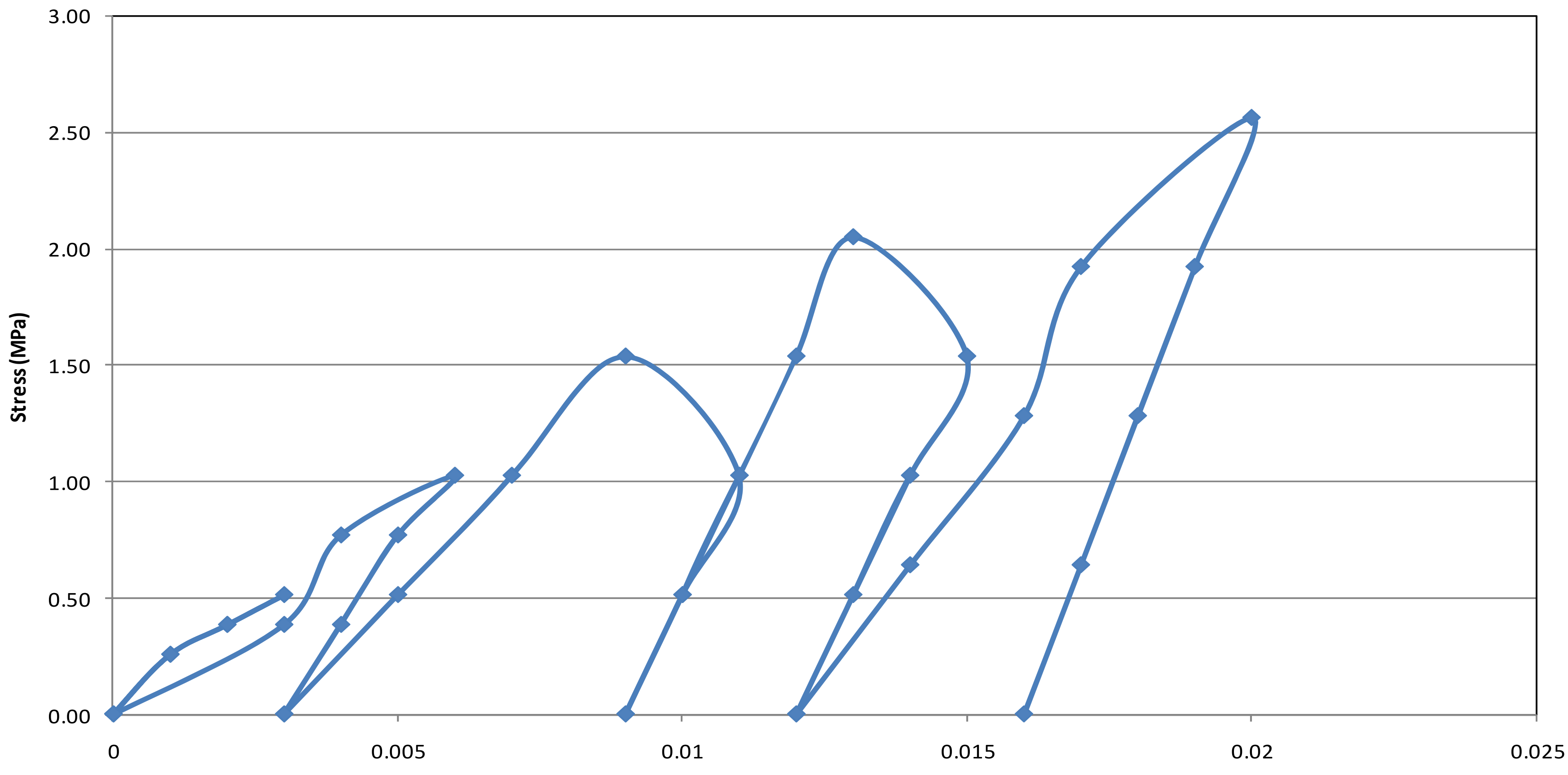


Fig. 47: Stress V/s Deformation Curve

RATLE H.E. PROJECT (J & K)

TABLE 20 : RESULTS OF PLATE LOADING TEST AT POWER HOUSE SITE

Location: Power House Drift (Right Bank Lower Most Drift)
RD : 195.00m
Test No.t : PJT-2 (Vertical) (Downward)
Maximum Bearing Pressure: 2.56MPa
Outer Radius of Circular Plate:500mm; Radius of Central Hole:39mm

Modulii of Deformation Based on Vertical (Downward) Anchors

Rock Deformation Data

Designation of Anchor	Depth of Anchor (cm)	Rock Deformation (mm)
1	50	0.134
2	100	0.088
3	200	0.048
4	300	0.020
5	600	-

Modulii of Deformation

Thickness of Layer Below Loaded Rock Surface (cm)	Deformation Modulus (GPa)
50-100	12.01
100-200	10.37
200-300	5.58
300-600	8.10

RATLE H.E. PROJECT (J & K)

Fig. 48: Uniaxial Rock Displacement Vs Depth Referenced to Deepest Anchor at 2.56 Mpa Bearing Pressure for Vertical (Downward) Anchors

Location: Power House Drift (Right Bank Lower Most Drift); RD: 195.00m

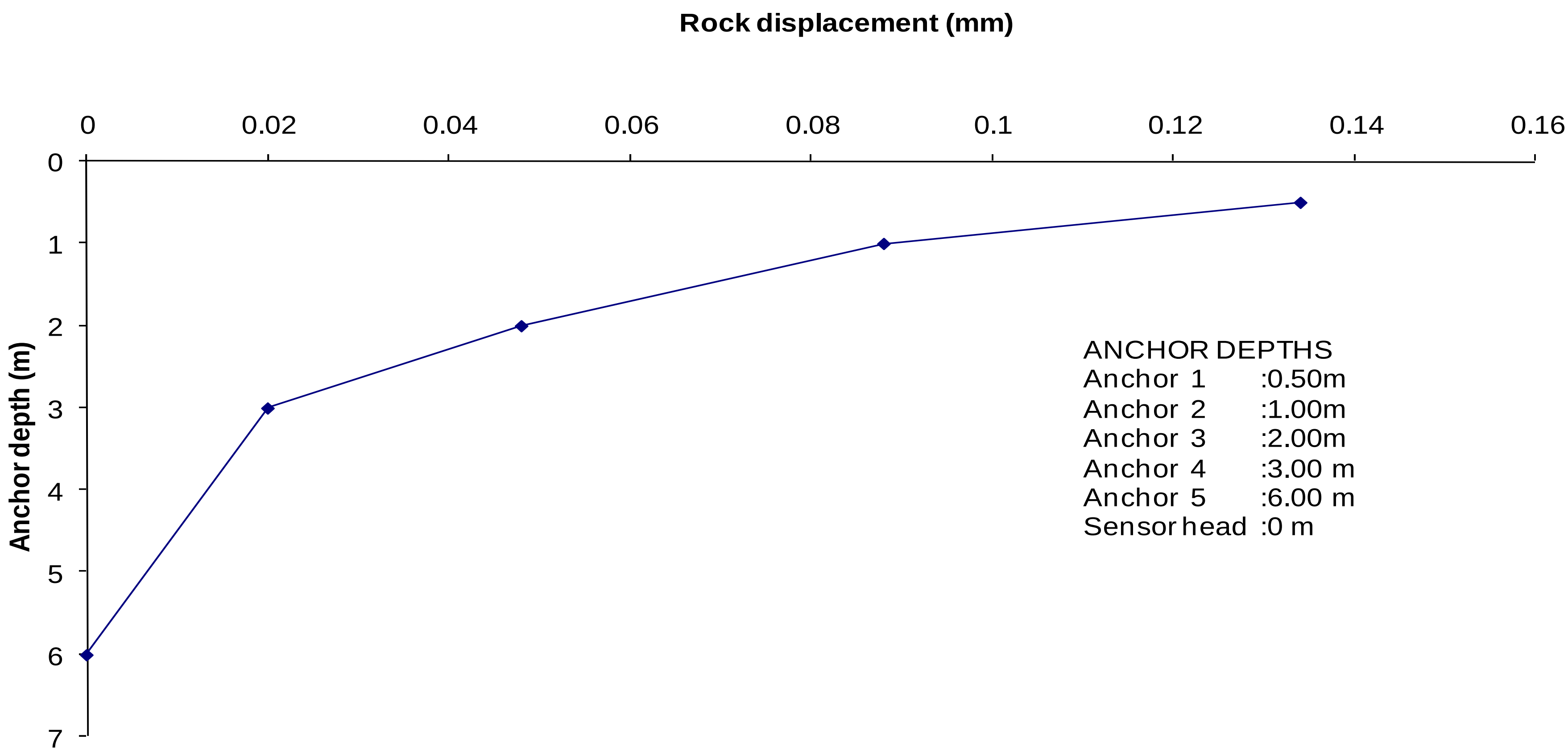


TABLE 21: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 28.00M)
Applied Normal Load: 500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD R/R1(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.02
50	0.02	48.30	4900	0.10	1.02
100	0.06	96.59	4900	0.20	1.02
150	0.10	144.89	4899	0.30	1.02
200	0.26	193.19	4898	0.39	1.02
250	0.62	241.48	4896	0.49	1.02
300	0.83	289.78	4894	0.59	1.02
350	1.14	338.07	4892	0.69	1.02
400	2.62	386.37	4882	0.79	1.02
450	4.25	434.67	4870	0.89	1.02
500	5.69	482.96	4860	0.99	1.02
550	7.40	531.26	4848	1.10	1.02
600	8.09	579.56	4843	1.20	1.02
650	8.36	627.85	4841	1.30	1.02
670	8.99	647.17	4837	**1.34**	1.02
560	9.36	540.92	4834	1.12	1.02
560	9.82	540.92	4831	1.12	1.02

Peak Shear Stress, τ = 1.34MPa

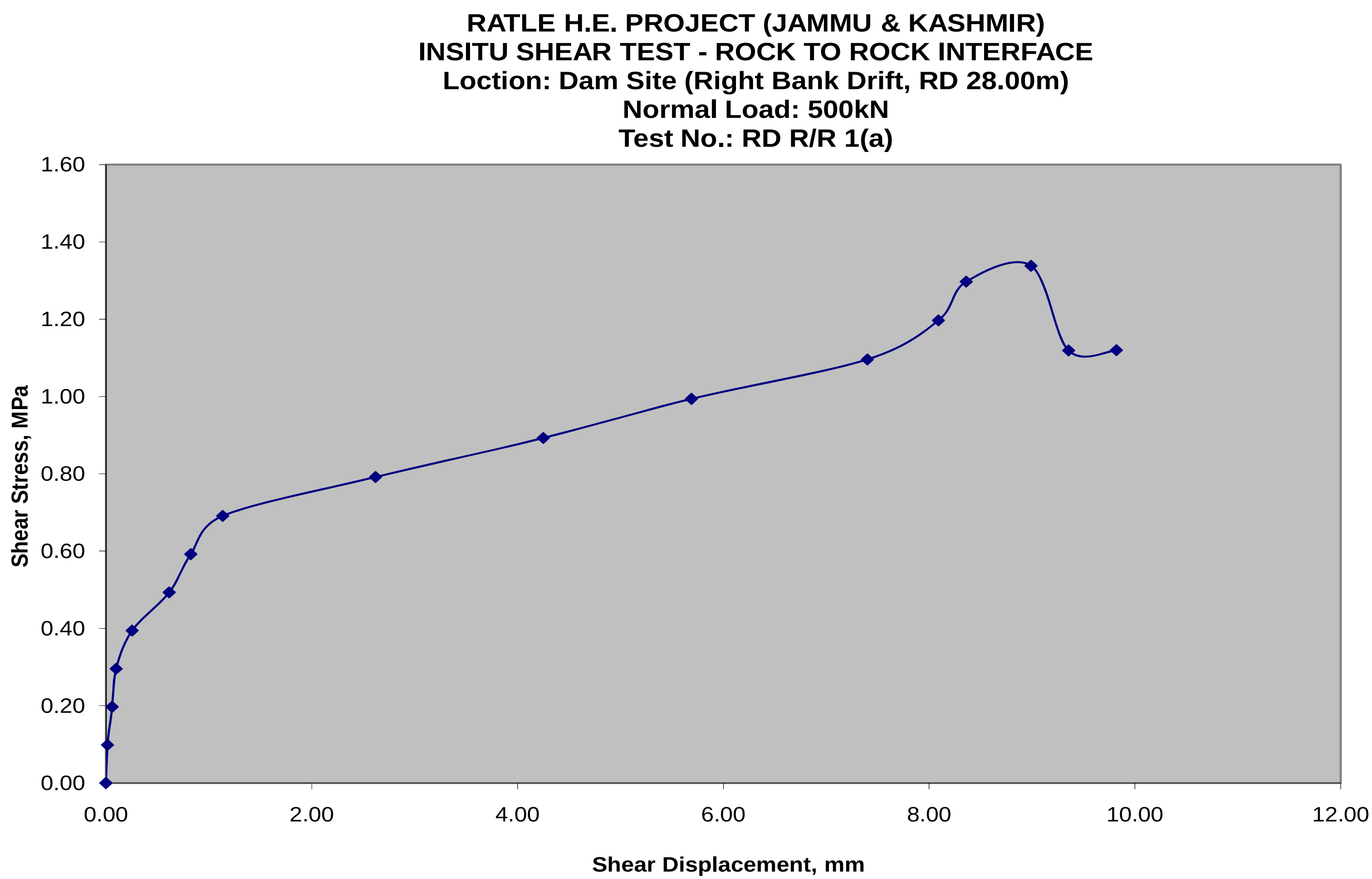


Fig. 49: Shear Stress Vs Shear Displacement

TABLE 22: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 30.50M)
Applied Normal Load: 750 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD R/R 1(b)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.53
50	0.14	48.30	4899	0.10	1.53
100	0.71	96.59	4895	0.20	1.53
150	1.13	144.89	4892	0.30	1.53
200	1.77	193.19	4888	0.40	1.53
250	2.22	241.48	4884	0.49	1.53
300	2.67	289.78	4881	0.59	1.53
350	3.12	338.07	4878	0.69	1.53
400	3.42	386.37	4876	0.79	1.53
450	4.02	434.67	4872	0.89	1.53
500	4.34	482.96	4870	0.99	1.53
550	5.00	531.26	4865	1.09	1.53
600	5.40	579.56	4862	1.19	1.53
650	5.81	627.85	4859	1.29	1.53
700	6.21	676.15	4857	1.39	1.53
750	6.93	724.44	4852	1.49	1.53
800	7.29	772.74	4849	1.59	1.53
850	7.92	821.04	4845	1.69	1.53
900	8.46	869.33	4841	1.80	1.53
950	8.73	917.63	4839	1.90	1.53
1000	9.23	965.93	4835	2.00	1.53
1050	9.67	1014.22	4832	2.10	1.53
1100	10.00	1062.52	4830	**2.20**	1.53
950	10.43	917.63	4827	1.90	1.53
950	10.79	917.63	4824	1.90	1.53

Peak Shear Stress, τ = 2.20 MPa

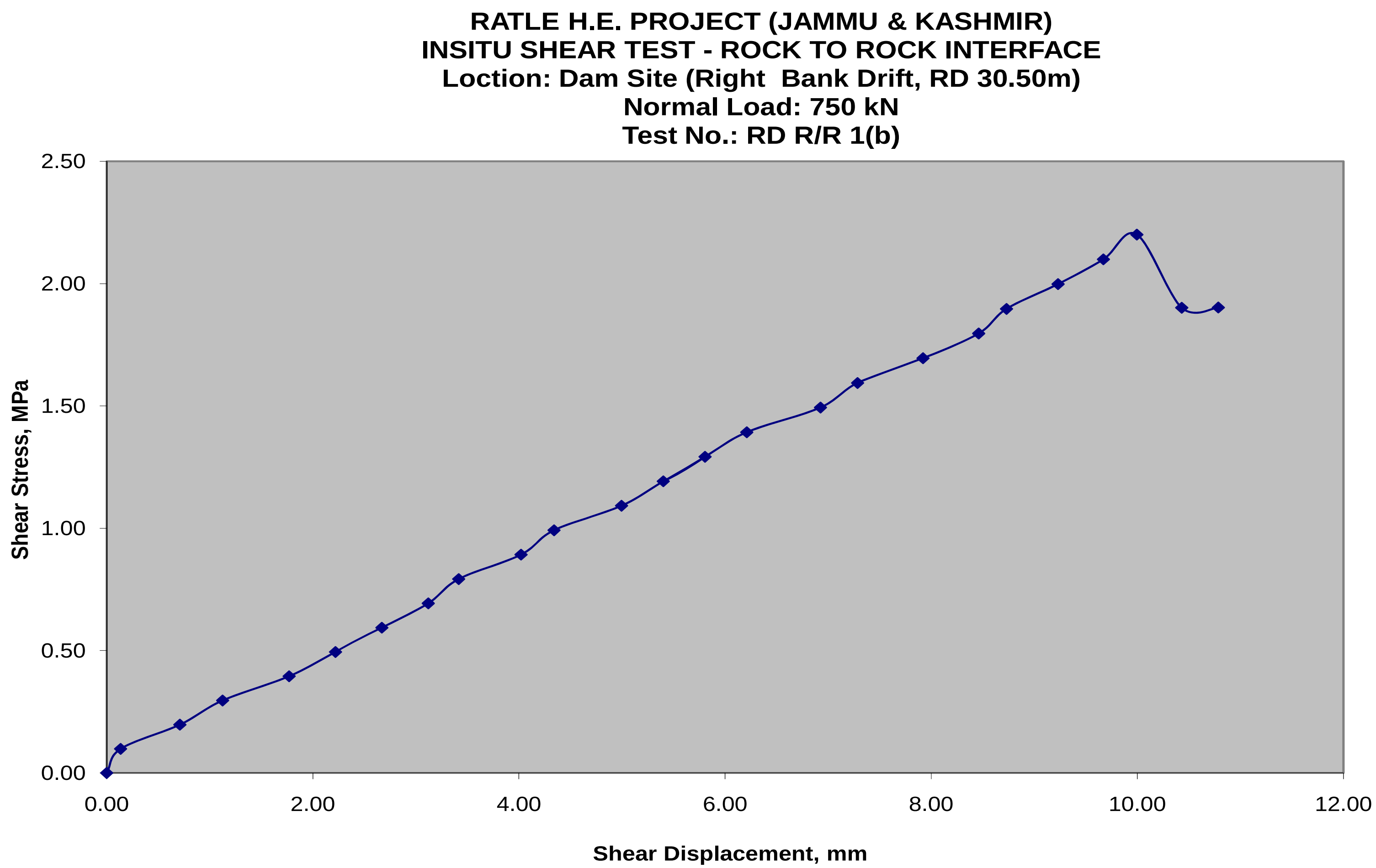


Fig. 50: Shear Stress Vs Shear Displacement

TABLE 23: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 32.20M)
Applied Normal Load: 1000 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD R/R 1(c)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.04
100	0.27	96.59	4898	0.20	2.04
200	0.55	193.19	4896	0.39	2.04
300	1.22	289.78	4891	0.59	2.04
400	2.36	386.37	4883	0.79	2.04
500	3.20	482.96	4878	0.99	2.04
600	3.84	579.56	4873	1.19	2.04
700	4.49	676.15	4869	1.39	2.04
800	5.77	772.74	4860	1.59	2.04
900	6.68	869.33	4853	1.79	2.04
1000	7.59	965.93	4847	1.99	2.04
1100	8.42	1062.52	4841	2.19	2.04
1200	9.25	1159.11	4835	2.40	2.04
1300	10.23	1255.70	4828	2.60	2.04
1350	10.76	1304.00	4825	**2.70**	2.04
1170	13.37	1130.13	4806	2.35	2.04
1170	13.95	1130.13	4802	2.35	2.04

Peak Shear Stress, τ = 2.70MPa

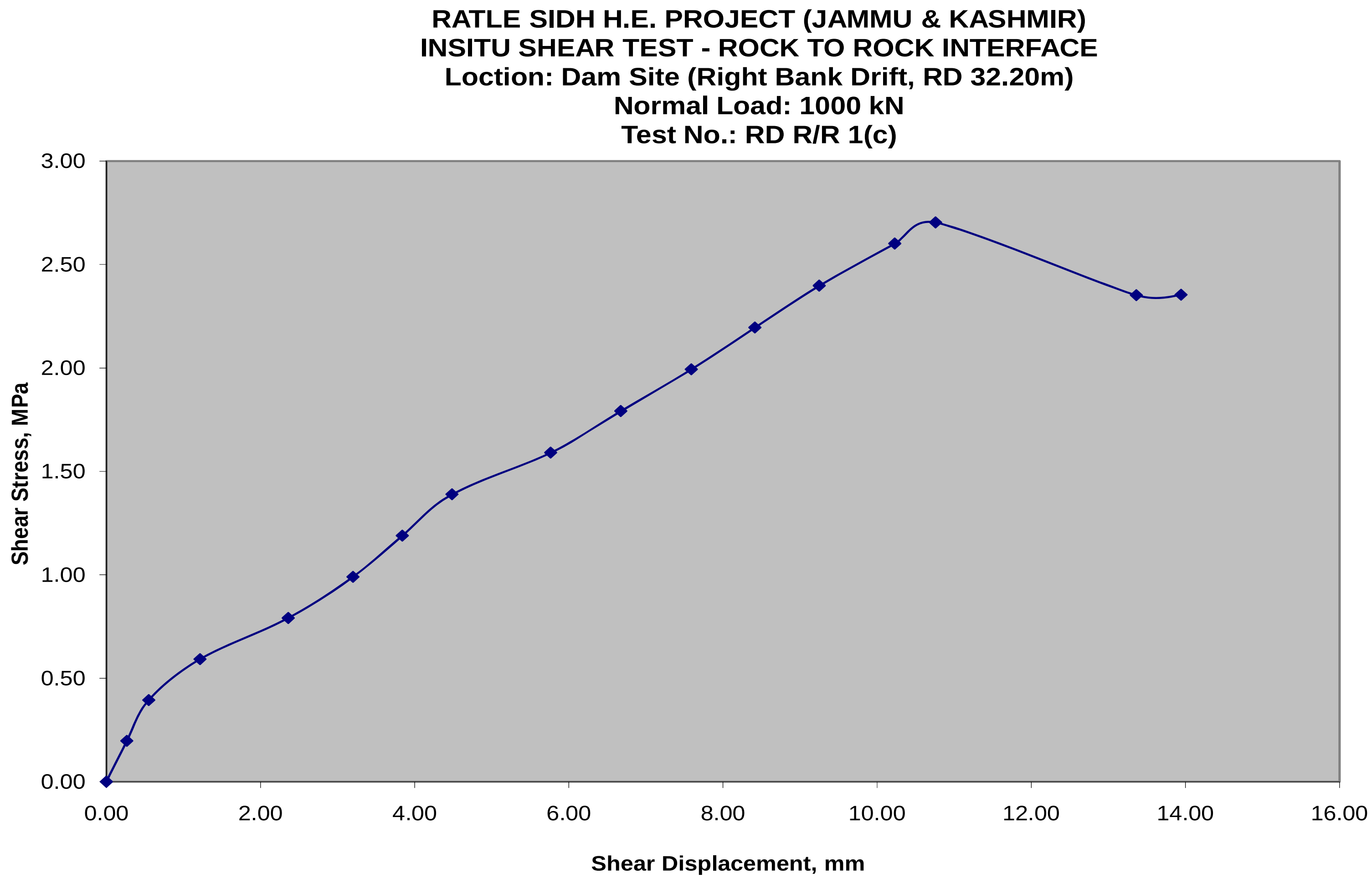


Fig. 51: Shear Stress Vs Shear Displacement

TABLE 24: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 33.70M)
Applied Normal Load: 1250 KN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD R/R 1(d)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.55
100	0.25	96.59	4898	0.20	2.55
200	0.55	193.19	4896	0.39	2.55
300	1.14	289.78	4892	0.59	2.55
400	1.82	386.37	4887	0.79	2.55
500	2.81	482.96	4880	0.99	2.55
600	3.53	579.56	4875	1.19	2.55
700	4.20	676.15	4871	1.39	2.55
800	5.22	772.74	4863	1.59	2.55
900	5.79	869.33	4860	1.79	2.55
1000	6.96	965.93	4851	1.99	2.55
1100	7.59	1062.52	4847	2.19	2.55
1200	8.74	1159.11	4839	2.40	2.55
1300	9.69	1255.70	4832	2.60	2.55
1400	10.57	1352.30	4826	2.80	2.55
1470	11.74	1419.91	4818	**2.95**	2.55
1290	14.18	1246.04	4801	2.60	2.55
1290	14.59	1246.04	4798	2.60	2.55

Peak Shear Stress, τ = 2.95MPa

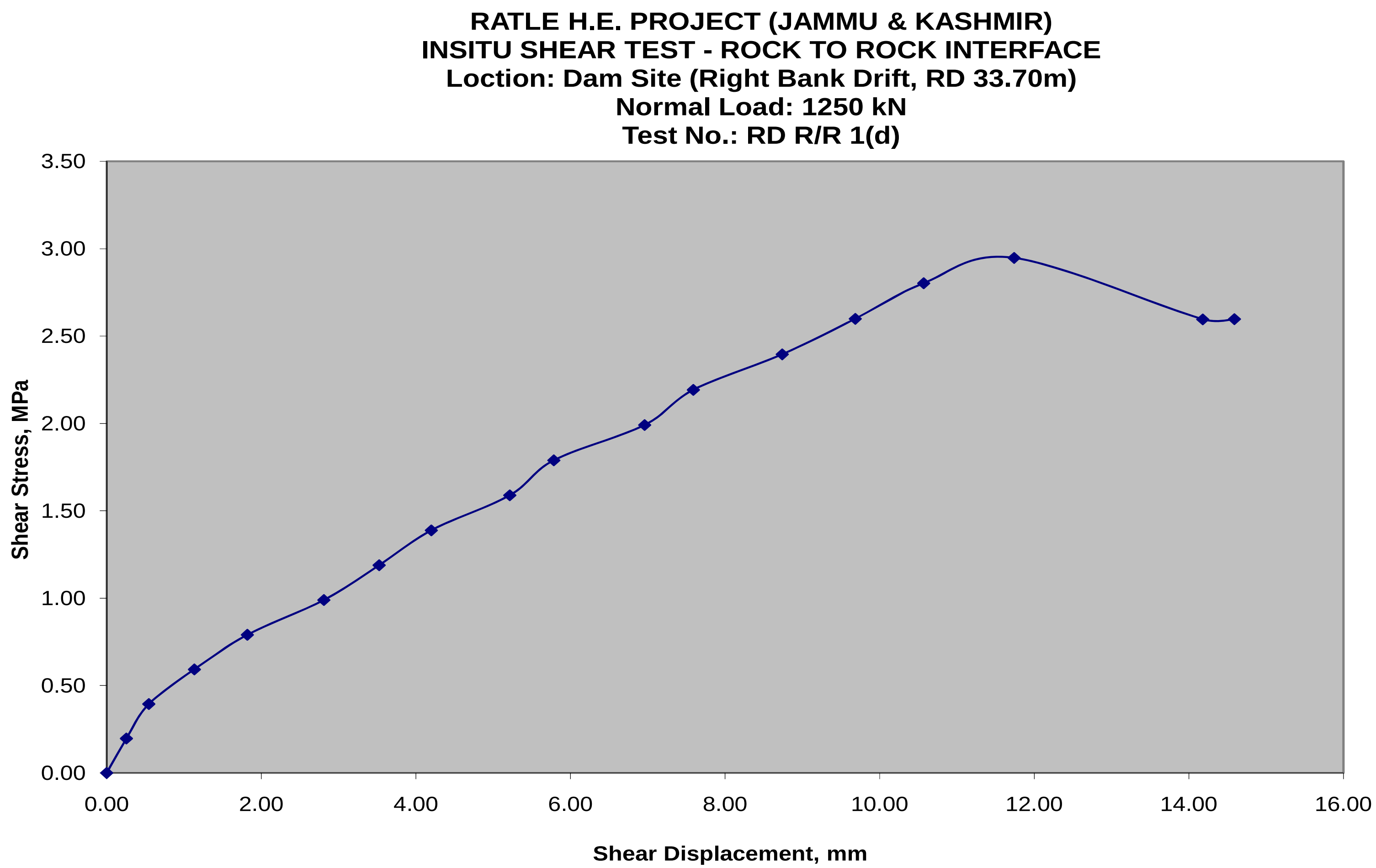


Fig. 52: Shear Stress Vs Shear Displacement

TABLE 25: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 35.70M)
Applied Normal Load: 1500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD R/R 1(e)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	3.06
100	0.21	96.59	4899	0.20	3.06
200	0.93	193.19	4894	0.39	3.06
300	2.03	289.78	4886	0.59	3.06
400	3.58	386.37	4875	0.79	3.06
500	4.04	482.96	4872	0.99	3.06
600	4.90	579.56	4866	1.19	3.06
700	6.03	676.15	4858	1.39	3.06
800	6.86	772.74	4852	1.59	3.06
900	7.48	869.33	4848	1.79	3.06
1000	8.81	965.93	4838	2.00	3.06
1100	9.35	1062.52	4835	2.20	3.06
1200	10.09	1159.11	4829	2.40	3.06
1300	10.89	1255.70	4824	2.60	3.06
1400	11.73	1352.30	4818	2.81	3.06
1500	12.40	1448.89	4813	3.01	3.06
1600	13.01	1545.48	4809	3.21	3.06
1700	13.54	1642.07	4805	3.42	3.06
1790	14.38	1729.01	4799	**3.60**	3.06
1610	17.19	1555.14	4780	3.25	3.06
1610	17.47	1555.14	4778	3.25	3.06

Peak Shear Stress, τ = 3.60MPa

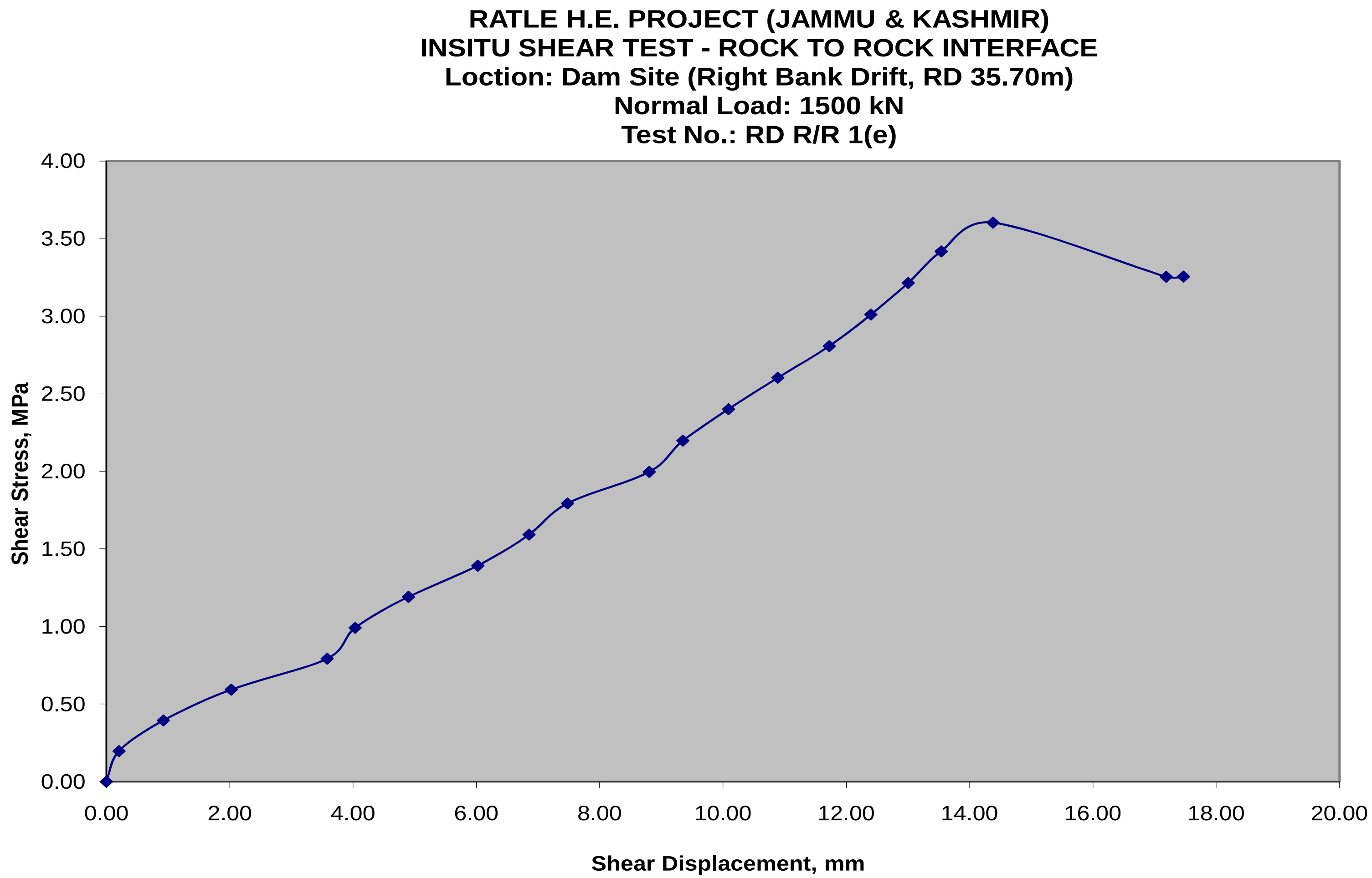


Fig. 53: Shear Stress Vs Shear Displacement

TABLE 26: VALUES OF PEAK & RESIDUAL SHEAR STRENGTH PARAMETERS

Type of Test: Insitu Shear Test
Test No.: RD R/R 1(a) to RD R/R 1(e)
Test Block Specification: Rock to Rock Interface
Location of Test Blocks: Dam Site (Right Bank Drift, RD 28.00M to RD 35.70M)

Normal Stress (MPa)	Peak Shear Stress (MPa)	Peak Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.34	0.46	46
1.53	2.20		
2.04	2.70		
2.55	2.95		
3.06	3.60		

Normal Stress (MPa)	Residual Shear Stress (MPa)	Residual Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.12	0.27	44
1.53	1.90		
2.04	2.35		
2.55	2.60		
3.06	3.25		

Dam Site (Right Bank Drift, RD 28.00M to 35.70M)
Rock to Rock Interface

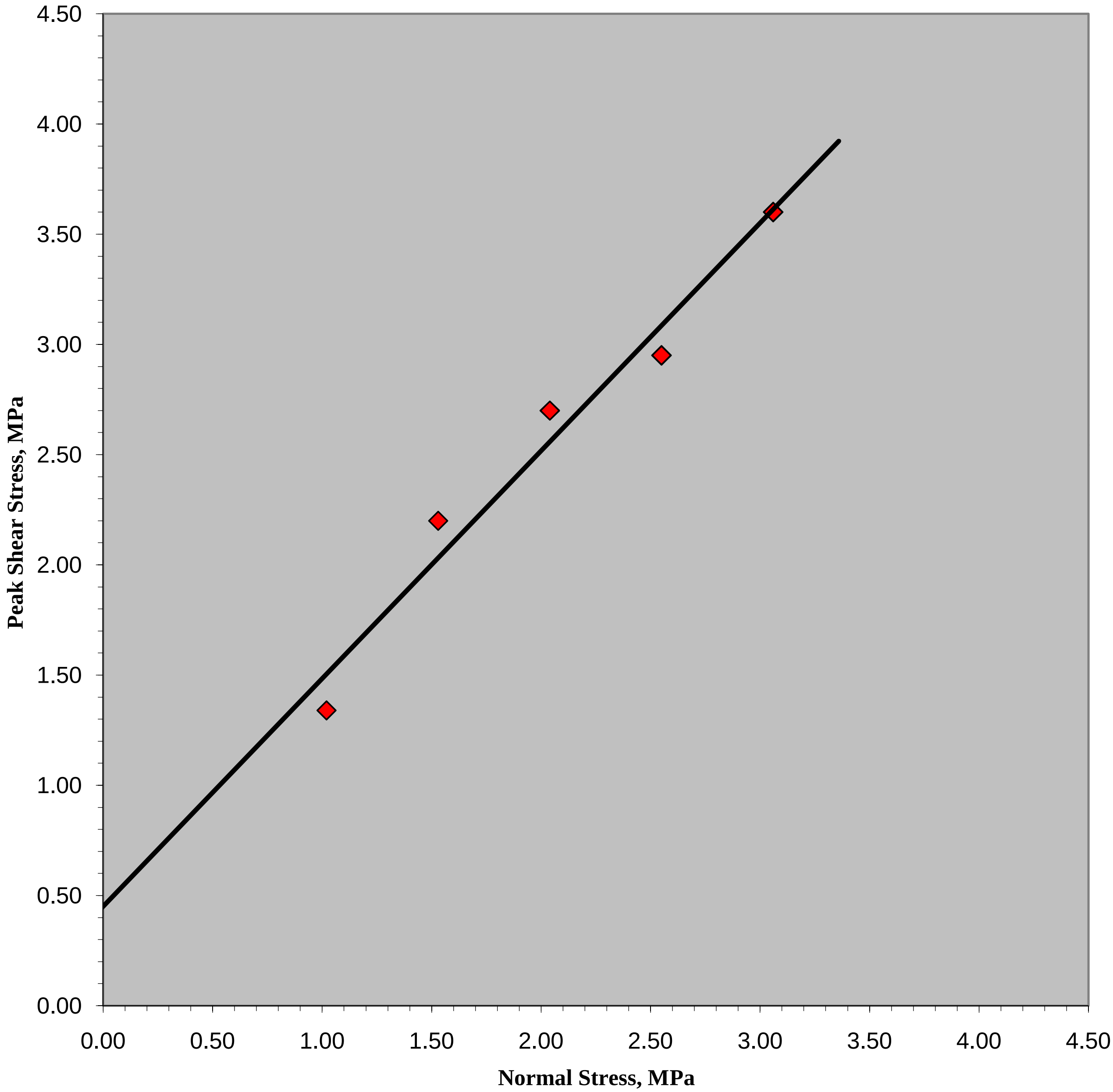


Fig. 54: Peak Shear Stress Vs Normal Stress

Dam Site (Right Bank Drift, RD 28.00M to 35.70M)
Rock to Rock Interface

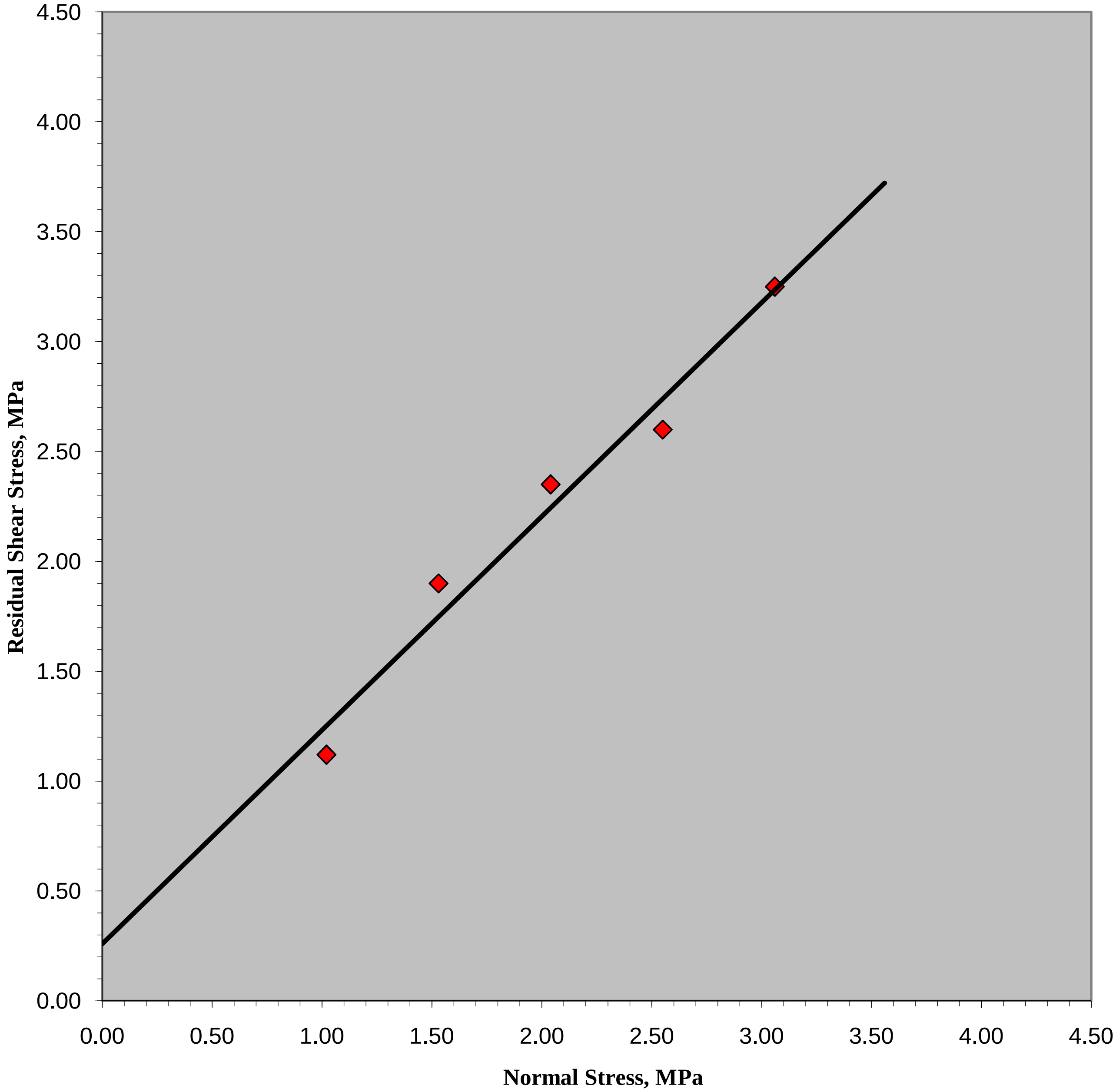


Fig. 55: Residual Shear Stress Vs Normal Stress

TABLE 27: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 8.00M)
Applied Normal Load: 500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD R/R 2(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.02
50	0.04	48.30	4900	0.10	1.02
100	0.12	96.59	4899	0.20	1.02
150	0.27	144.89	4898	0.30	1.02
200	0.34	193.19	4898	0.39	1.02
250	0.44	241.48	4897	0.49	1.02
300	0.67	289.78	4895	0.59	1.02
350	1.02	338.07	4893	0.69	1.02
400	1.65	386.37	4888	0.79	1.02
450	2.70	434.67	4881	0.89	1.02
500	3.83	482.96	4873	0.99	1.02
550	4.96	531.26	4865	1.09	1.02
600	6.09	579.56	4857	1.19	1.02
650	7.22	627.85	4849	1.29	1.02
700	8.35	676.15	4842	1.40	1.02
710	9.48	685.81	4834	**1.42**	1.02
670	10.61	647.17	4826	1.34	1.02
670	11.74	647.17	4818	1.34	1.02

Peak Shear Stress, τ = 1.42MPa

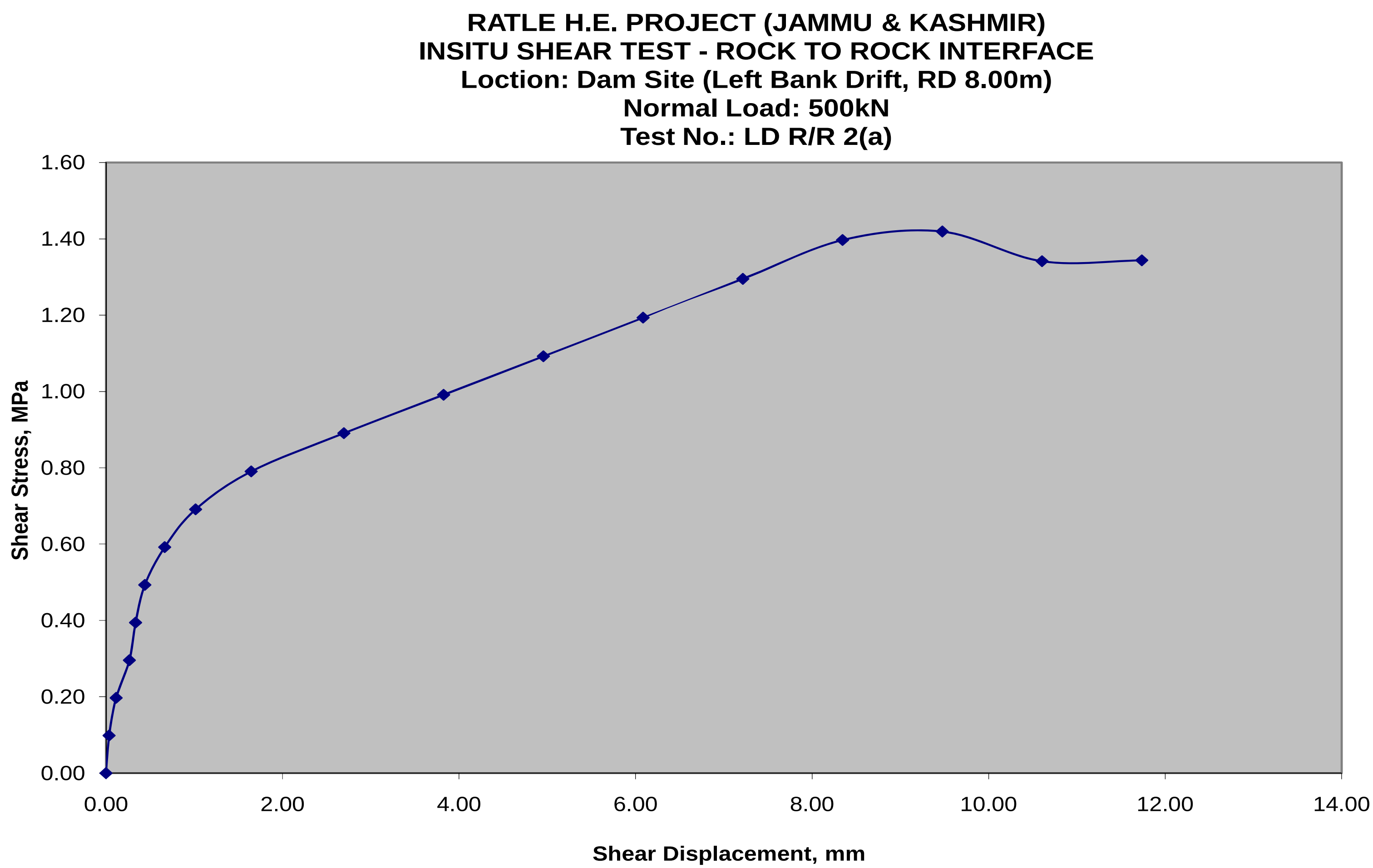


Fig. 56: Shear Stress Vs Shear Displacement

TABLE 28: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 10.00M)
Applied Normal Load: 750 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD R/R 2(b)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.53
50	0.48	48.30	4897	0.10	1.53
100	0.74	96.59	4895	0.20	1.53
150	1.22	144.89	4891	0.30	1.53
200	1.77	193.19	4888	0.40	1.53
250	2.06	241.48	4886	0.49	1.53
300	2.79	289.78	4881	0.59	1.53
350	3.07	338.07	4879	0.69	1.53
400	3.46	386.37	4876	0.79	1.53
450	3.75	434.67	4874	0.89	1.53
500	4.20	482.96	4871	0.99	1.53
550	4.96	531.26	4865	1.09	1.53
600	6.33	579.56	4856	1.19	1.53
650	7.68	627.85	4846	1.30	1.53
700	8.89	676.15	4838	1.40	1.53
750	10.23	724.44	4828	1.50	1.53
800	11.05	772.74	4823	1.60	1.53
850	11.59	821.04	4819	1.70	1.53
900	12.14	869.33	4815	1.81	1.53
950	12.68	917.63	4811	1.91	1.53
1000	13.23	965.93	4807	2.01	1.53
1050	13.52	1014.22	4805	2.11	1.53
1100	13.88	1062.52	4803	2.21	1.53
1150	14.25	1110.81	4800	2.31	1.53
1200	14.50	1159.11	4799	2.42	1.53
1250	14.84	1207.41	4796	2.52	1.53
1260	16.95	1217.07	4781	**2.55**	1.53
1030	19.02	994.90	4767	2.09	1.53
1030	19.50	994.90	4764	2.09	1.53

Peak Shear Stress, τ = 2.55 MPa

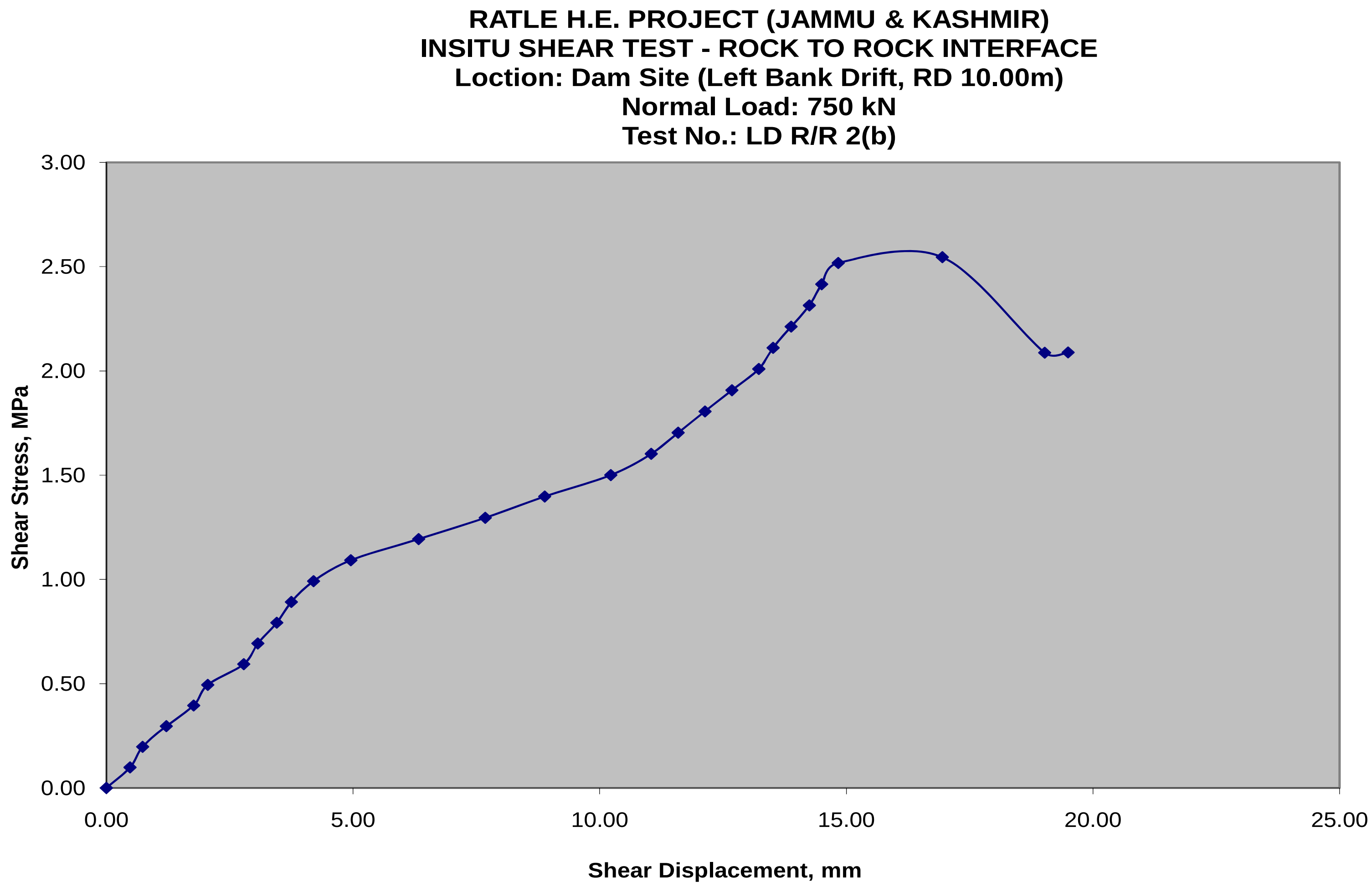


Fig.57: Shear Stress Vs Shear Displacement

TABLE 29: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 11.20M)
Applied Normal Load: 1000 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD R/R 2(c)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.04
50	0.26	48.30	4898	0.10	2.04
100	0.62	96.59	4896	0.20	2.04
200	3.00	193.19	4879	0.40	2.04
300	3.45	289.78	4876	0.59	2.04
400	4.83	386.37	4866	0.79	2.04
500	5.02	482.96	4865	0.99	2.04
600	5.39	579.56	4862	1.19	2.04
700	6.05	676.15	4858	1.39	2.04
800	6.71	772.74	4853	1.59	2.04
900	7.37	869.33	4848	1.79	2.04
1000	8.03	965.93	4844	1.99	2.04
1100	8.69	1062.52	4839	2.20	2.04
1200	9.35	1159.11	4835	2.40	2.04
1300	10.01	1255.70	4830	2.60	2.04
1400	10.67	1352.30	4825	2.80	2.04
1500	11.29	1448.89	4821	3.01	2.04
1560	12.37	1506.84	4813	**3.13**	2.04
1370	14.42	1323.32	4799	2.76	2.04
1370	15.12	1323.32	4794	2.76	2.04

Peak Shear Stress, τ = 3.13MPa

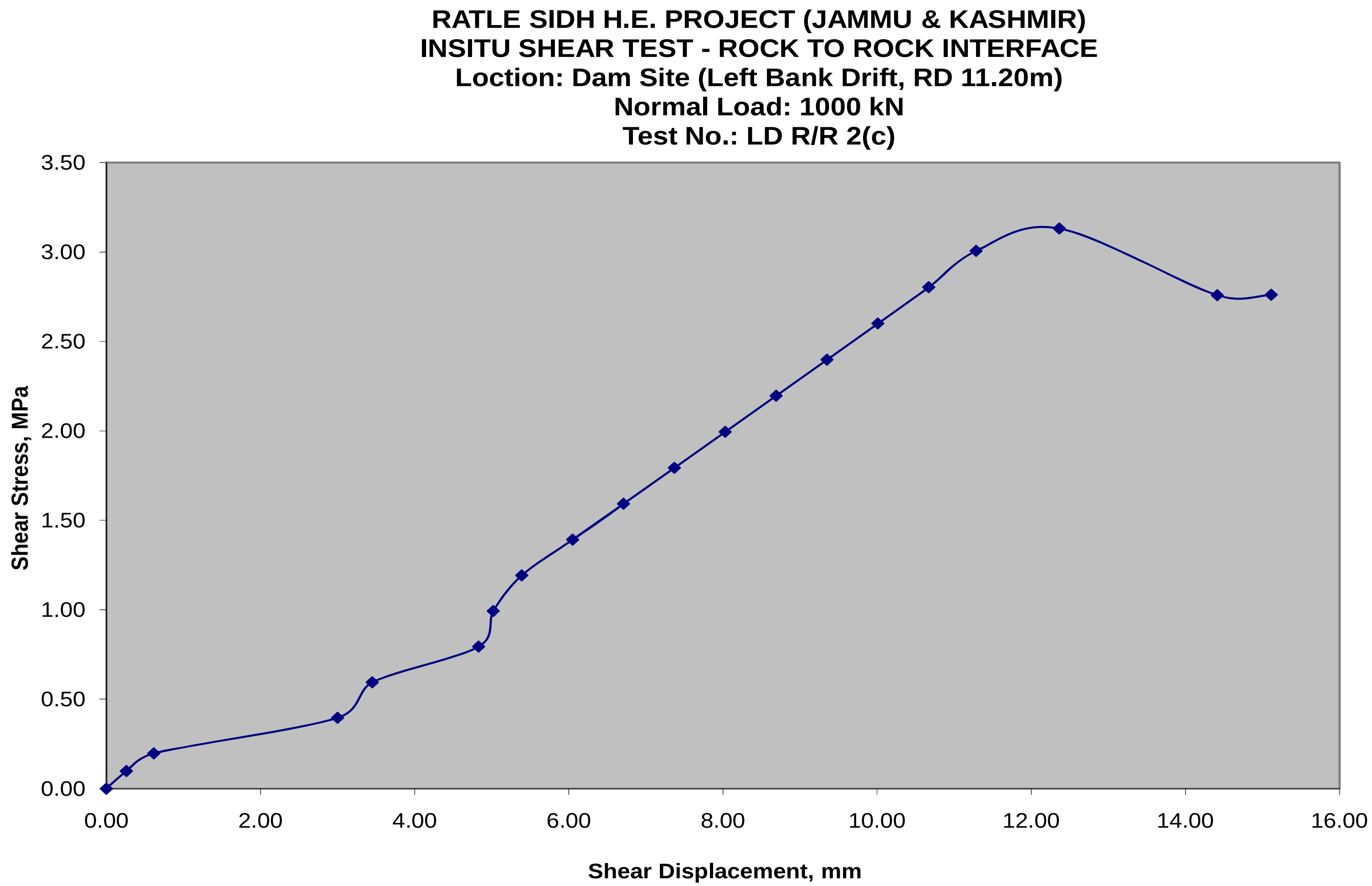


Fig.58: Shear Stress Vs Shear Displacement

TABLE 30: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 13.40M)
Applied Normal Load: 1250 KN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD R/R 2(d)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.55
100	0.45	96.59	4897	0.20	2.55
200	0.76	193.19	4895	0.39	2.55
300	1.13	289.78	4892	0.59	2.55
400	1.37	386.37	4890	0.79	2.55
500	1.73	482.96	4888	0.99	2.55
600	2.14	579.56	4885	1.19	2.55
700	2.65	676.15	4881	1.39	2.55
800	3.07	772.74	4879	1.58	2.55
900	3.63	869.33	4875	1.78	2.55
1000	4.37	965.93	4869	1.98	2.55
1100	4.88	1062.52	4866	2.18	2.55
1200	5.54	1159.11	4861	2.38	2.55
1300	6.16	1255.70	4857	2.59	2.55
1400	6.79	1352.30	4852	2.79	2.55
1500	7.41	1448.89	4848	2.99	2.55
1600	8.04	1545.48	4844	3.19	2.55
1700	8.66	1642.07	4839	3.39	2.55
1800	9.29	1738.67	4835	3.60	2.55
1840	9.86	1777.30	4831	**3.68**	2.55
1540	12.24	1487.53	4814	3.09	2.55
1540	12.91	1487.53	4810	3.09	2.55

Peak Shear Stress, τ = 3.68MPa

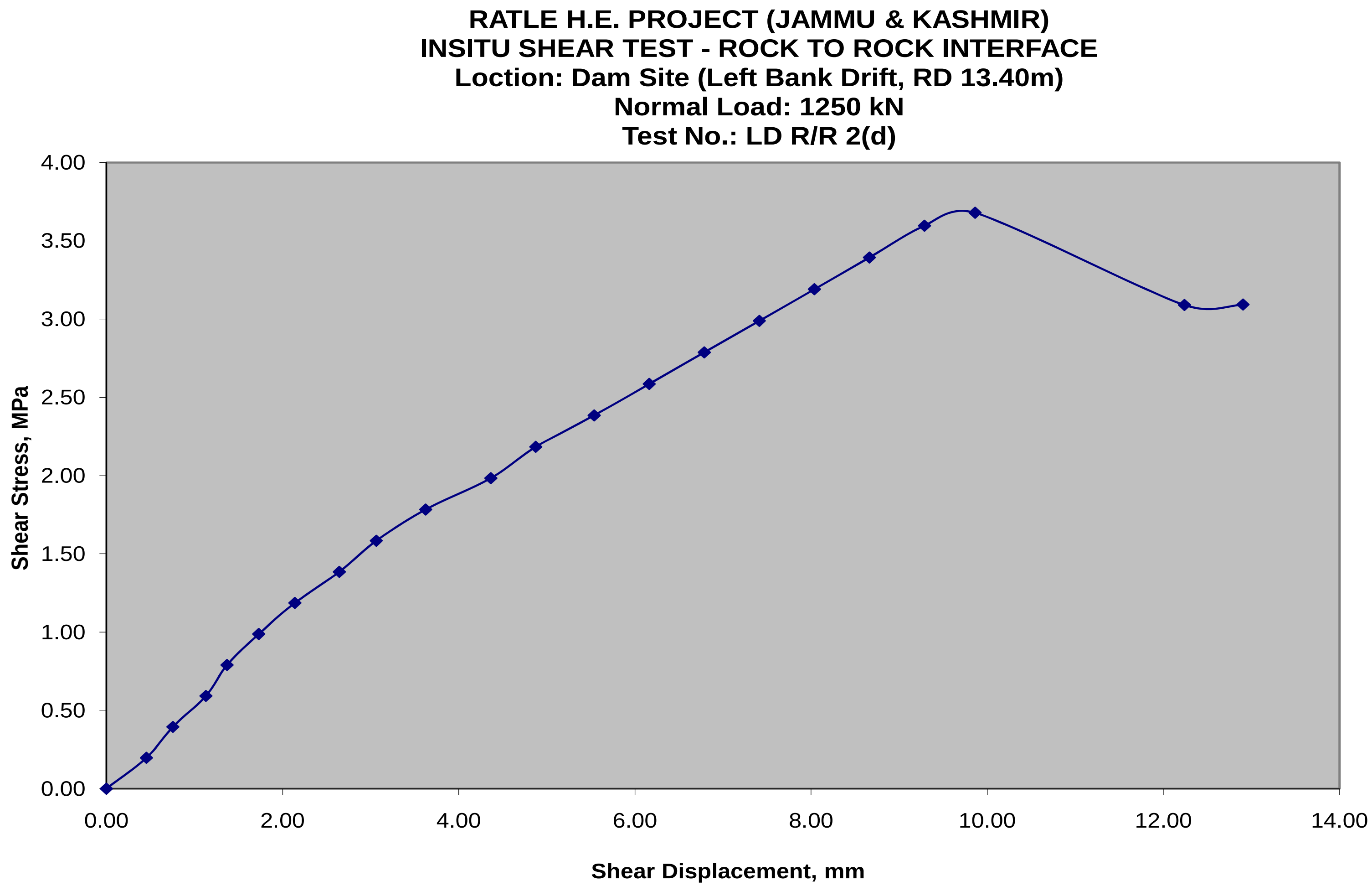


Fig. 59: Shear Stress Vs Shear Displacement

TABLE 31: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 15.00M)
Applied Normal Load: 1500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD R/R 2(e)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	3.06
100	0.12	96.59	4899	0.20	3.06
200	0.32	193.19	4898	0.39	3.06
300	0.75	289.78	4895	0.59	3.06
400	1.20	386.37	4892	0.79	3.06
500	1.46	482.96	4890	0.99	3.06
600	2.58	579.56	4882	1.19	3.06
700	3.00	676.15	4879	1.39	3.06
800	3.36	772.74	4876	1.58	3.06
900	4.46	869.33	4869	1.79	3.06
1000	4.95	965.93	4865	1.99	3.06
1100	5.55	1062.52	4861	2.19	3.06
1200	5.93	1159.11	4858	2.39	3.06
1300	6.43	1255.70	4855	2.59	3.06
1400	7.09	1352.30	4850	2.79	3.06
1500	7.91	1448.89	4845	2.99	3.06
1600	8.40	1545.48	4841	3.19	3.06
1700	9.31	1642.07	4835	3.40	3.06
1800	9.90	1738.67	4831	3.60	3.06
1900	10.85	1835.26	4824	3.80	3.06
1960	11.41	1893.21	4820	**3.93**	3.06
1820	14.01	1757.99	4802	3.66	3.06
1820	14.47	1757.99	4799	3.66	3.06

Peak Shear Stress, τ = 3.93MPa

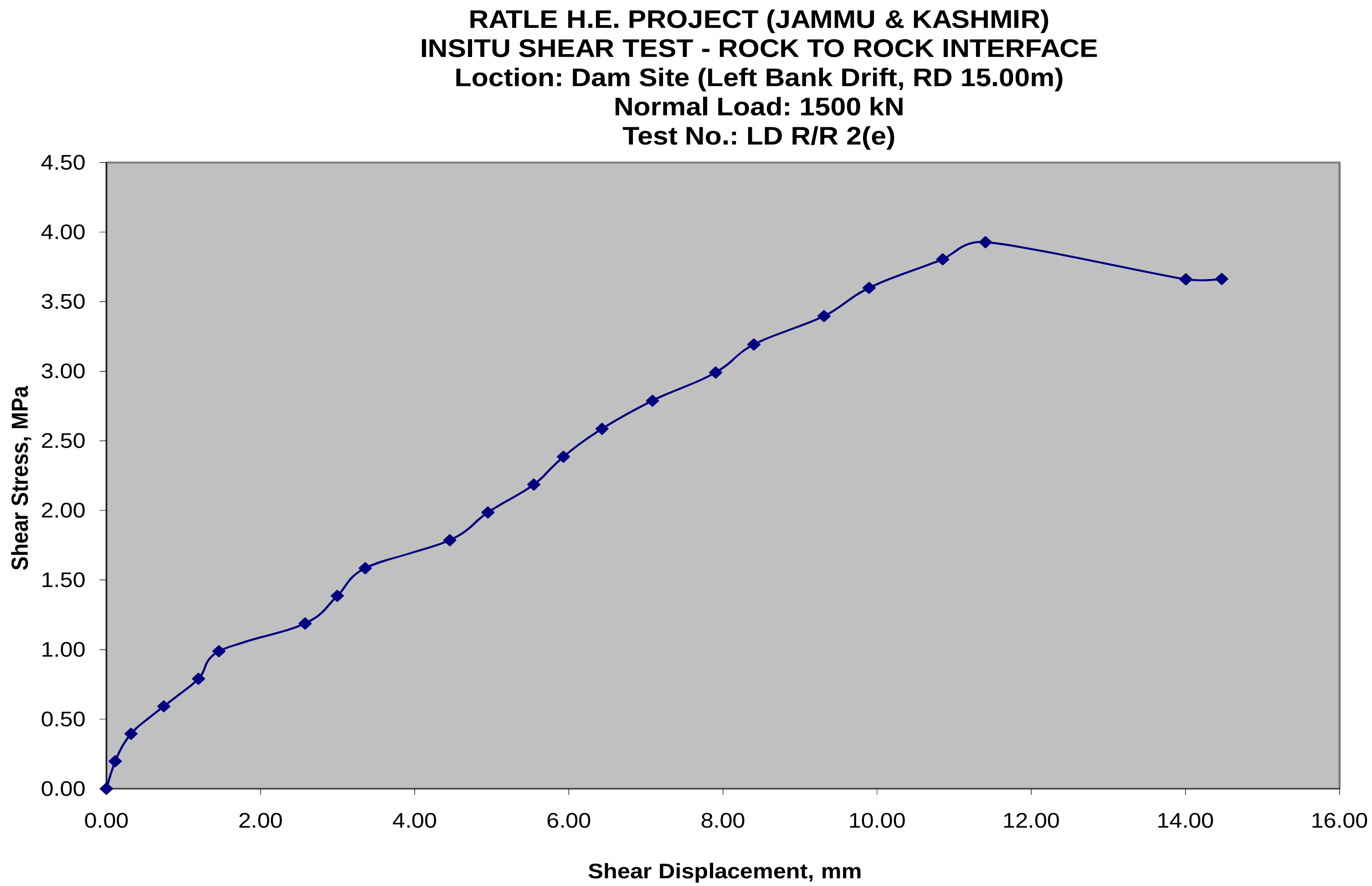


Fig.60: Shear Stress Vs Shear Displacement

TABLE 32: VALUES OF PEAK & RESIDUAL SHEAR STRENGTH PARAMETERS

Type of Test: Insitu Shear Test
Test No.: LD R/R 2(a) to LD R/R 2(e)
Test Block Specification: Rock to Rock Interface
Location of Test Blocks: Dam Site (Left Bank Drift, RD 8.00M to RD 15.00M)

Normal Stress (MPa)	Peak Shear Stress (MPa)	Peak Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.42	0.48	50
1.53	2.55		
2.04	3.13		
2.55	3.68		
3.06	3.93		

Normal Stress (MPa)	Residual Shear Stress (MPa)	Residual Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.34	0.33	47.5
1.53	2.09		
2.04	2.76		
2.55	3.09		
3.06	3.66		

Dam Site (Left Bank Drift, RD 8.00M to 15.00M)
Rock to Rock Interface

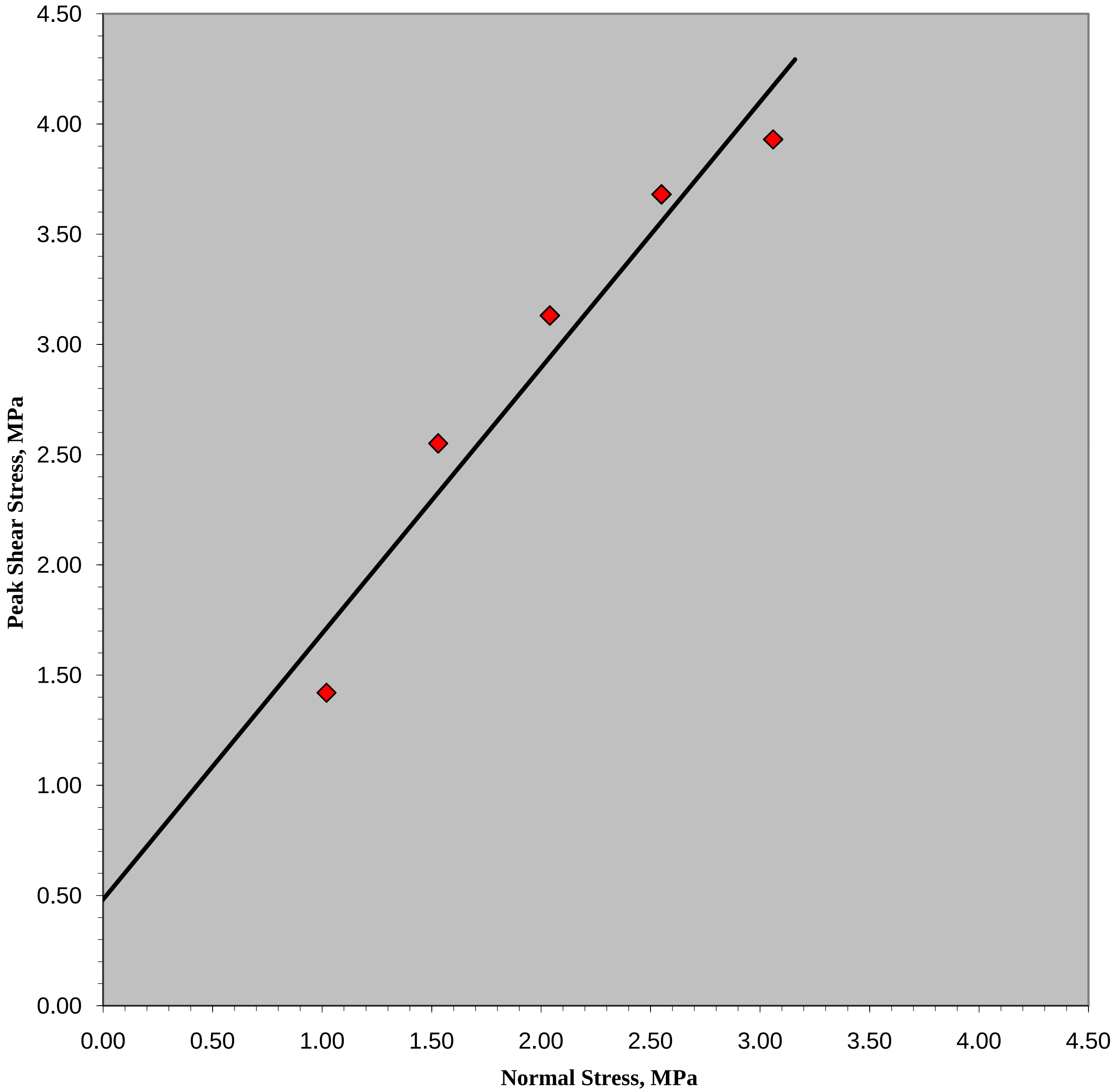


Fig. 61: Peak Shear Stress Vs Normal Stress

Dam Site (Left Bank Drift, RD 8.00M to 15.00M)
Rock to Rock Interface

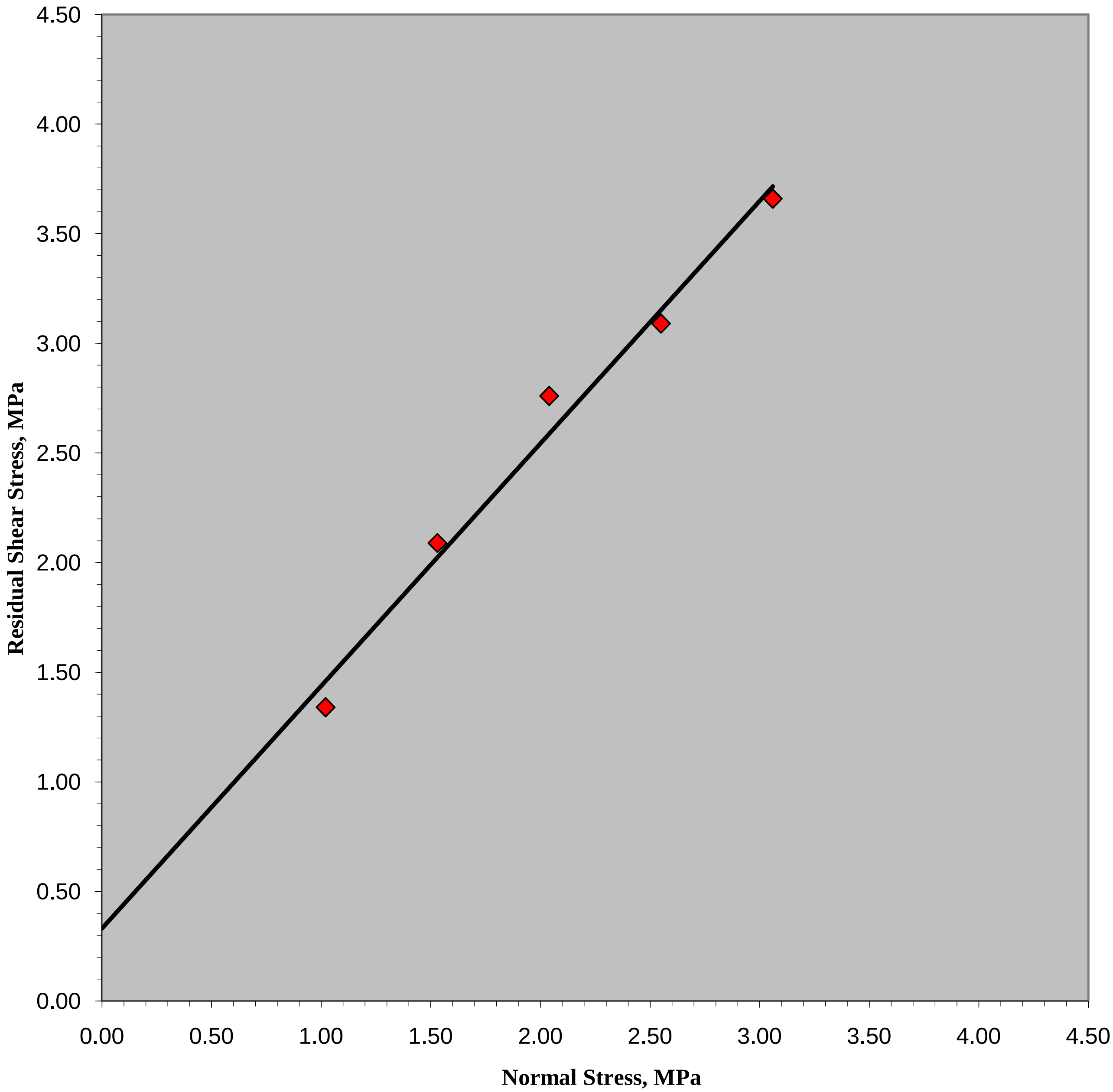


Fig. 62: Residual Shear Stress Vs Normal Stress

TABLE 33: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: Power House Drift (RD 202.00M + 8.40M Towards U/S)
Applied Normal Load: 500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: PH R/R 3(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.02
50	0.39	48.30	4897	0.10	1.02
100	0.97	96.59	4893	0.20	1.02
150	1.20	144.89	4892	0.30	1.02
200	1.79	193.19	4888	0.40	1.02
250	2.26	241.48	4884	0.49	1.02
300	2.64	289.78	4882	0.59	1.02
350	3.15	338.07	4878	0.69	1.02
400	3.64	386.37	4875	0.79	1.02
450	3.97	434.67	4872	0.89	1.02
500	5.24	482.96	4863	0.99	1.02
550	5.97	531.26	4858	1.09	1.02
600	7.01	579.56	4851	1.19	1.02
620	7.51	598.87	4847	**1.24**	1.02
530	7.97	511.94	4844	1.06	1.02
530	8.53	511.94	4840	1.06	1.02

Peak Shear Stress, τ = 1.24MPa

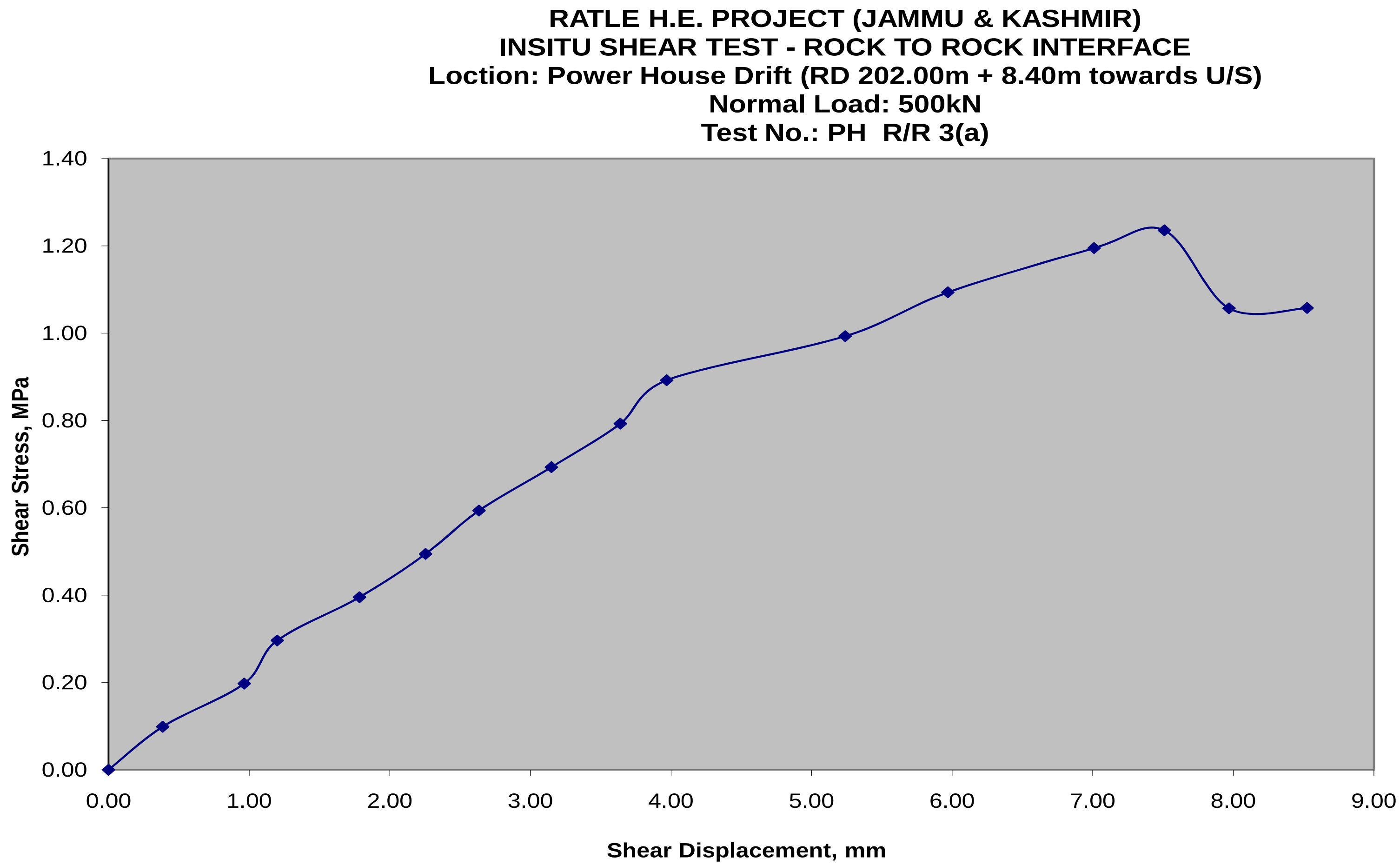


Fig. 63: Shear Stress Vs Shear Displacement

TABLE 34: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: Power House Drift (RD 202.00M + 6.25M Towards U/S)
Applied Normal Load: 750 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: PH R/R 3(b)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.53
50	0.37	48.30	4897	0.10	1.53
100	0.59	96.59	4896	0.20	1.53
150	0.93	144.89	4893	0.30	1.53
200	1.24	193.19	4891	0.39	1.53
250	1.47	241.48	4890	0.49	1.53
300	1.85	289.78	4887	0.59	1.53
350	2.24	338.07	4884	0.69	1.53
400	2.46	386.37	4883	0.79	1.53
450	2.96	434.67	4879	0.89	1.53
500	4.33	482.96	4870	0.99	1.53
550	5.37	531.26	4862	1.09	1.53
600	6.35	579.56	4856	1.19	1.53
650	7.30	627.85	4849	1.29	1.53
700	8.14	676.15	4843	1.40	1.53
750	8.98	724.44	4837	1.50	1.53
800	9.82	772.74	4831	1.60	1.53
850	10.66	821.04	4825	1.70	1.53
900	11.51	869.33	4819	1.80	1.53
950	12.34	917.63	4814	1.91	1.53
1000	12.78	965.93	4811	2.01	1.53
1050	13.14	1014.22	4808	2.11	1.53
1100	13.28	1062.52	4807	2.21	1.53
1150	13.70	1110.81	4804	2.31	1.53
1170	14.03	1130.13	4802	**2.35**	1.53
1040	15.00	1004.56	4795	2.10	1.53
1040	15.16	1004.56	4794	2.10	1.53

Peak Shear Stress, τ = 2.35 MPa

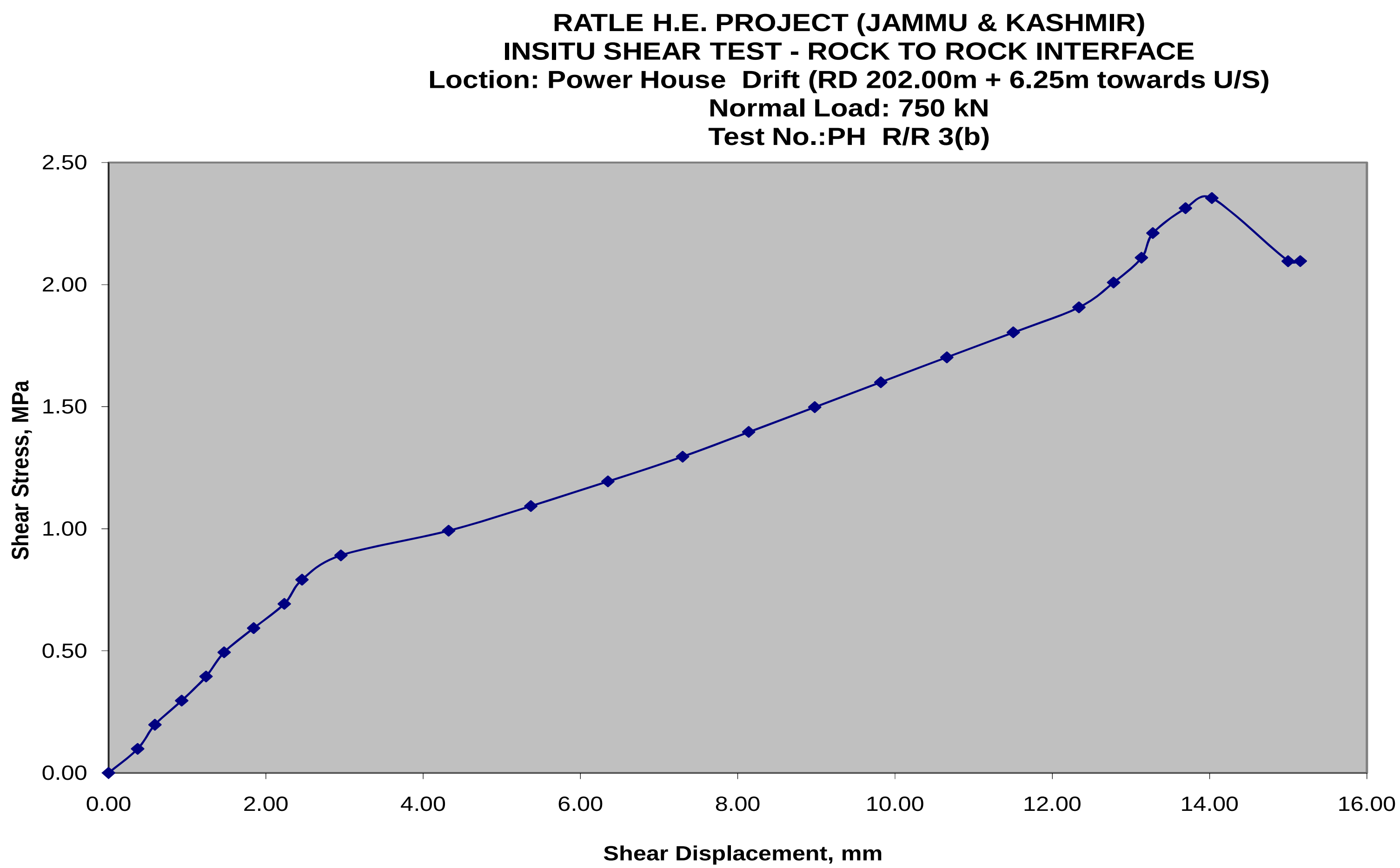


Fig. 64: Shear Stress Vs Shear Displacement

TABLE 35: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: Power House Drift (RD 202.00M + 3.45M Towards U/S)
Applied Normal Load: 1000 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: TP R/R 3(c)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.04
100	0.14	96.59	4899	0.20	2.04
200	0.31	193.19	4898	0.39	2.04
300	1.94	289.78	4886	0.59	2.04
400	2.47	386.37	4883	0.79	2.04
500	3.72	482.96	4874	0.99	2.04
600	4.65	579.56	4867	1.19	2.04
700	6.56	676.15	4854	1.39	2.04
800	7.22	772.74	4849	1.59	2.04
900	8.33	869.33	4842	1.80	2.04
1000	9.04	965.93	4837	2.00	2.04
1100	10.29	1062.52	4828	2.20	2.04
1200	11.44	1159.11	4820	2.40	2.04
1300	12.54	1255.70	4812	2.61	2.04
1370	13.19	1323.32	4808	**2.75**	2.04
1290	15.58	1246.04	4791	2.60	2.04
1290	16.28	1246.04	4786	2.60	2.04

Peak Shear Stress, τ = 2.75MPa

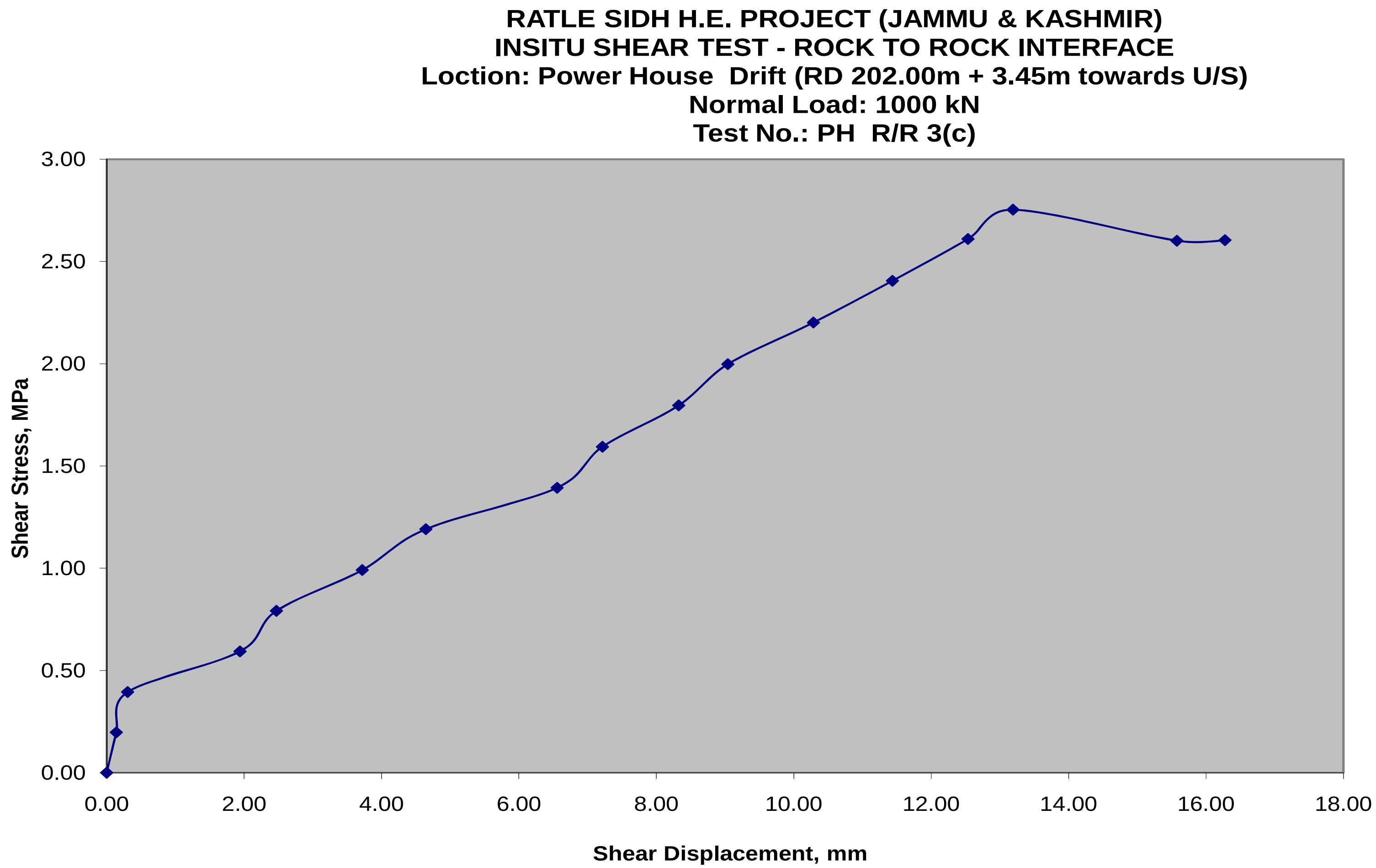


Fig. 65: Shear Stress Vs Shear Displacement

TABLE 36: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: Power House Drift (RD 202.00M + 2.10M Towards U/S)
Applied Normal Load: 1250 KN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: PH R/R 3(d)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.55
100	0.30	96.59	4898	0.20	2.55
200	1.04	193.19	4893	0.39	2.55
300	1.76	289.78	4888	0.59	2.55
400	2.93	386.37	4880	0.79	2.55
500	3.92	482.96	4873	0.99	2.55
600	5.24	579.56	4863	1.19	2.55
700	5.87	676.15	4859	1.39	2.55
800	6.82	772.74	4852	1.59	2.55
900	7.83	869.33	4845	1.79	2.55
1000	9.47	965.93	4834	2.00	2.55
1100	10.46	1062.52	4827	2.20	2.55
1200	11.38	1159.11	4820	2.40	2.55
1300	12.57	1255.70	4812	2.61	2.55
1400	13.56	1352.30	4805	2.81	2.55
1500	14.57	1448.89	4798	3.02	2.55
1590	15.78	1535.82	4790	**3.21**	2.55
1470	17.53	1419.91	4777	2.97	2.55
1470	18.07	1419.91	4774	2.97	2.55

Peak Shear Stress, τ = 3.21MPa

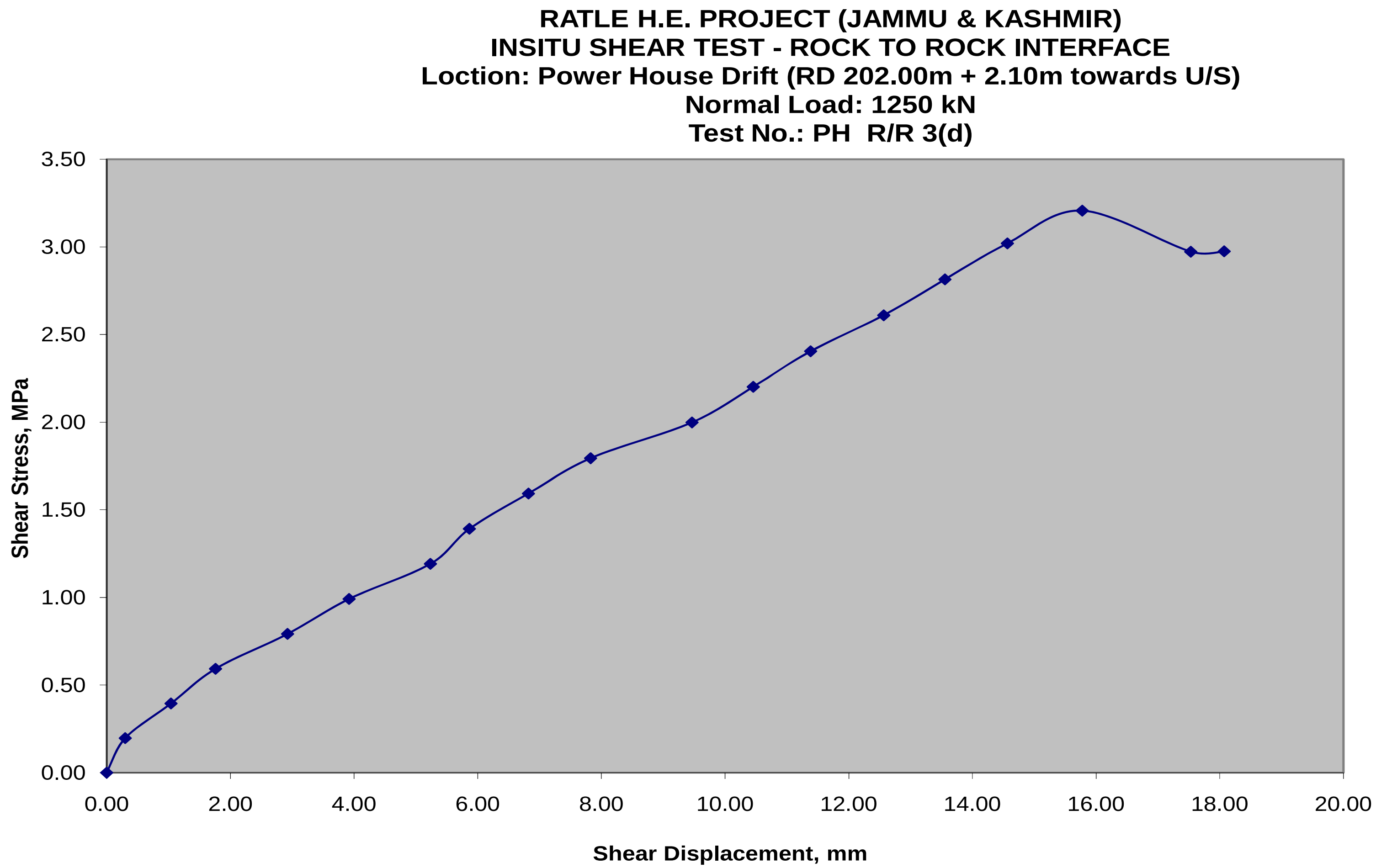


Fig. 66: Shear Stress Vs Shear Displacement

TABLE 37: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: Power House Drift (RD 202.00M)
Applied Normal Load: 1500 kN
Test Block Specification: Rock to Rock Interface
Initial Area: 4900 cm^2
Test No.: PH R/R 3(e)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	3.06
100	0.23	96.59	4898	0.20	3.06
200	0.59	193.19	4896	0.39	3.06
300	1.17	289.78	4892	0.59	3.06
400	1.99	386.37	4886	0.79	3.06
500	3.28	482.96	4877	0.99	3.06
600	4.18	579.56	4871	1.19	3.06
700	4.97	676.15	4865	1.39	3.06
800	6.04	772.74	4858	1.59	3.06
900	6.90	869.33	4852	1.79	3.06
1000	8.22	965.93	4842	1.99	3.06
1100	9.39	1062.52	4834	2.20	3.06
1200	10.46	1159.11	4827	2.40	3.06
1300	12.08	1255.70	4815	2.61	3.06
1400	13.05	1352.30	4809	2.81	3.06
1500	13.50	1448.89	4805	3.02	3.06
1600	14.56	1545.48	4798	3.22	3.06
1700	15.61	1642.07	4791	3.43	3.06
1720	17.05	1661.39	4781	**3.47**	3.06
1540	19.34	1487.53	4765	3.12	3.06
1540	19.91	1487.53	4761	3.12	3.06

Peak Shear Stress, τ = 3.47MPa

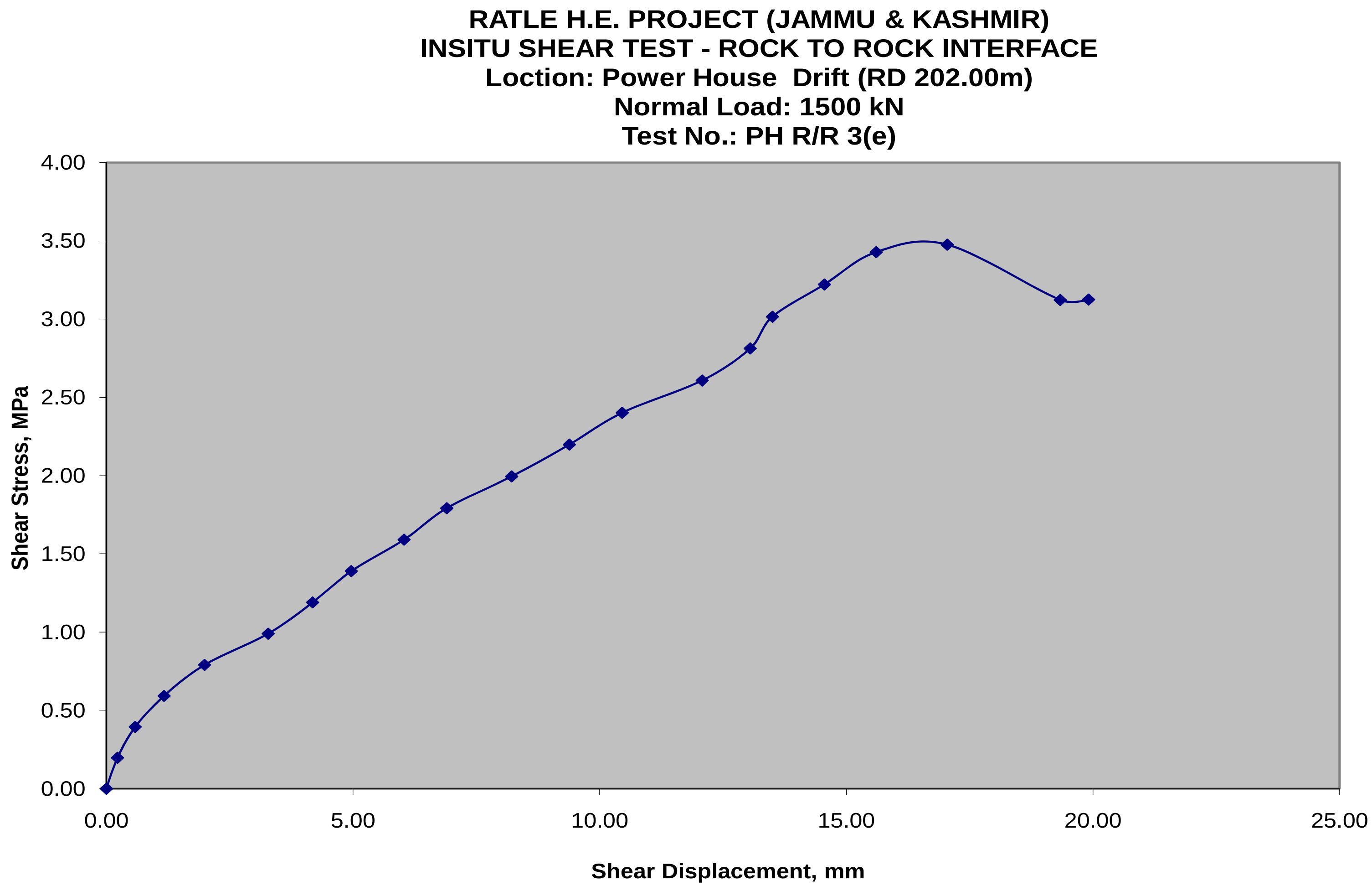


Fig. 67: Shear Stress Vs Shear Displacement

TABLE 38: VALUES OF PEAK & RESIDUAL SHEAR STRENGTH PARAMETERS

Type of Test: Insitu Shear Test
Test No.: PH R/R 3(a) to PH R/R 3(e)
Test Block Specification: Rock to Rock Interface
Location of Test Blocks: Power House Drift (RD 202.00M + 8.40M towards U/S to RD 202.00M)

Normal Stress (MPa)	Peak Shear Stress (MPa)	Peak Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.24	0.47	46
1.53	2.35		
2.04	2.75		
2.55	3.21		
3.06	3.47		

Normal Stress (MPa)	Residual Shear Stress (MPa)	Residual Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.06	0.37	44.5
1.53	2.10		
2.04	2.60		
2.55	2.97		
3.06	3.12		

Power House Drift (RD 202.00 + 8.40M towards U/S to 202.00M)
Rock to Rock Interface

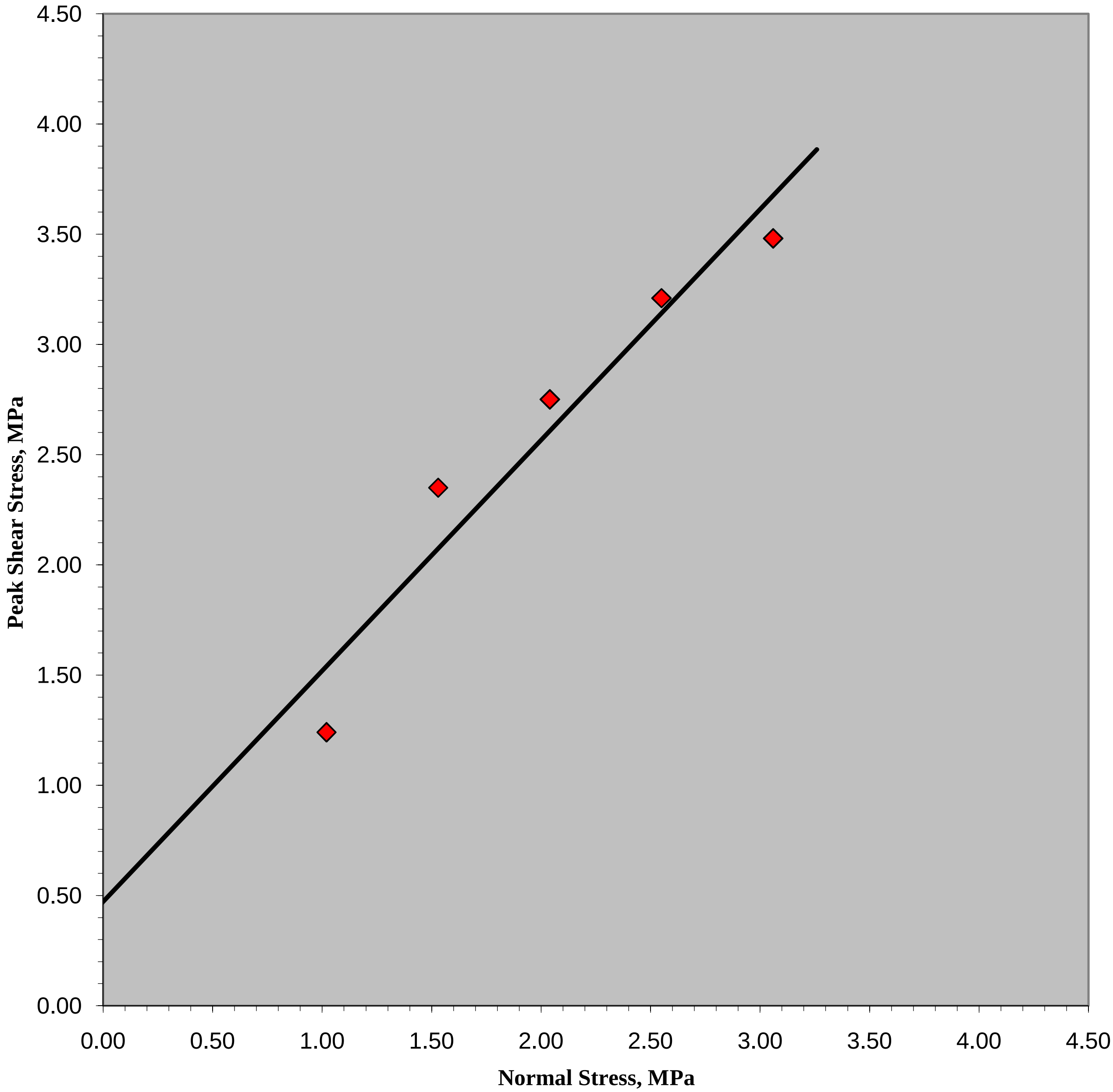


Fig. 68: Peak Shear Stress Vs Normal Stress

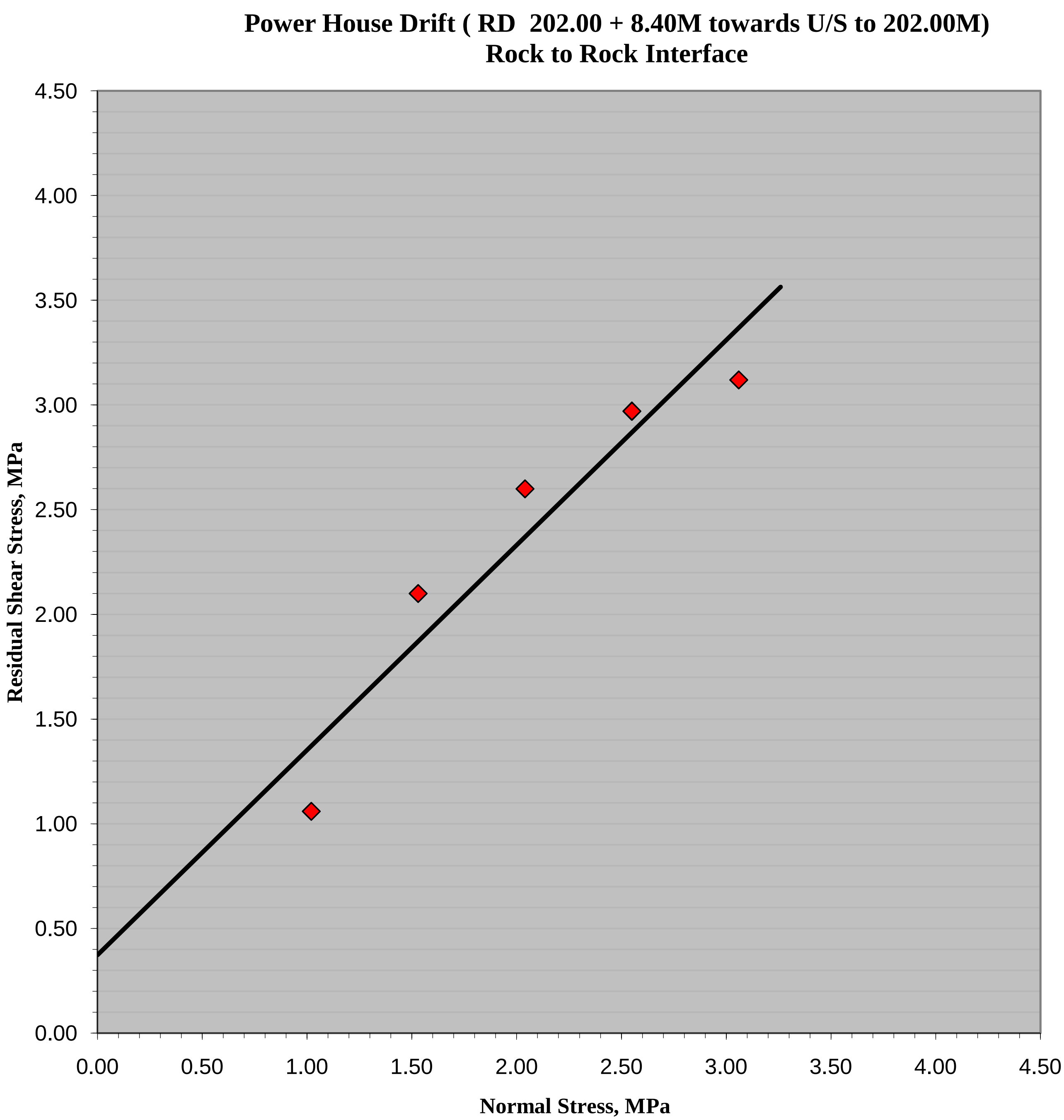


Fig. 69: Residual Shear Stress Vs Normal Stress

TABLE 39: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 19.00M)
Applied Normal Load: 500 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD C/R 4(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.02
50	0.32	48.30	4898	0.10	1.02
100	0.85	96.59	4894	0.20	1.02
150	1.11	144.89	4892	0.30	1.02
200	1.55	193.19	4889	0.40	1.02
250	1.97	241.48	4886	0.49	1.02
300	2.74	289.78	4881	0.59	1.02
350	3.33	338.07	4877	0.69	1.02
400	4.21	386.37	4871	0.79	1.02
450	5.05	434.67	4865	0.89	1.02
500	5.70	482.96	4860	0.99	1.02
550	6.48	531.26	4855	1.09	1.02
600	7.00	579.56	4851	1.19	1.02
650	7.53	627.85	4847	**1.30**	1.02
610	7.88	589.21	4845	1.22	1.02
610	8.60	589.21	4840	1.22	1.02

Peak Shear Stress, τ = 1.30MPa

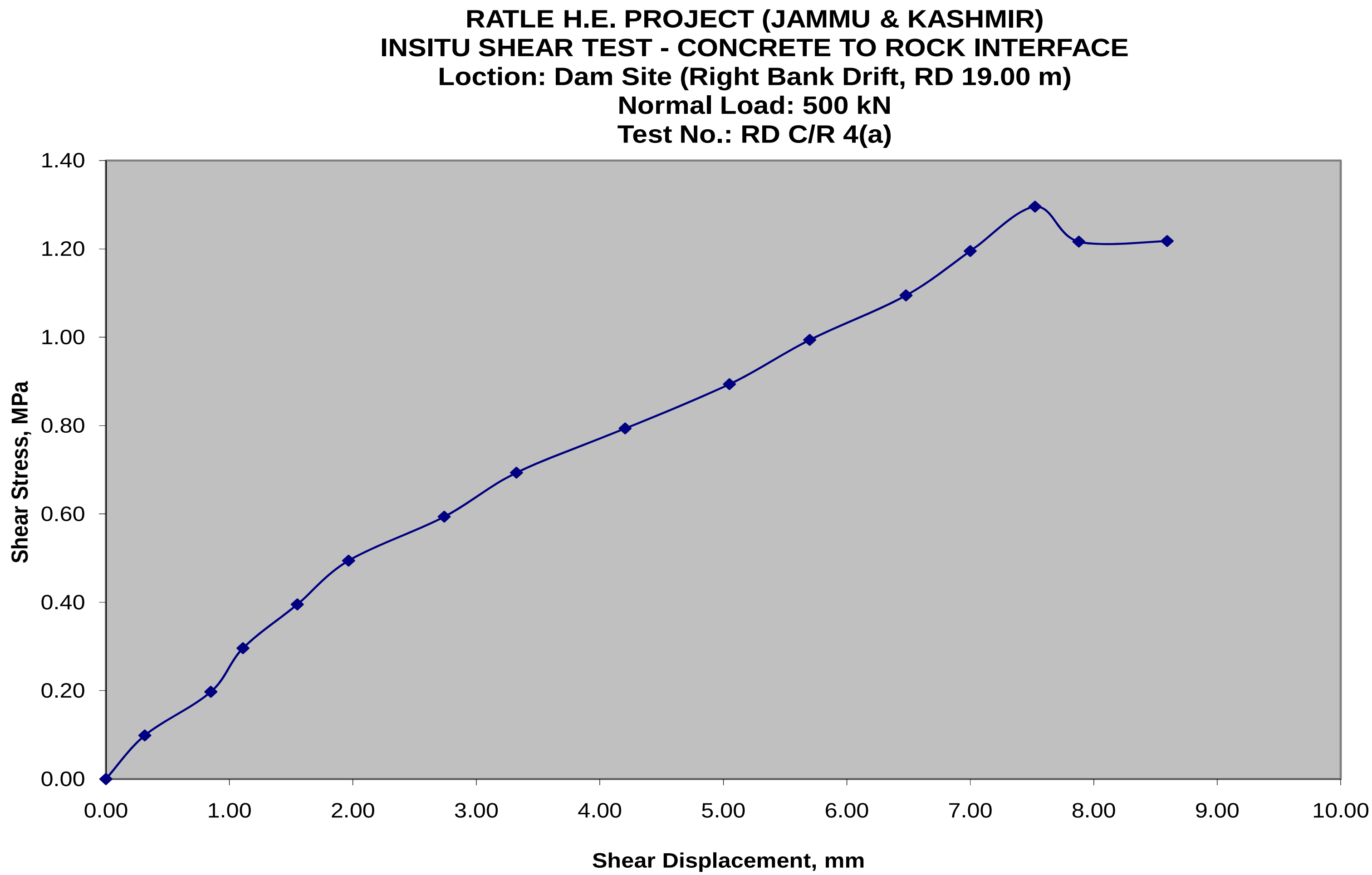


Fig. 70: Shear Stress Vs Shear Displacement

TABLE 40: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 20.00M)
Applied Normal Load: 750 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD C/R 4(b)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.53
50	0.02	48.30	4900	0.10	1.53
100	0.03	96.59	4900	0.20	1.53
150	0.04	144.89	4900	0.30	1.53
200	0.13	193.19	4899	0.39	1.53
250	0.38	241.48	4897	0.49	1.53
300	0.55	289.78	4896	0.59	1.53
350	0.79	338.07	4895	0.69	1.53
400	1.11	386.37	4892	0.79	1.53
450	1.36	434.67	4890	0.89	1.53
500	1.59	482.96	4889	0.99	1.53
550	1.74	531.26	4888	1.09	1.53
600	2.11	579.56	4885	1.19	1.53
650	2.50	627.85	4883	1.29	1.53
700	3.74	676.15	4874	1.39	1.53
750	4.82	724.44	4866	1.49	1.53
800	5.80	772.74	4859	1.59	1.53
850	6.47	821.04	4855	1.69	1.53
900	7.67	869.33	4846	1.79	1.53
950	8.40	917.63	4841	**1.90**	1.53
800	8.99	772.74	4837	1.60	1.53
800	9.32	772.74	4835	1.60	1.53

Peak Shear Stress, τ = 1.90 MPa

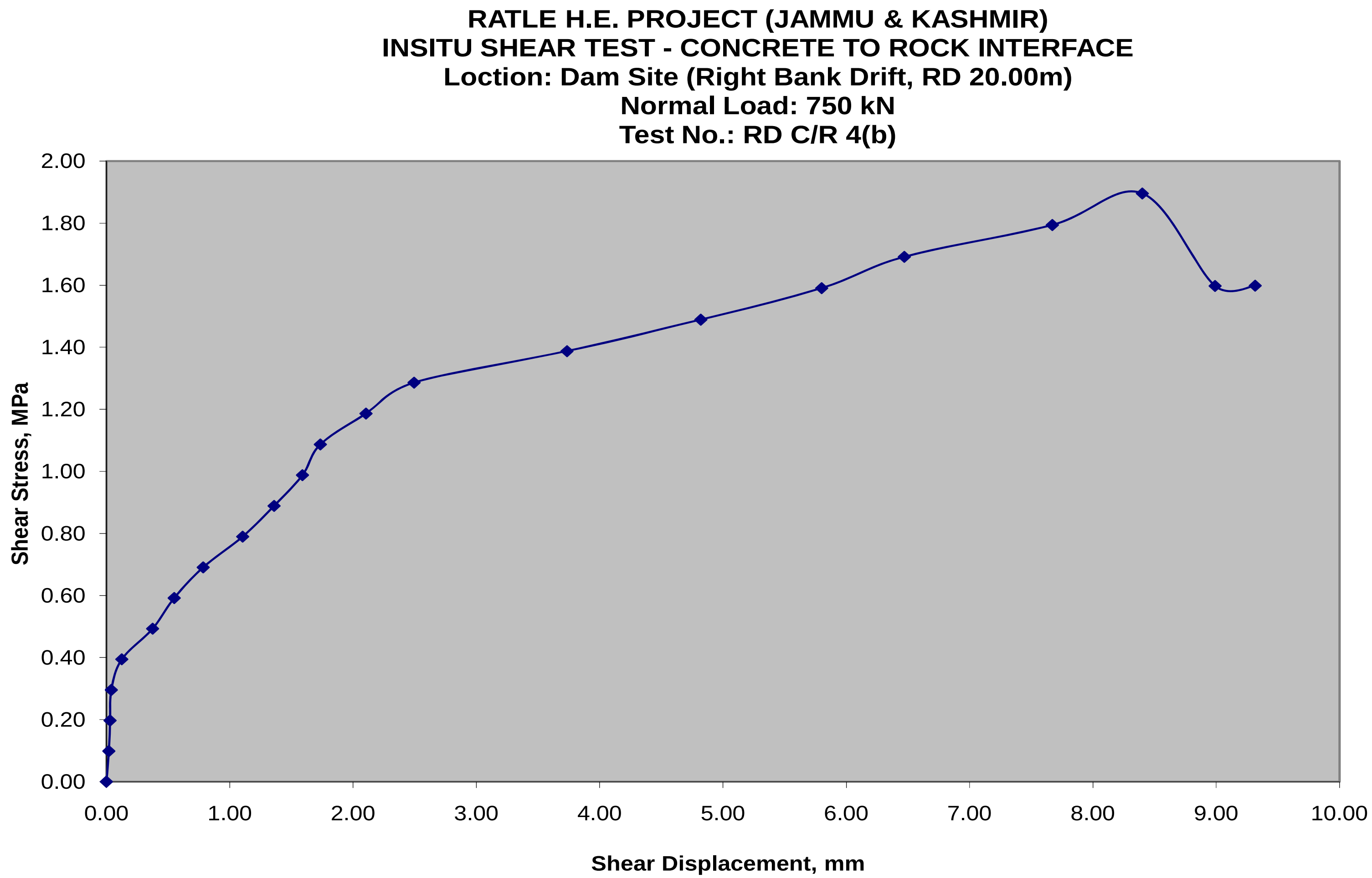


Fig. 71: Shear Stress Vs Shear Displacement

TABLE 41: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 21.40M)
Applied Normal Load: 1000 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD C/R 4(c)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.04
100	0.23	96.59	4898	0.20	2.04
200	0.55	193.19	4896	0.39	2.04
300	1.60	289.78	4889	0.59	2.04
400	2.87	386.37	4880	0.79	2.04
500	3.50	482.96	4876	0.99	2.04
600	4.48	579.56	4869	1.19	2.04
700	5.19	676.15	4864	1.39	2.04
800	5.69	772.74	4860	1.59	2.04
900	6.42	869.33	4855	1.79	2.04
1000	7.22	965.93	4849	1.99	2.04
1100	8.41	1062.52	4841	2.19	2.04
1200	9.78	1159.11	4832	2.40	2.04
1300	11.14	1255.70	4822	2.60	2.04
1320	12.38	1275.02	4813	**2.65**	2.04
1000	15.88	965.93	4789	2.02	2.04
1000	16.14	965.93	4787	2.02	2.04

Peak Shear Stress, τ = 2.65MPa

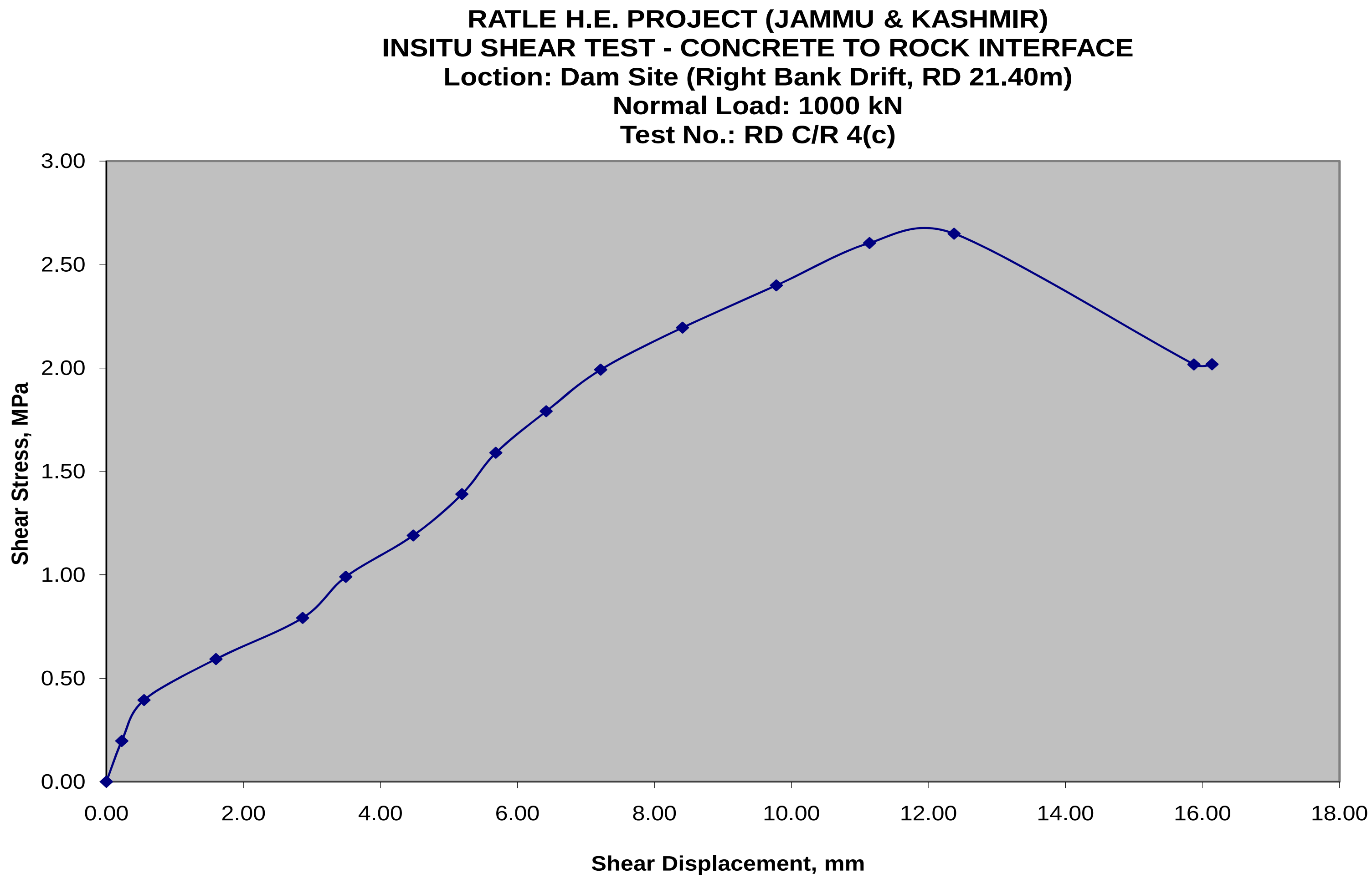


Fig. 72: Shear Stress Vs Shear Displacement

TABLE 42: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 25.00M)
Applied Normal Load: 1250 KN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD C/R 4(d)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.55
100	0.39	96.59	4897	0.20	2.55
200	0.65	193.19	4895	0.39	2.55
300	0.89	289.78	4894	0.59	2.55
400	2.37	386.37	4883	0.79	2.55
500	3.25	482.96	4877	0.99	2.55
600	4.48	579.56	4869	1.19	2.55
700	5.57	676.15	4861	1.39	2.55
800	6.46	772.74	4855	1.59	2.55
900	7.29	869.33	4849	1.79	2.55
1000	9.11	965.93	4836	2.00	2.55
1100	9.74	1062.52	4832	2.20	2.55
1200	10.20	1159.11	4829	2.40	2.55
1300	10.64	1255.70	4826	2.60	2.55
1400	11.03	1352.30	4823	2.80	2.55
1470	12.05	1419.91	4816	**2.95**	2.55
1340	14.43	1294.34	4799	2.70	2.55
1340	14.78	1294.34	4797	2.70	2.55

Peak Shear Stress, τ = 2.95MPa

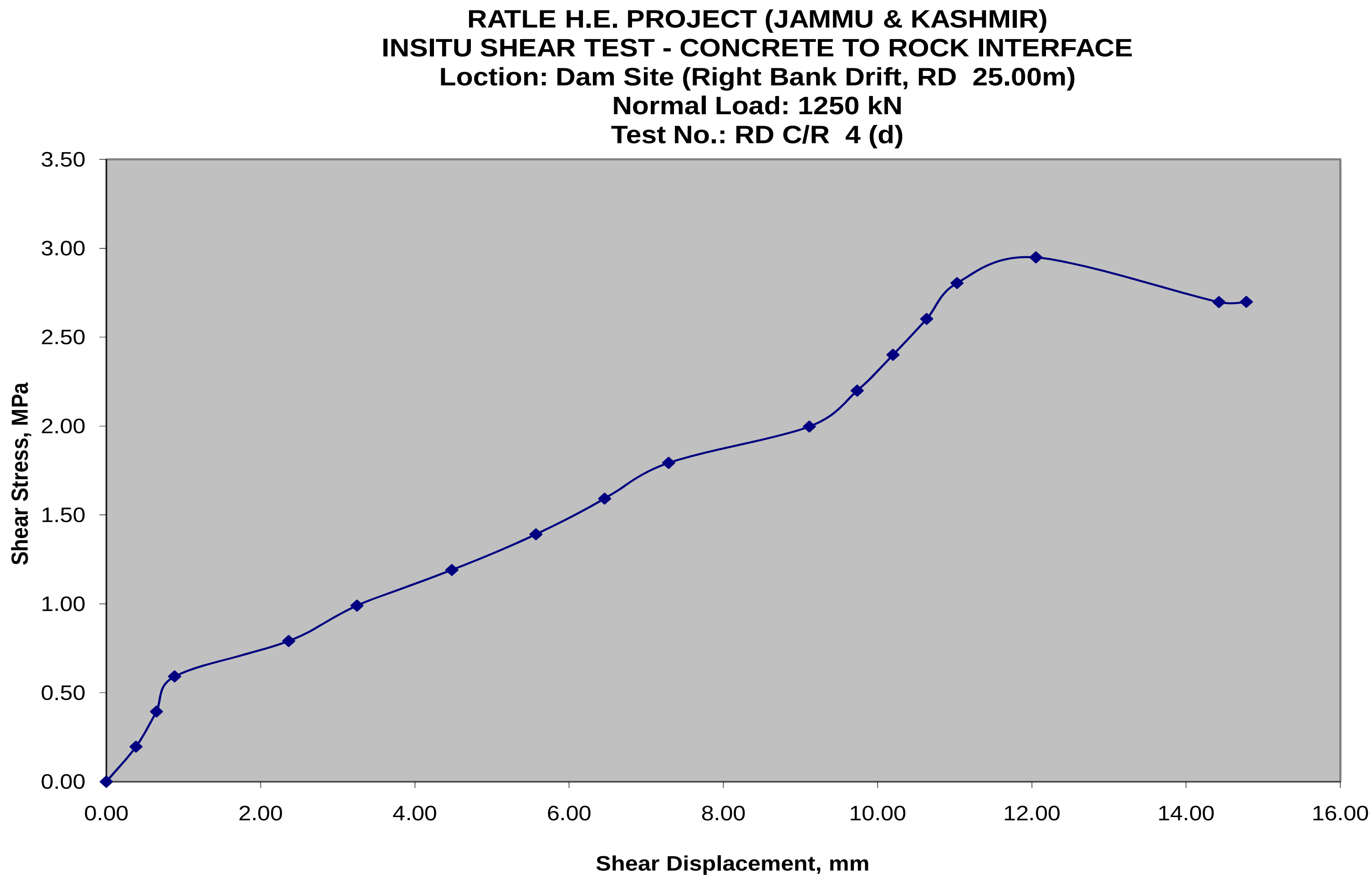


Fig. 73: Shear Stress Vs Shear Displacement

TABLE 43: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (RIGHT BANK DRIFT, RD 26.50M)
Applied Normal Load: 1500 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: RD C/R 4(e)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	3.06
100	0.26	96.59	4898	0.20	3.06
200	0.44	193.19	4897	0.39	3.06
300	0.89	289.78	4894	0.59	3.06
400	1.50	386.37	4890	0.79	3.06
500	2.30	482.96	4884	0.99	3.06
600	3.41	579.56	4876	1.19	3.06
700	4.00	676.15	4872	1.39	3.06
800	4.83	772.74	4866	1.59	3.06
900	5.58	869.33	4861	1.79	3.06
1000	6.18	965.93	4857	1.99	3.06
1100	6.96	1062.52	4851	2.19	3.06
1200	7.66	1159.11	4846	2.39	3.06
1300	8.44	1255.70	4841	2.59	3.06
1400	8.72	1352.30	4839	2.79	3.06
1500	9.08	1448.89	4836	3.00	3.06
1600	9.92	1545.48	4831	3.20	3.06
1700	10.59	1642.07	4826	3.40	3.06
1720	11.78	1661.39	4818	**3.45**	3.06
1520	13.18	1468.21	4808	3.05	3.06
1520	13.35	1468.21	4807	3.05	3.06

Peak Shear Stress, τ = 3.45MPa

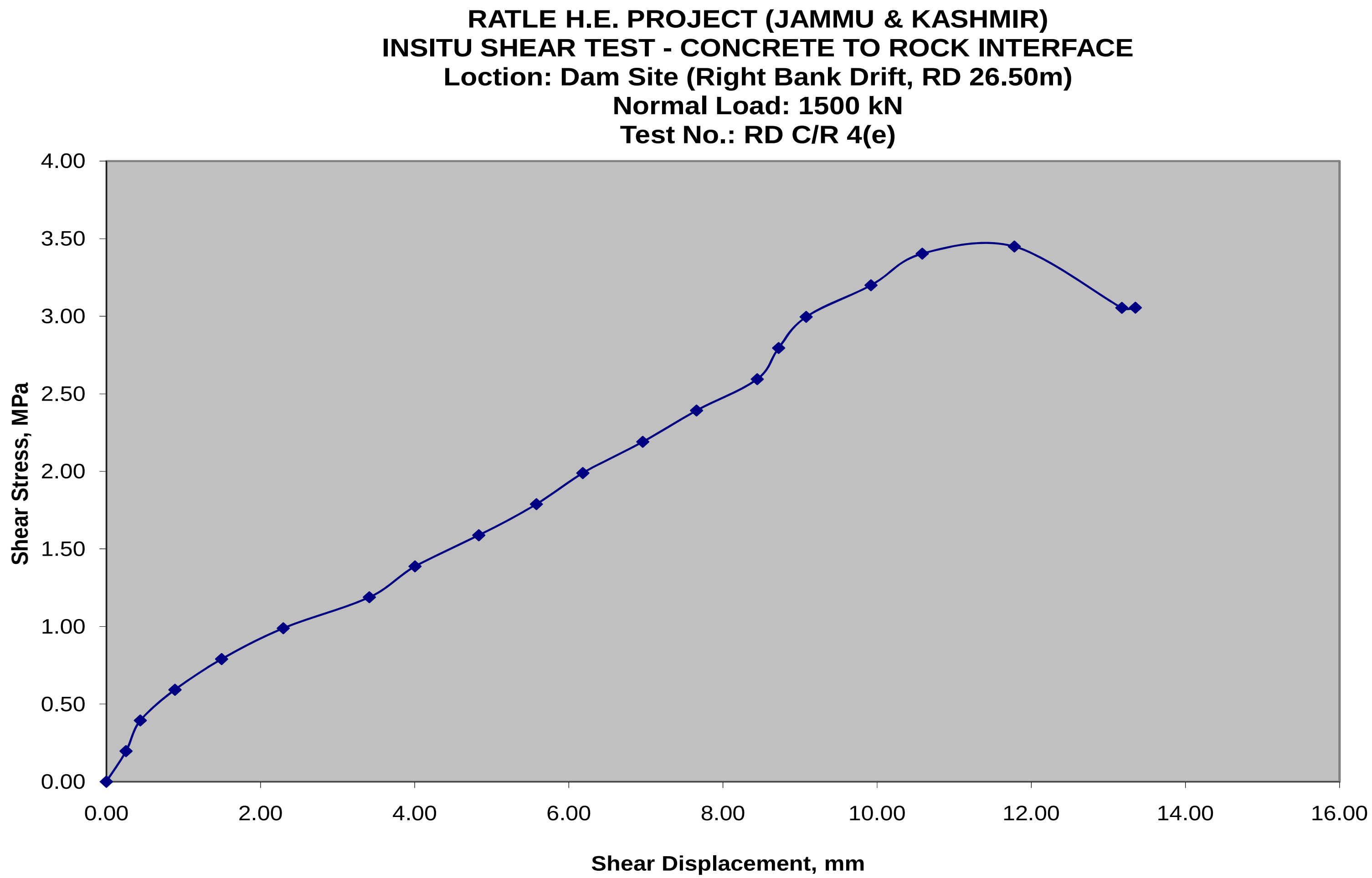


Fig. 74: Shear Stress Vs Shear Displacement

TABLE 44: VALUES OF PEAK & RESIDUAL SHEAR STRENGTH PARAMETERS

Type of Test: Insitu Shear Test
Test No.: RD C/R 4(a) to RD C/R 4(e)
Test Block Specification: Concrete to Rock Interface
Location of Test Blocks: Dam Site (Right Bank Drift, RD 19.00M to RD 26.50M)

Normal Stress (MPa)	Peak Shear Stress (MPa)	Peak Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.30	0.31	46
1.53	1.90		
2.04	2.65		
2.55	2.95		
3.06	3.45		

Normal Stress (MPa)	Residual Shear Stress (MPa)	Residual Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.22	0.21	43
1.53	1.60		
2.04	2.02		
2.55	2.70		
3.06	3.05		

Dam Site (Right Bank Drift, RD 19.00M to 26.50M)
Concrete to Rock Interface

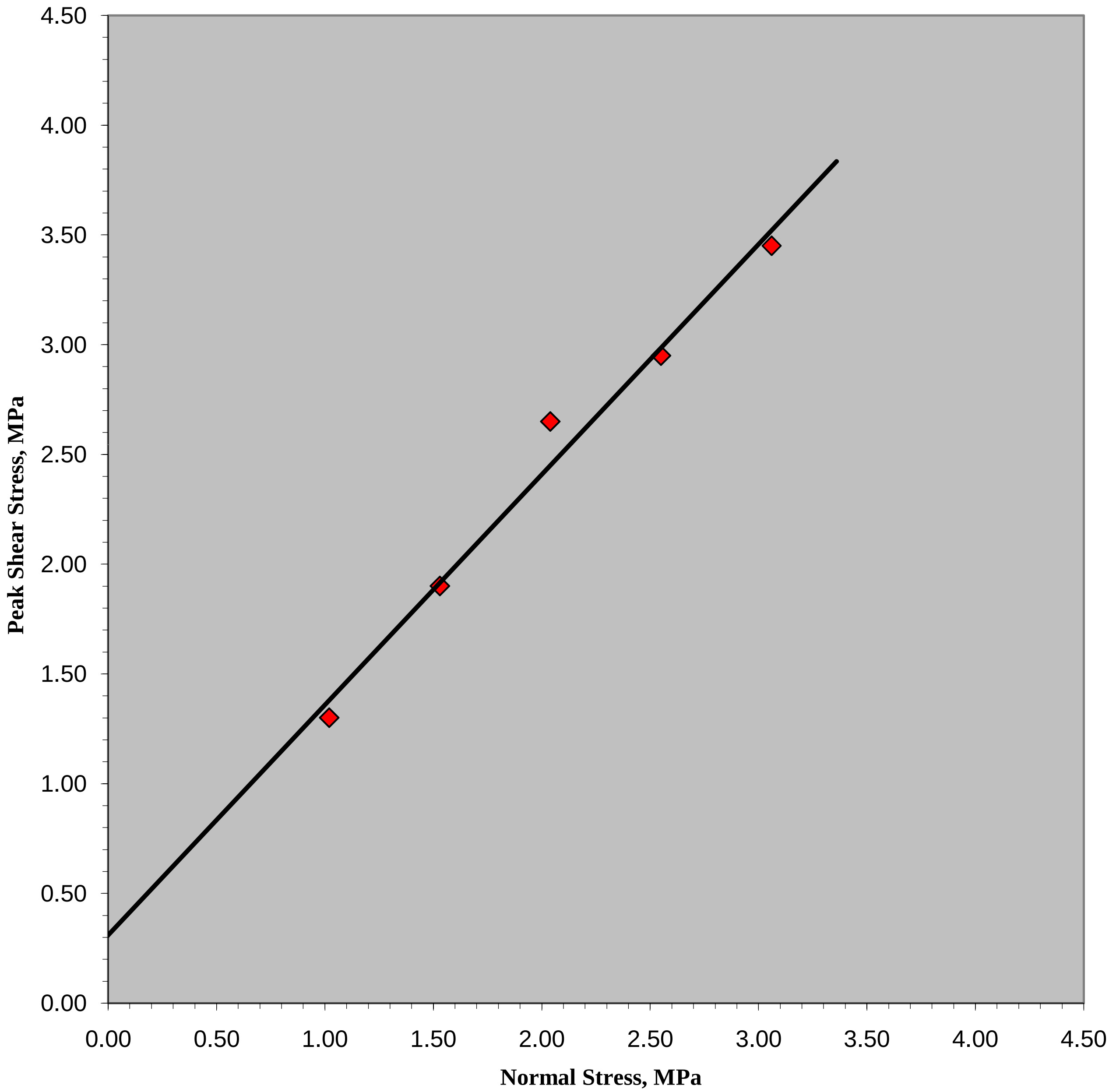


Fig. 75: Peak Shear Stress Vs Normal Stress

Dam Site (Right Bank Drift, RD 19.00M to 26.50M)
Concrete to Rock Interface

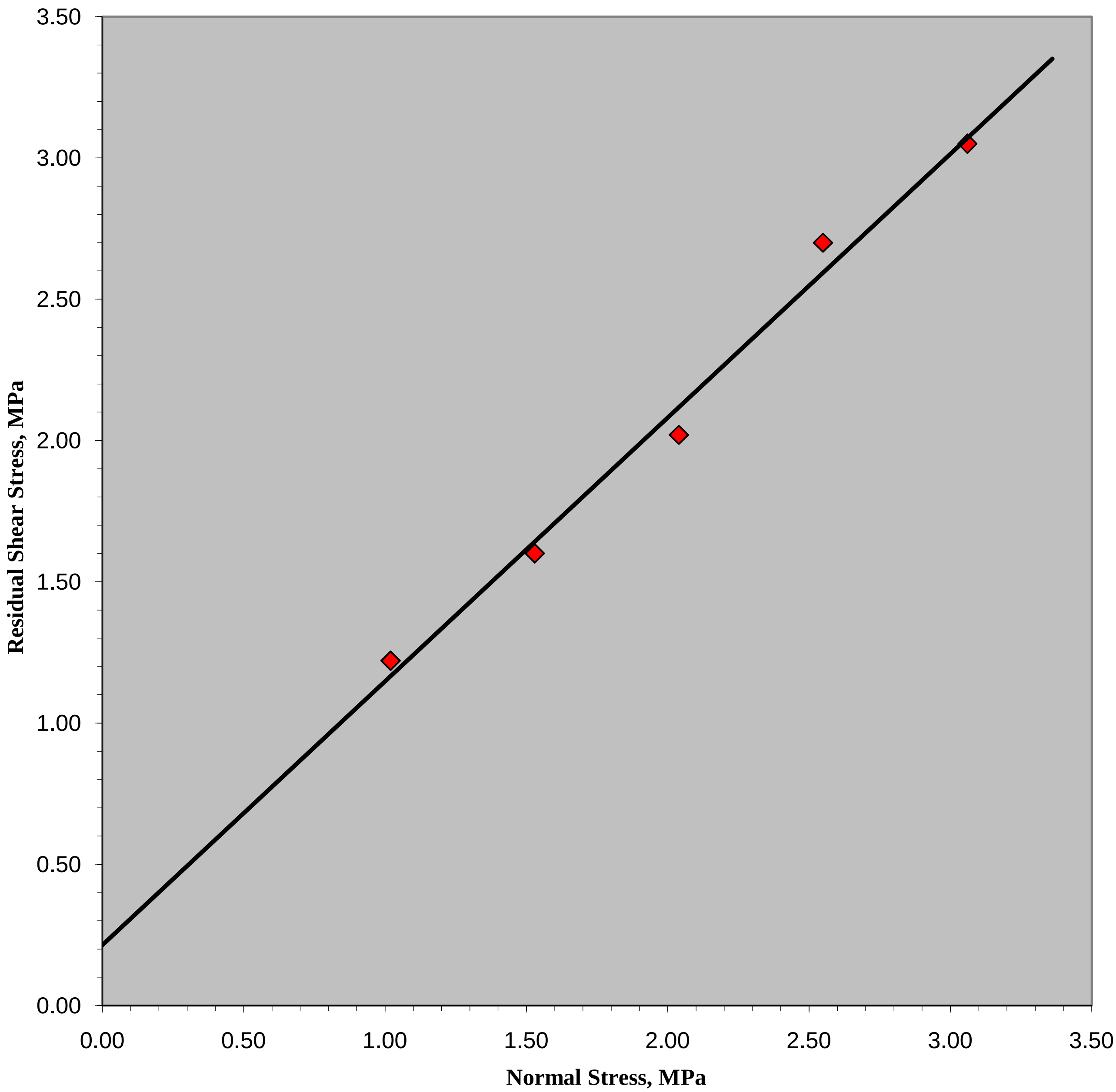


Fig. 76 : Residual Shear Stress Vs Normal Stress

TABLE 45: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 18.60M)
Applied Normal Load: 500 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD C/R 5(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.02
50	0.17	48.30	4899	0.10	1.02
100	0.24	96.59	4898	0.20	1.02
150	0.34	144.89	4898	0.30	1.02
200	0.87	193.19	4894	0.39	1.02
250	1.32	241.48	4891	0.49	1.02
300	3.96	289.78	4872	0.59	1.02
350	4.67	338.07	4867	0.69	1.02
400	5.53	386.37	4861	0.79	1.02
450	6.29	434.67	4856	0.90	1.02
500	7.08	482.96	4850	1.00	1.02
550	7.87	531.26	4845	1.10	1.02
600	8.66	579.56	4839	1.20	1.02
650	9.44	627.85	4834	**1.30**	1.02
600	10.23	579.56	4828	1.20	1.02
600	10.61	579.56	4826	1.20	1.02

Peak Shear Stress, τ = 1.30MPa

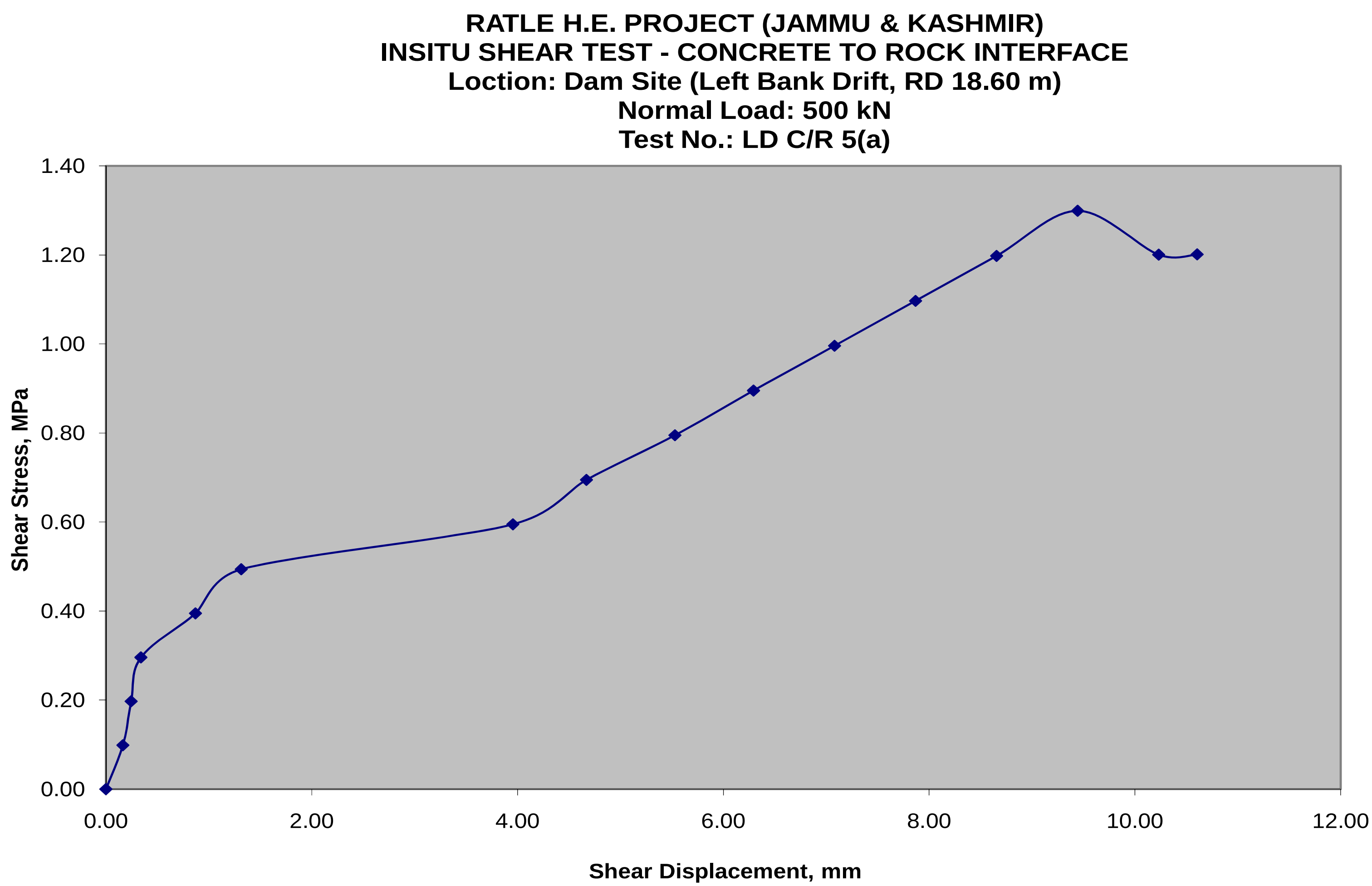


Fig.77: Shear Stress Vs Shear Displacement

TABLE 46: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 20.70M)
Applied Normal Load: 750 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD C/R 5(b)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	1.53
50	0.38	48.30	4897	0.10	1.53
100	0.64	96.59	4896	0.20	1.53
150	0.82	144.89	4894	0.30	1.53
200	1.12	193.19	4892	0.39	1.53
250	1.84	241.48	4887	0.49	1.53
300	2.60	289.78	4882	0.59	1.53
350	3.24	338.07	4877	0.69	1.53
400	3.85	386.37	4873	0.79	1.53
450	4.65	434.67	4867	0.89	1.53
500	5.55	482.96	4861	0.99	1.53
550	6.70	531.26	4853	1.09	1.53
600	7.78	579.56	4846	1.20	1.53
650	8.31	627.85	4842	1.30	1.53
700	8.98	676.15	4837	1.40	1.53
750	9.60	724.44	4833	1.50	1.53
800	10.21	772.74	4829	1.60	1.53
850	10.83	821.04	4824	1.70	1.53
900	11.46	869.33	4820	1.80	1.53
950	11.86	917.63	4817	1.90	1.53
1000	12.55	965.93	4812	2.01	1.53
1010	12.99	975.59	4809	**2.03**	1.53
890	15.88	859.67	4789	1.80	1.53
890	16.56	859.67	4784	1.80	1.53

Peak Shear Stress, τ = 2.03 MPa

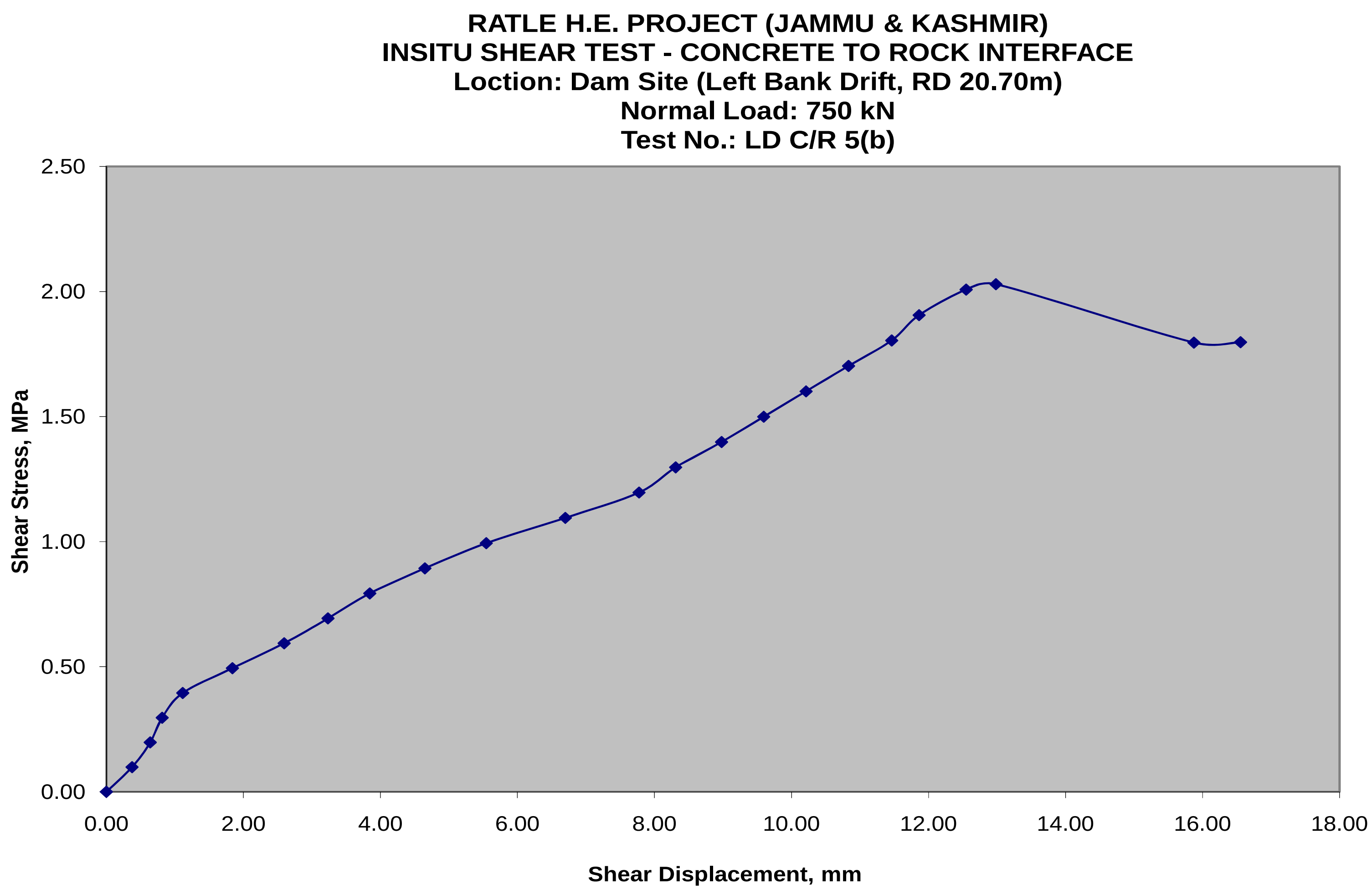


Fig.78: Shear Stress Vs Shear Displacement

TABLE 47: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 21.50M)
Applied Normal Load: 1000 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD C/R 5(c)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.04
100	0.54	96.59	4896	0.20	2.04
200	0.78	193.19	4895	0.39	2.04
300	1.07	289.78	4893	0.59	2.04
400	1.73	386.37	4888	0.79	2.04
500	2.37	482.96	4883	0.99	2.04
600	2.80	579.56	4880	1.19	2.04
700	3.78	676.15	4874	1.39	2.04
800	4.25	772.74	4870	1.59	2.04
900	4.90	869.33	4866	1.79	2.04
1000	6.08	965.93	4857	1.99	2.04
1100	6.47	1062.52	4855	2.19	2.04
1200	7.18	1159.11	4850	2.39	2.04
1300	7.73	1255.70	4846	2.59	2.04
1350	9.28	1304.00	4835	**2.70**	2.04
990	10.75	956.27	4825	1.98	2.04
990	11.06	956.27	4823	1.98	2.04

Peak Shear Stress, τ = 2.70MPa

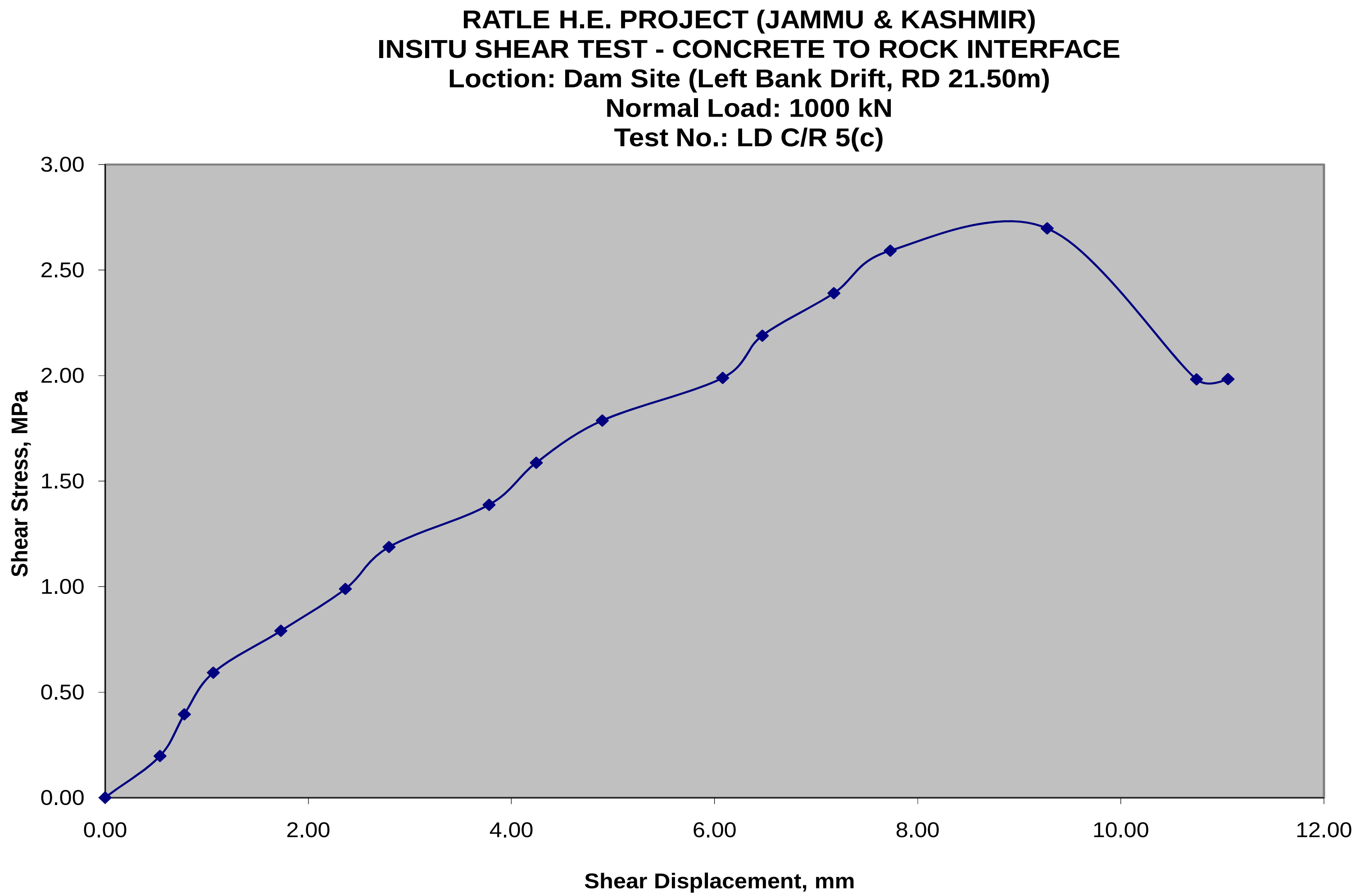


Fig. 79: Shear Stress Vs Shear Displacement

TABLE 48: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 22.50M)
Applied Normal Load: 1250 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD C/R 5(d)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	2.55
100	0.23	96.59	4898	0.20	2.55
200	0.62	193.19	4896	0.39	2.55
300	0.88	289.78	4894	0.59	2.55
400	1.11	386.37	4892	0.79	2.55
500	1.77	482.96	4888	0.99	2.55
600	2.71	579.56	4881	1.19	2.55
700	3.27	676.15	4877	1.39	2.55
800	4.11	772.74	4871	1.59	2.55
900	5.13	869.33	4864	1.79	2.55
1000	6.40	965.93	4855	1.99	2.55
1100	7.11	1062.52	4850	2.19	2.55
1200	8.02	1159.11	4844	2.39	2.55
1300	9.23	1255.70	4835	2.60	2.55
1400	10.20	1352.30	4829	2.80	2.55
1500	10.41	1448.89	4827	3.00	2.55
1510	10.72	1458.55	4825	**3.02**	2.55
1440	16.03	1390.93	4788	2.91	2.55
1440	16.33	1390.93	4786	2.91	2.55

Peak Shear Stress, τ = 3.02MPa

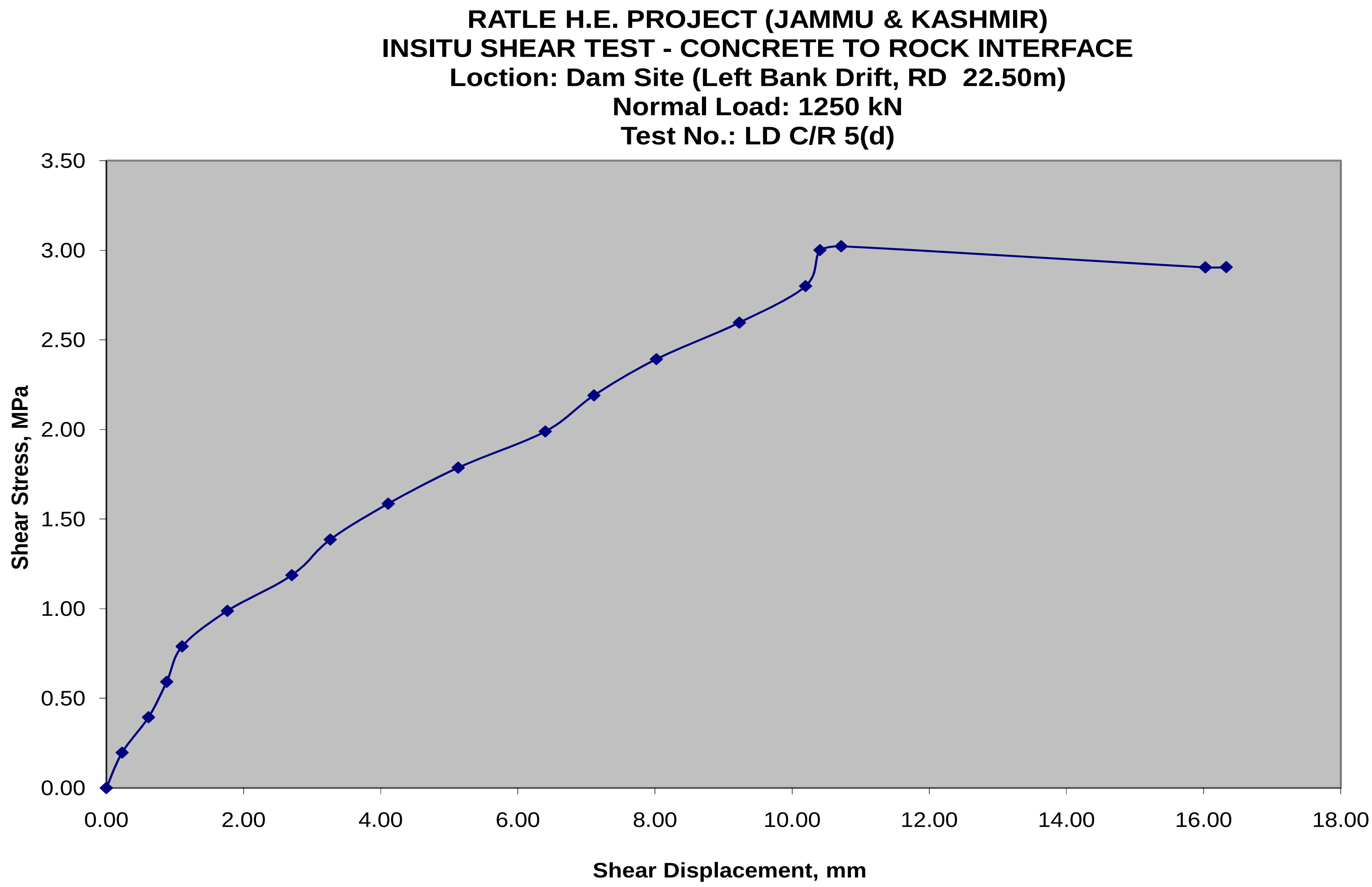


Fig. 80: Shear Stress Vs Shear Displacement

TABLE 49: INSITU SHEAR TEST DATA SHEET

Project: RATLE H.E. PROJECT (JAMMU & KASHMIR)
Test Block Location: DAM SITE (LEFT BANK DRIFT, RD 23.60M)
Applied Normal Load: 1500 kN
Test Block Specification: Concrete to Rock Interface
Initial Area: 4900 cm^2
Test No.: LD C/R 5(a)

Applied Shear Force (P_{sa})	Shear Displacement	Total Shear Force (P_s)	Corrected Area (A')	Shear Stress, τ	Normal Stress, σ
(kN)	(mm)	(kN)	(cm^2)	(MPa)	(MPa)
0	0.00	0.00	4900	0.00	3.06
100	0.28	96.59	4898	0.20	3.06
200	0.48	193.19	4897	0.39	3.06
300	0.94	289.78	4893	0.59	3.06
400	1.13	386.37	4892	0.79	3.06
500	1.49	482.96	4890	0.99	3.06
600	1.79	579.56	4888	1.19	3.06
700	2.41	676.15	4883	1.38	3.06
800	3.26	772.74	4877	1.58	3.06
900	3.61	869.33	4875	1.78	3.06
1000	4.89	965.93	4866	1.99	3.06
1100	5.87	1062.52	4859	2.19	3.06
1200	7.11	1159.11	4850	2.39	3.06
1300	8.31	1255.70	4842	2.59	3.06
1400	9.13	1352.30	4836	2.80	3.06
1500	10.00	1448.89	4830	3.00	3.06
1600	10.99	1545.48	4823	3.20	3.06
1700	11.75	1642.07	4818	3.41	3.06
1800	12.58	1738.67	4812	**3.61**	3.06
1560	15.58	1506.84	4791	3.15	3.06
1560	16.01	1506.84	4788	3.15	3.06

Peak Shear Stress, τ = 3.61MPa

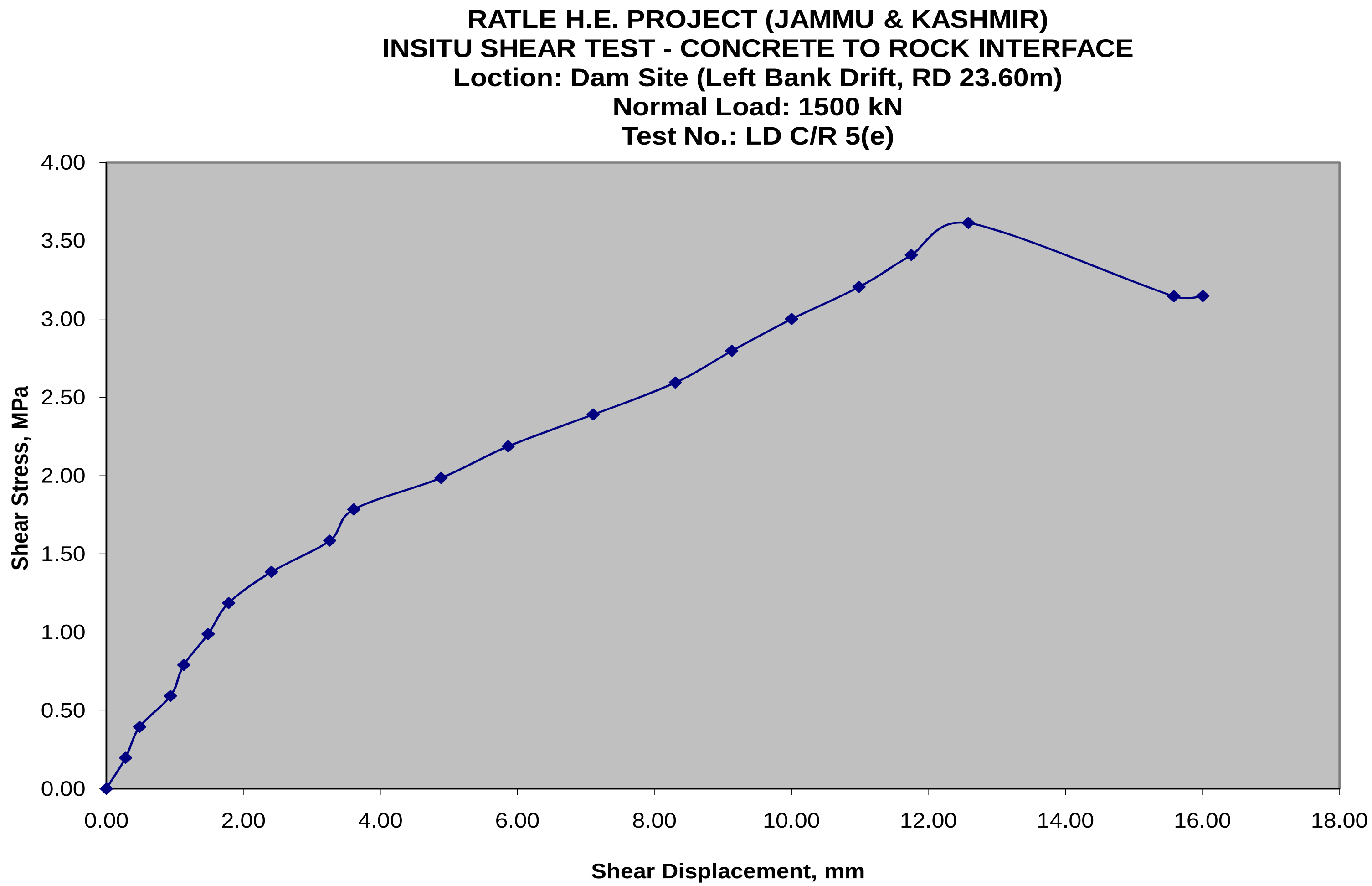


Fig. 81: Shear Stress Vs Shear Displacement

TABLE 50: VALUES OF PEAK & RESIDUAL SHEAR STRENGTH PARAMETERS

Type of Test: Insitu Shear Test
Test No.: LD C/R 5(a) to LD C/R 5(e)
Test Block Specification: Concrete to Rock Interface
Location of Test Blocks: Dam Site (Left Bank Drift, RD 18.60M to RD 23.60M)

Normal Stress (MPa)	Peak Shear Stress (MPa)	Peak Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.30	0.29	48
1.53	2.03		
2.04	2.70		
2.55	3.02		
3.06	3.61		

Normal Stress (MPa)	Residual Shear Stress (MPa)	Residual Shear Strength Parameters	
		C (MPa)	ϕ (Degree)
1.02	1.20	0.21	44
1.53	1.80		
2.04	1.98		
2.55	2.91		
3.06	3.15		

Dam Site (Left Bank Drift, RD 18.60M to 23.60M)
Concrete to Rock Interface

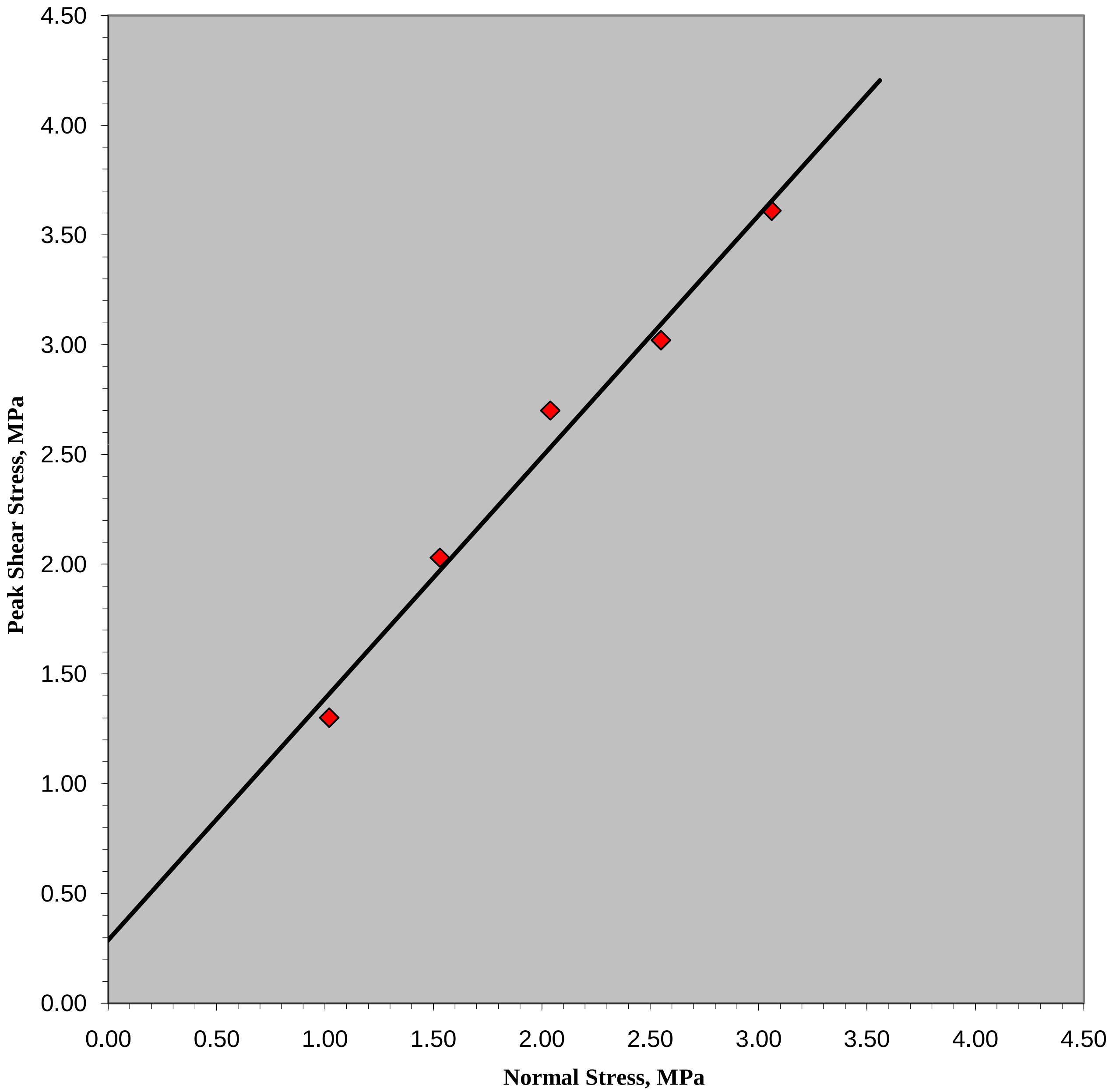


Fig. 82: Peak Shear Stress Vs Normal Stress

Dam Site (Left Bank Drift, RD 18.60M to 23.60M)
Concrete to Rock Interface

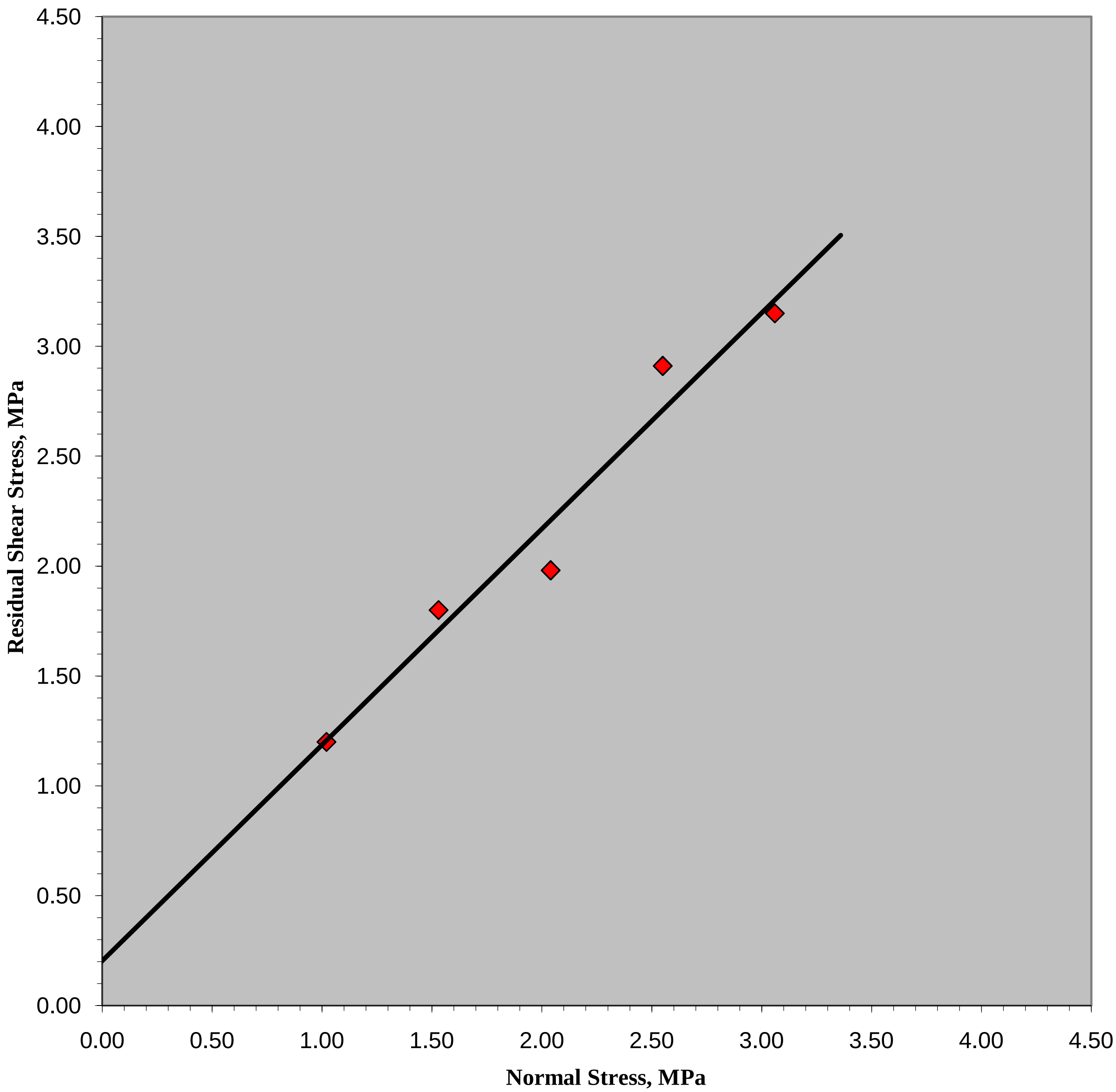


Fig. 83 : Residual Shear Stress Vs Normal Stress