# PACKER TEST AND CURTAIN GROUTING IN FOUNDATION OF DAM

A DISSERTATION

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**GEOTECHNICAL ENGINEERING**

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I **RAJEEV KUMAR**, Roll No. **2K15GTE14** of M.TECH GEOTECHNICAL ENGINEERING, hereby declare that the project Dissertation titled "**PACKER TEST AND CURTAIN GROUTING IN FOUNDATION OF DAM"** which is submitted by me to the Department of **Civil Engineering,** Delhi technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title or recognition.

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# CERTIFICATE

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**RAJEEV KUMAR**

# ABSTRACT

Curtain grouting is an important component for the Foundation treatment of Dam and other hydraulic structure. The main purpose of curtain grouting is to form a zone of low permeability up to a design depth on the upstream side of the dam. Before performing the curtain grouting, the Packer test is performed to know whether grouting is necessary or not, and if it is necessary then at what pressure grouting is to be carried out. The grouting and the drainage pipe install in downstream side controls the uplift pressure and piping which are main cause of failure of concrete gravity dam. This thesis deals with performing packer test to know the permeability of rock mass and providing grout curtain to Pare Dam foundation from Block no. 03,04,05,06,07 and 10 in different-2 bore hole and at different-2 elevation level which was provided by design and drawing contractor SNC.LAVALIN and sanctioned by NEEPCO.

The Lugeon value generally observed during packer test lies less than 1.0 but sometimes it goes beyond15.00. The Lugeon value less than1.0 represent that the Condition of Rock Mass Discontinuities is very tight. The rock mass below the foundation is mainly consisting of sand stone. The single line grout curtain has been effectively performed below the foundation, despite very poor strata having maximum pre – grout permeability up to 54 Lugeon. Each bore hole is consisting of 76 mm in diameter and depth varies from 28 m to 43m.

***Keywords:*** Packer test; Lugeon value; Drilling; Grouting; Dam; Foundation; Drainage hole

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# Chapter 1

# INTRODUCTION

## 1.1. Physical Features of the Project Area

### 1.1.1 Geological Disposition

The Pare Hydroelectric Project, is located in the Papumpare district of Arunachal Pradesh (Longitude: 93°48’56”, Latitude: 27°14’13”) which is about 5 Km downstream of the Ranganadi stage-I Tail Race. The project area falls in the Upper Siwalik formation (Tertiary Group) comprising brownish & grey coloured, fine to medium grained concretionary, soft, and friable, pebble impregnated, salt-pepper textured sandstone, sand rock and pebble beds which are moderately jointed.

### 1.1.2 Climate

The rainfall in the basin is quite high and varies from about 1000 mm in higher reaches to about 5750 mm in the foot hills spread over 8-9 months excepting the drier days in winter. The upper regions also receive precipitation from snow clad mountains which contribute to the river flow during lean period. On this account fairly high perennial discharge continues to be available in the river all the year around. Such a favourable river discharge pattern and the fact that a total fall of more than 3000 m is available in the river system make it very attractive for developing a series of hydro-electric power stations on the main river and its tributaries.

The climate of the project area is humid with an average annual rainfall in the state of Arunachal Pradesh at about 2900 mm. Winter rains are also quite frequent. Temperature is moderate, generally highest temperature in summer goes up to 36°C and lowest temperature in winter is 4°C.

### 1.1.3 Population

The population of Arunachal Pradesh is 1091117 (2001 estimate). The people are of Mongoloid stock with heritage of arts and crafts, enchanting folk songs with their own distinct and diverse culture, dialects and lifestyles. There are 20 major tribes in the State namely Adi, Nyishi, Apatani, Bugun, Galo, Hrusso, Koro, Meyor, Monpa, Tagin, Mishmi, Sajolang, Sartang, Tai Khamti, Yobin, Singpho, Sherduken, Khamba, Tangshang and Memba. The State has a literacy rate of 54.74%.The project area is inhabited mainly by the two tribes of Arunachal Pradesh namely Apatani and Nishi. The habitation in the area consists of small villages and people practice cultivation on patches of hill slopes. Cattle farming are also widely practiced by the local community.

### 1.1.4 Project Proposal

Pare H.E. Project has been contemplated as a run-of-the river scheme situated in the Papumpare district of Arunachal Pradesh. The project envisages utilisation of water of both river Dikrong and water discharged through tailrace of RHEP-Stage-1 water through a gross head of 75.35 m (design head 67.36 m) for generation of a maximum of

110 MW of power. The project comprises a concrete diversion dam of 78 m height above the deepest foundation level, across the river Dikrong at about 5 Km downstream of existing RHEP-Stage – I tail race channel. The water conductor system consists of a 2.81 Km long concrete lined 7.5 m diameter Horseshoe section head race tunnel leading to a 58 m high surge shaft of 18 m diameter . Pressure shaft of 6.4 m diameter takes off from the surge shaft and is designed for a maximum discharge of 185 cumecs, 6.4 m diameter pressure shaft bifurcates into two penstock of 4.5 M diameter each leading to two vertical Francis turbines of 55 MW capacity installed in a surface power house on the right bank of the river Dikrong, near the village Sopo. The project is scheduled to be completed in December 2011 in 4 years after completion of 2nd stage activities by December 2007. The infrastructure facilities constructed for Ranganadi H.E. Project, Stage-I are already available and can be utilised for the proposed project. Additional requirement shall be developed concurrently with the process of obtaining various Govt. clearances.

The energy generation shall be 506.424 million units (MU) in a 90% dependable year at 95% plant availability. The saleable energy at bus bar shall be 441.197 MU after deducting losses and water royalty.

The project has been estimated to cost Rs. 503.99 Crores at June 2007 price level and Rs 586.85 Crores including interest during construction at commissioning of the project in December 2011. The levellised tariff and the first year tariff have been worked out as Rs. 2.01 and Rs. 2.38 per kW/h considering 11.25% interest rate, 14% return on equity, 12% water royalty and 11.10% discounting rate.

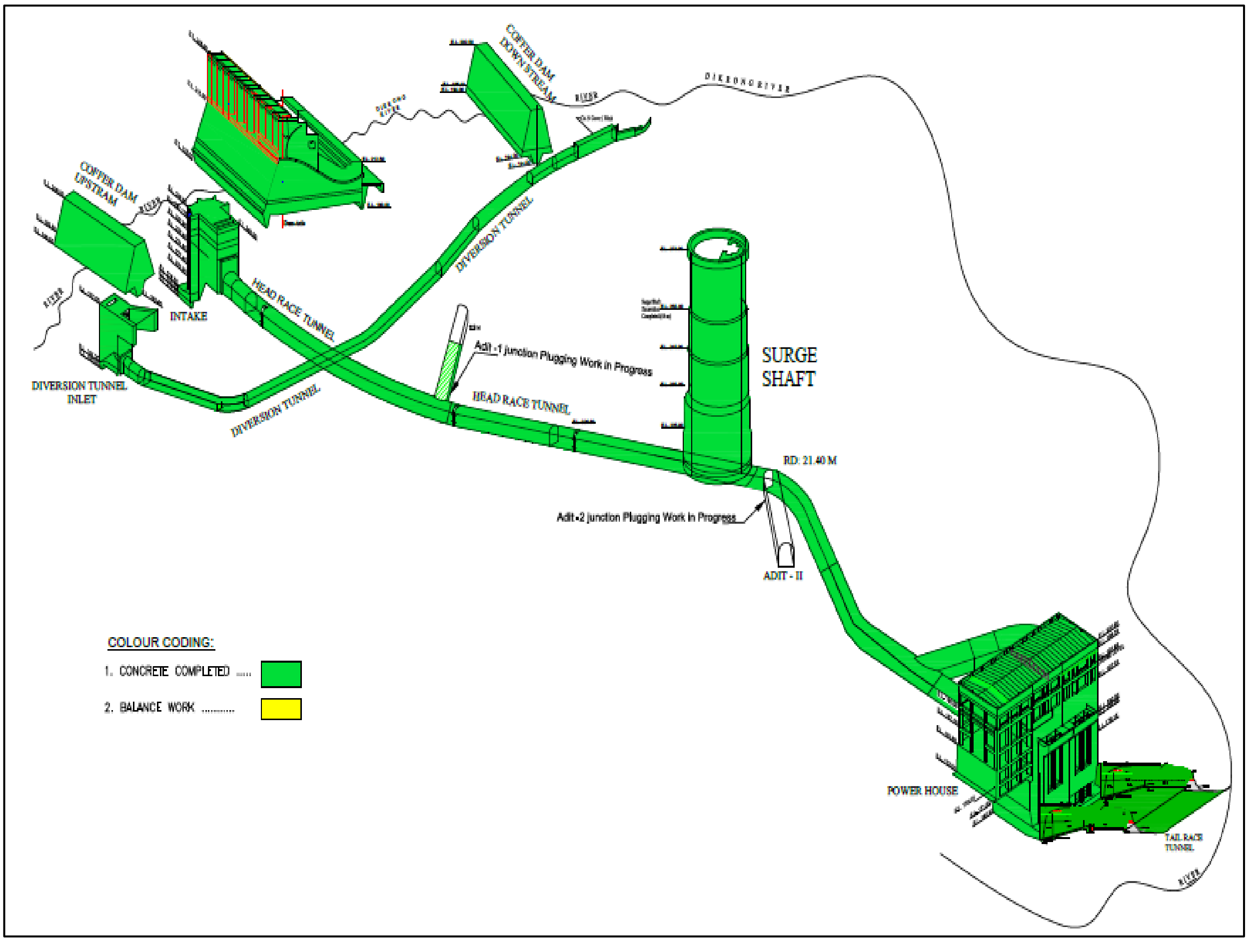



Fig 1.1 Pictorial View of Pare Hydroelectric Project (PaHEP)

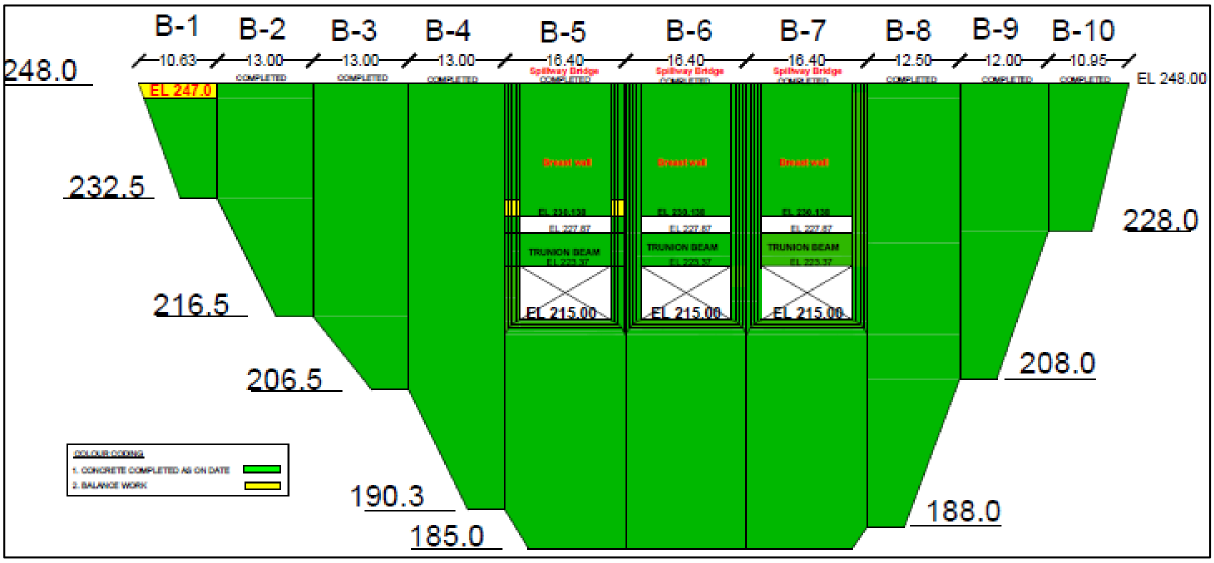



Fig 1.2 Cross-Sectional View of Pare Dam

## 1.2. Background

Rock grouting for dam foundations has been carried out in the U.S. since at least 1893 when the limestone bedrock of a dam in the New Croton Project, NY, was treated with cement grout

(Franklin and Dusseault, 1989). Opinions differ on the method of injections (Glossop, 1961, Littlejohn, 2003), although other reports (Verfel, 1989) strongly suggest that U.S. grouting procedures had made "a good start."

For the best part of the following hundred years, the intense history of dam grouting in the U.S. is, to some extent, a picture of objectives not fully achieved, innovative procedures and insightful ideas inconsistently implemented, and a number of questionable practices unthinkingly perpetuated. During the last 15 years, however, in many but not all parts of our practice, there has been a radical change in our concepts and in our approaches to such work. Partly drawing from knowledge made available in the U.S. by European specialists, for example at the seminal grouting conferences hosted in New Orleans in 1982, 1992 and 2003, and partly by the very challenging problems posed by the need to construct remedial grout curtains in our own dams, especially on karst, there has been a technological revolution in dam grouting practices in the U.S. This revolution has greatly benefited the owners of these dams, and dams themselves, and by association the grouting profession at large.

However, the proven advantages and successes of this uniquely tailored advance have not yet everywhere been recognized, and have not always been upheld and consistently defended. We therefore find that in some regions, or in certain organizations or most sadly in certain sections of certain organizations rock grouting is still being specified in the terms of 50 years ago. Equally, there are increasing numbers of projects being specified and run according to "new concepts" which, in reality, are new only to the designers and represent a retrogressive step of almost 30 years.

In the following sections, the old, the new and the retrogressive concepts of rock fissure grouting are presented to provide a platform for logically arguing against the old and the retrogressive ways of approaching work of this type. Given the relatively high volume of dam grouting especially for remedial applications being conducted today, we have now arrived at a particularly important time to have this debate.

### 1.3. Historical Concepts

There is a trove of published information to be found on this subject, including the Proceedings from the New Orleans Grouting Conference in 1982, the "Foundations for Dams" Conference (1974) and textbooks by Houlsby (1990), and Weaver (1991) in particular. Even more important are the unpublished reports, memoranda and manuals produced on a project-specific basis, or by companies or governmental organizations. These had special gravitas because their authors strongly influenced the next generation of grouting engineers while they, themselves, were elevated to the position of "consultants" on other projects in different governances. Bearing in mind the unprecedented level of activity in those years in new dam grouting, as well as the national puritanism towards "low bid" contracting, specifications were highly prescriptive and restrictive. Such prescriptions did nothing to stimulate innovation since the contractor was reduced to the status of the cheapest purveyor of labour, equipment and materials, while the goal of the owners' inspectors was to ensure that the

specifications were enforced to the letter, via "hole by hole" direction of the grouting activities.
By the way of illustration, in 1974, Polatty was invited to give an overview of U.S. Dam Grouting Practices: "In preparing this paper, I requested copies of current specifications for foundation grouting from several Corps of Engineers districts, the TVA and the Bureau of Reclamation. In comparing these current specifications with copies of specifications that I had in my file that are 30 years old, plus my observations and experience, I concluded that we in the United States have not, in general, changed any of our approaches on grouting. AND THIS IS GOOD" (emphasis added). Interestingly, he then went on to list "difficulty in having sufficient flexibility in the field to make necessary changes to ensure a good grouting job" as a problem. What a surprise!
As a consequence, several important historical paradigms became embedded in our national practice as late as the 1980's. These include:

- The drilling of vertical holes, to a target depth (as opposed to stratigraphic horizon). The only common exception (e.g., Albritton, 1982) would be the concept of inclining the curtain upstream, so as to physically distance it from the downstream drains.

- The use of rotary drilling (often just coring) since in the early days of the 20th century since only such drills could use water flush. Percussion drilling was then synonymous with the use of air flush, which many (but not all) did recognize as detrimental to fissure cleanliness and amenability to grout. (The age old debate about rotary versus percussion drilling as being more suitable for grout holes was wrongly focused: it should have been water versus air.)

- The concept of a "one row curtain," except notably under the cores of embankment dams, where even then the shallowest possible excuse was taken to revert to one row.

- The use of relatively low grout pressures, resulting from the recurrent specification to provide "constant" pressures which therefore meant the use of progressive cavity pumps ("Moynos") as opposed to higher pressure piston or ram pumps.

- The use of "thin" grouts (with excessive water: cement ratios often well in excess of 10 by weight – although typically mixes were measured by volume). Such mixes of course were easy to pump due to their low apparent viscosity, but naturally had extremely high bleed values and horrible pressure filtration resistance. These mixes were allied with a fundamental distrust/unawareness of the benefits of additives (except for calcium chloride in "taker" situations) although, latterly, the use of bentonite was entertained and on-going though somewhat misguided experimentation with super plasticizers was conducted in certain quarters.

- Curtains were grouted until a certain cement refusal was obtained (e.g., 1 bag per foot) as opposed to a measured residual permeability. This is, however, a charitable view: often the grouting was discontinued when the budget was expended and, in the aftermath when the under seepage became of alarming quantities, the cry was made that "the grouting didn't work!" The general result (Weaver and Bruce, 2007) of these deficiencies was either a) a poor travel of grout in the ground, leading to the drilling of families of higher order holes at ridiculously close centres (e.g., 1 foot at Chickamauga Dam, TN), or b) uncontrollable flow of "thin" grouts into karstic voids or similar major features.

It is somewhat of a testament to the enlightened, the lucky, and the meticulous that so many of the curtains constructed in the period from the 1920's to the early 1980's in particular appear to have actually functioned adequately given the restrictions, the misconceptions and the prescriptions. Uncharitable views would have it that such curtains may not have been needed at all, from a dam performance or safety viewpoint, and that the curtain was inserted by rote and by paradigm. On the other hand, the fact that so many of our dams have now been remediated, or are facing remediation as a result of an ineffective, incomplete and/or deteriorating grout curtain, does lead us back to the inescapable fact that the "old ways" in retrospect contained major flaws in their workings. One definition of the word "insanity" is to continue to do the same thing even when it has been repeatedly proved to fail or to be wrong. To persist with, or revert to, the "old" ways of grouting dam foundations is an example of this definition.

### 1.4. Current Principles

There had arrived in the North American scene by the mid-1990's a potent mixture of knowledge and opportunity. As arguably first articulated at a Grouting Seminar in Toronto, ON in 1989, but certainly emphasized to the cognoscenti in New Orleans in 1992, the world of dam grouting in North America had begun to change dramatically. This statement is made with all due recognition of Dr. Wally Baker who, some years before, had instigated an advance into new technical fields, but an advance which proved economically unsustainable in the face of prevalent contracting and procurement vehicles of the time.

Of particular significance was a paper by DePaoli et al. (1992) which, in a deceptively understated way, explained quite clearly the critical control and importance of pressure filtration coefficient over the effective travel of grouts into fissures, and hence their efficiency in generating low and durable residual rock mass permeability. As described in Weaver and Bruce (2007), pressure filtration can be conceived as follows:

"The injection of particulate grouts into small apertures is similar to pressing the grout against a filter material: depending on the formulation of the grout, water can be expelled from the grout in motion, leading to the development of cementitious filter cake at the borehole wall. With more time, the cake blocks off the entrance to the aperture and so render the aperture inaccessible to further injection via that avenue. This tendency of the grout to lose water during injection is quantified by the term pressure filtration coefficient (Kpf)"

"To enhance the penetrability of a grout, a low-pressure filtration coefficient that minimizes the increase in apparent viscosity (Figure 1) is required. The general relationship between the two vital parameters of cohesion and pressure filtration coefficient is shown in Figure 2. Whereas cohesion was traditionally minimized in simple cement–water grouts by using extremely high w: c ratios (Albritton 1982), such mixes have high Kpf values, which severely curtail their penetrability. However, by using much lower water contents (typically less than 1.5 weight by volume) and combinations of stabilizing and plasticizing admixtures and additives (including bentonite, silica fume, and Welan Gum), grouts of low viscosity (less than 60 seconds Marsh), low cohesion, minimal bleed, and excellent Kpf values (less than 0.02 min–1/2) can be produced.

DePaoli et al. (1992) found that even under moderate injection pressures, such balanced, stabilized grouts provided enhanced penetrability and performance via the following:

- An increased radius of travel;
- A more efficient sealing ability as a result of the improved penetrability and the lower permeability of the mix;
- A high volumetric yield, with uniformly filled voids; and
- A higher erosion resistance because of improved mechanical strength for given cement content."

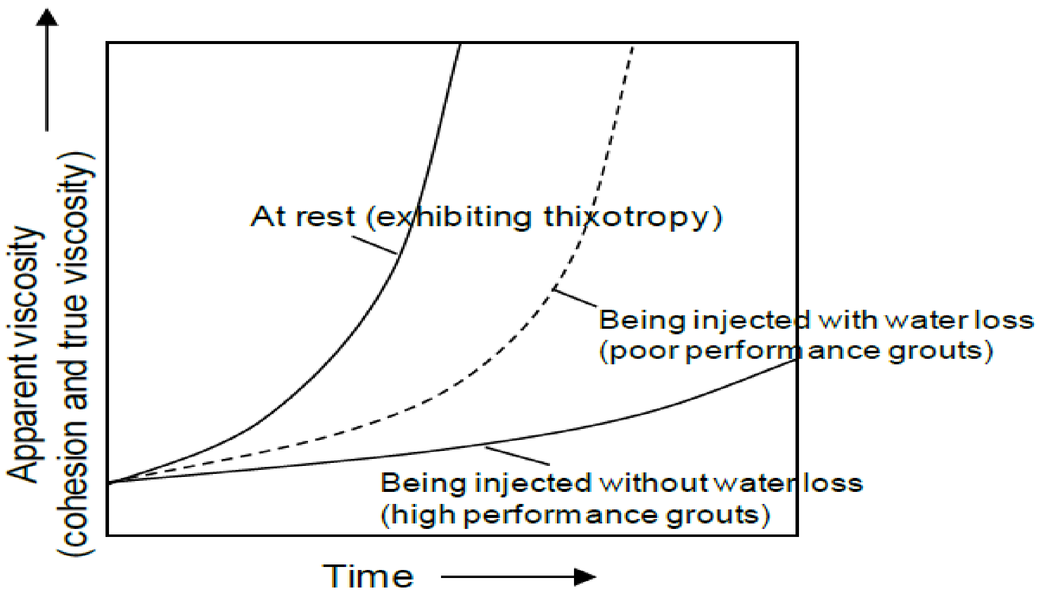



Fig.1.3 Rheological behaviour of typical Binghamian fluids
(Modified after Mongilardi and Tornaghi, 1986)

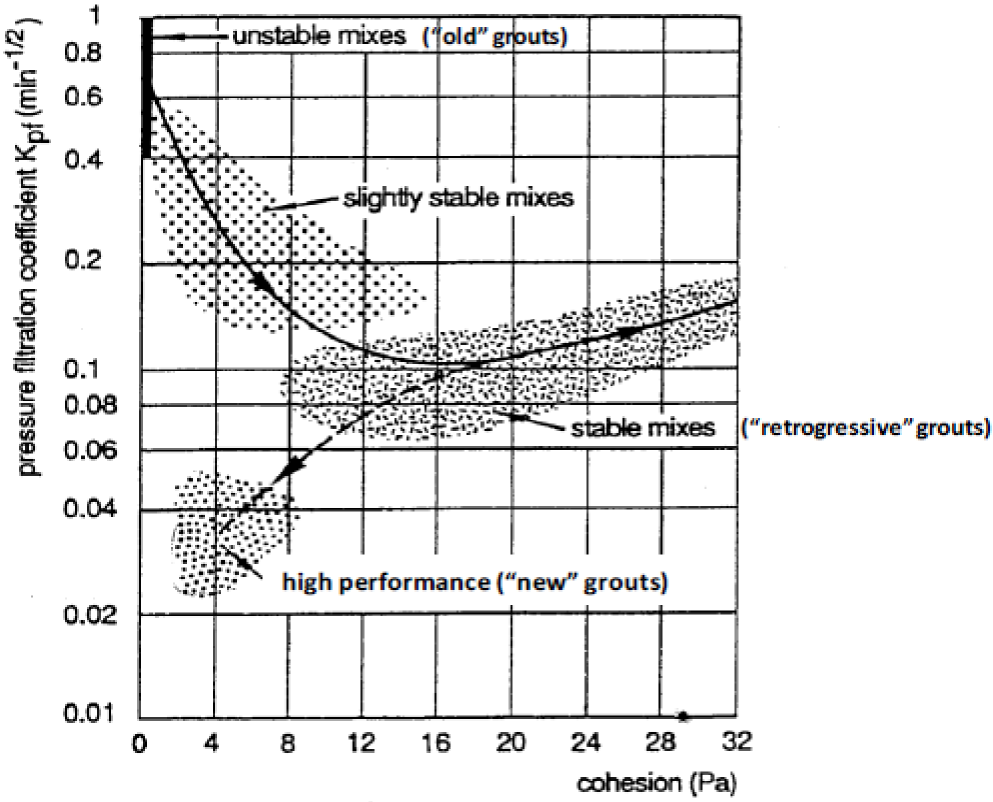



Fig.1.4. Historical path of development from unstable mixes to contemporary balanced multi-component mixes (Modified after De Paoli et al,1992)

The U.S. literature before 1992 was the significance (or even concept) of "pressure filtration" mentioned in conjunction with rock grouting Of course, it must be acknowledged that other factors will impact curtain effectiveness, but never in.

It is only fair to separate from the comparison between "old" and "new" those elements which are, by invention and technology, the exclusive privilege of the "new." Much has been written and rightly so, about the tremendously beneficial effect that the use of computer-based systems have had on the collection, processing, interpretation and display of data from the field (Dreese et al., 2003). No reputable grouting project of any significant scale or importance does now not have such a capability, feeding news back into a central "mission control" and back into the Project Executive's desk in head office, as well. The best of these systems can now integrate all the drilling and water testing data, as well as the grouting data, to compliment and compare with the historical site investigation data (and original grouting information) which may be available on any particular project. Given the power of this knowledge, curtains can be constructed to engineered standards with a degree of reliability and confidence which was unthinkable under old regimes.

Another child of the new age is the Optical or Acoustics Televiewer, an extremely acute and reliable instrument which basically provides a "flat core" of a pre-existing hole (Photograph 2). With this capability, the borehole wall conditions of drill holes formed "destructively" without the expense of core drilling can be closely scrutinized, and compared with results from permeability tests and grout injections. This is an extremely important diagnostic tool, and represents compatibility far beyond the grainy, boring images hitherto provided by down-the-hole video cameras.

Returning to a comparison of “old” and “new” concepts, the fundamental change in attitudes towards mix designs and mix properties has already been discussed: it is one absolutely vital component in the revolution. However, even today, the author finds specifications or worse, projects where the grout mix design comprises three components at best, and mixes are changed from “thin” to “thick” by changing from water: cement ratios of 3:1 to 0.8:1, or 0.6:1 in the case of “gulpers.” This is simply inexcusable and not acceptable given the state of knowledge which currently exists and is freely available on this subject.

Other areas of important distinction in contemporary grout curtain design and construction may be summarized as follows:

**Curtain geometry:**

Curtains must have, as a minimum, 2 rows of holes, which extend, wherever feasible technically, into a confining layer. They are not simply installed to a target depth below ground surface. Also, the holes in each row are inclined say 15º off vertical. The inclination of each row of holes is in the opposite direction, thereby producing a “criss cross” effect, assured to intercept all fissure sets, especially those vertically oriented. The zone between these “outer rows,” typically about 10 feet wide, is then available for additional “tightening” holes, perhaps using special or different grouting materials, and for drilling and testing Verification Borings which are installed to demonstrate the residual permeability achieved by the curtain.

**Residual Permeability:**

The purpose of a grout curtain is to stop water flowing through the rock mass. Therefore, its acceptability as an engineered structure must be verified by measuring its residual permeability — to water, not some arbitrary limiting grout take. (As described above, an inappropriate grout will have premature refusal in certain fissures, while not reducing the permeability of the ground further away.) This test is best done in cored (or Optilogged) holes, using multi pressure Lugeon Tests as first described by Houlsby (1976).

**Declaring the Target Residual Permeability:**

Residual permeability is the goal which must be declared as part of the design by the Engineer and which therefore must be satisfied by the Contractor. A grout curtain truly now is a “Quantitatively Engineered” structure (Wilson and Dreese, 2003), created by real-time control of subsurface construction processes. This “measure of success” will vary from project to project, as articulated by, for example, Houlsby (1990), but is vital to declare and essential to satisfy.

**Stage Refusal:**

Each and every stage should now be brought to a virtually total refusal. When viewing the grouting process on the computer monitor, this means an Apparent Lugeon Value of practically zero for each stage (i.e., the (stable) grout is used as a test fluid in the same way as water is). In reality, this means that the stage in question is consuming grout at less than 0.1 gpm over a period of, say, 5 minutes, at target pressure. More lax refusal criteria will result in incompletely and inefficiently grouted stages, and so higher than desirable residual permeability in the rock mass.

**Drilling methods and concepts:**

Water is the drilling and flushing medium of choice in rock masses. Whether the drilling is done by percussive methods (top hole, or water-powered down-the-hole hammer) or rotary methods (which tend now to be less competitive and have greater deviations) is technically immaterial. Also, the development of commercially viable rotary-sonic systems (Bruce and Davis, 2005) has provided a method which has entirely satisfied federal regulations (USACE, 1997) for drilling through existing embankment dams without fear of hydro or pneumatic fracture. In this regard, it is also the case that innovative contractors can devise other conforming overburden drilling systems which are equally protective of embankment fills In all drilling operations, the recording of drilling parameters (e.g., rate of penetration, flush characteristics, torque and so on) has been regularized by developing automatic recorders as opposed to relying on drillers or junior field engineers: the overall rise in the quality and usefulness of these data has been predictably spectacular.

**Specifications and Contractor Procurement Processes:**

Specifications are no longer so prescriptive ("yes: we do need the head of the contractor as well as his arms") and so all contracts are not let on the low bid basis, although to do otherwise is still not permissible for many organizations, especially in the public sector. Grouting contractors are being hired, correctly, based on their skills and experience and not just their capability of calculating a low price. There is absolutely no doubt that this "Best Value" approach has raised technical standards across the board and has, interestingly, honed the competitive instincts of all competent contractors: all this is to the inestimable benefit of the projects themselves. Further insight on specifications is provided in Bruce and Dreese (2010).

### 1.5 Necessity of Curtain Grouting in Dam Foundation

(a) To reduce the quantity of seepage through the base of dam foundation.
(b) For strengthening the rock mass by filling up the open joints and cracks.
(c) For strengthening the shattered rock mass around the excavation.
(d)To safeguard the foundation against erodibility hazard
(e) For the filling of voids and cavities present in rock mass.

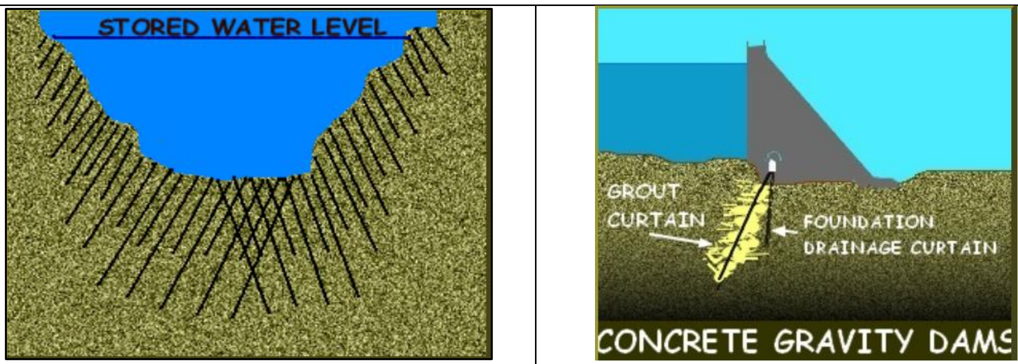



Fig.1.5. Typical layout of grout holes in a grout curtain (Courtesy: http://users.tpg.com.au)

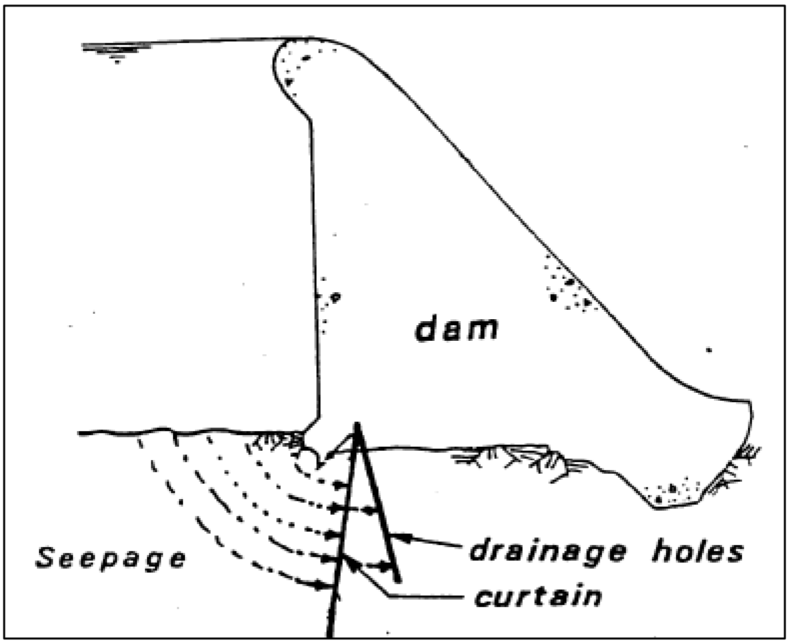



Fig.1.6. Grout curtain under concrete gravity dam (Courtesy: A guide to grouting in rock foundation by Houlsby,1990)

### 1.6. Work Approach

In this work, the whole projected is divided into three stages i.e. Drillings, packer test or Lugeon value test or the water percolation test and last one is Grouting process.
First of all drilling is carried out after that packet test is carried out at every interval of five meter drilling and Lugeon value is determined for every 5m section. The final work of my is to carried out grouting according to the value of Lugeon value obtained.

### 1.7. Objectives of the Study

The main purpose of the project is to form a zone of low permeability up to a designed depth below a specific portion of the upstream of the dam. To control the uplift pressure and piping which are potential hazard for a water retaining structure.

## 1.8. Organization of the Thesis

This thesis organised in following manner.
Chapter 1: Introduction.
Chapter 2: Literature review
Chapter 3: Methods, materials and Equipment used.
Chapter 4: Results and analysis.
Chapter 5: Summary and conclusions.

# Chapter 2
# LITERATURE REVIEW

## 2.1 Introduction

The objective of this objective is to perform the literature survey. This chapter deals with the various research associated with the concrete gravity dam and cement grouting which has been performed earlier.

**Lopez-Molina, Valencia-Quintanar, Espinosa-Guillen (2015**) stated that the design and implementation of grouting treatments in rock masses are procedures that require continuous adjustment of parameters and criteria to optimize the results. In this proposal we describe a set of tools that enhance decision-making for this type of jobs particularly in dam projects. The methodology is focused on hydrogeological zoning of the site and its constant update combining engineer's experience with artificial intelligence techniques to integrate the site knowledge; as well as the evaluation of grouting results for different scrutiny scales, with special attention on the relationship between water absorption and grout consumption.

**Singh, Dev, Vidyarth (2011)** this paper deals with quality control assurance of the grouting operations (contact/consolidation) and permeability tests carried out in head race tunnel (HRT) of Tala Hydroelectric Project in Bhutan. Efficacy of grouting was determined by conducting permeability tests before and after consolidation grouting. Contact grouting is done to fill the cavities/voids between concrete and rock mass on account of shrinkage of concrete and uneven over breaks. Consolidation grouting is done to strengthen the surrounding rock mass by filling up the open joints, fissures, cracks etc. Proper grouting of surrounding rock mass around the opening helps in monolithic behaviour of the rock mass. The quality assurance during grouting was ensured by checking the properties of all materials being used and by conducting permeability tests in pre/post grouting stage.

**Bidasaria (2004)** concludes that Curtain Grouting is an important component of Foundation treatment of Dam and other hydraulic structures. The purpose of Curtain Grouting is to form a zone of low permeability up to a designed depth on the upstream of the dam. This grout curtain along with downstream drainage system controls the uplift pressure and piping which are potential hazards. The present case study deals with providing grout curtain to Almatti Masonry Dam from Block No. 1 to 52. Besides curtain grouting this case study also deals with the treatment of weak zone (unconformity zone) of thickness up to 6 m, which was existing in the foundation from Block No. 45 to 52 below the joint of base granite rock and overlying quartzite foundation. The single line grout curtain of permeability less than 3 Lugeon has been effectively formed below the foundation, despite very poor strata having maximum pre-grout permeability of 90 Lugeon.

**Pearson &Money (2015)** Told that the Lugeon or packer test for estimating the rock permeability is not standardized and often yields anomalous result. A programme of repeated tests has been carried out in shallow drill holes in sandstones and greywacke's making use of constant head tanks with continuous monitoring of flow rate and pressure measurement in the test section. These improved techniques make it possible to distinguish during tests between test systems faults, such as packer leakage, and non-equilibrium effects due to the hydraulic properties of the rock mass. It is shown that the results obtained depend on the duration of test and on the sequence in which different test pressures are applied and it is suggested that these effects are due to the presence of small but significant storage capacity in the rock mass. The relationship between equilibrium flows and pressures appears to be non-linear in both rock types.

**Se-Yeong Hamm, MoonSu Kim et.al. (2007)** showed that Hydraulic conductivity is closely related to fracture characteristics like fracture aperture and frequency, fracture length, fracture orientation and angle, fracture interconnectivity, filling materials, and fracture plane features. In this study, water injection tests were conducted at six boreholes of different depths drilled in fractured granite in the Mt. Geumjeong area, Korea. Hydraulic conductivity was calculated using the USBR (United States Bureau of Reclamation) water injection test method. Hydraulic conductivity was related to fracture frequency, squared fracture aperture, and the squared aperture of major fracture orientation obtained from acoustic televiewer and core log data. Fracture aperture had a stronger relationship to hydraulic conductivity than fracture frequency did. In addition, the correlation between transmissivity and cubed fracture aperture was higher than the correlation between hydraulic conductivity and the squared fracture aperture. The squared aperture with respect to major fracture Orientation had a weaker correlation with hydraulic conductivity than the squared aperture using all fracture orientations. This suggests greater complexity for groundwater flow at the borehole scale compared to the regional scale. Meanwhile, the correlation coefficient between hydraulic conductivity and fracture frequency obtained from acoustic televiewer data was higher than that from core log data.

**Quinn,Cherry &Parker (2015)** according to them A combination of high resolution hydraulic tests using straddle packers and transmissivity (T) profiling using the FLUT flexible liner method (liner profiling) in densely fractured rock boreholes is shown to be efficient for the determination of the vertical distribution of T along the entire hole. The liner T profiling method takes a few hours or less to scan the entire borehole length resulting in a T profile. Under favourable conditions this method has good reliability for identifying the highest T zones identified by distinct decreases in liner velocity when these zones are covered by the descending liner. In contrast, for one short test interval (e.g., 1–2 m) the multiple-test, straddle-packer method takes a few hours to measure T with good precision and accuracy using a combination of steady-state and transient tests (e.g., constant head step tests, slug tests, and constant rate pumping tests). Because of the time consuming aspect of this multiple-test method, it is most efficient in each borehole to conduct straddle packer testing only in priority zones selected after assessment of other borehole data collected prior to packer testing. The T profile from the liner method is instrumental in selecting high permeable zones for application of the

multiple-test method using straddle packers, which in turn, refines the T estimation from the liner profile.
Results from three boreholes in densely fractured sandstone demonstrate this approach showing the synergistic use of the methods with emphasis on information important for determining hydraulic apertures.

**Chen and Zhang (1989)** Fracture grouting has been widely used to seal leaks through the poorly compacted cohesive fill of embankment dams since the 1960s in China. Its effectiveness has been explained by the formation of a vertical mud wall parallel to the dam axis along the minor principal stress plane, and intrusion of grout into seepage channels across the dam axis (Chen 1982). Review of water loss from boreholes in an embankment dam (Chen 1987) has offered some direct evidence of the mechanism of fracture grouting, but this mechanism would not be credible without verification by large-scale tests. The results of a field test presented in this paper provided an opportunity for Verification and the test results suggest ways to improve the technique of fracture grouting.

**Joshi and Tedd (2010)** they revealed that Slurry trench cut-off walls, constructed using self-hardening slag-cement-betonies (Slag-CB), are the most common form of in-ground vertical contaminant barrier in the U.K., Europe, and Japan, and are increasingly being used in the United States. This paper presents a case study of the hydraulic conductivity evaluation of an 11-year-old (Slag-CB) wall material at a sulphate-contaminated site, using different in situ techniques and laboratory tests. The laboratory results suggest that the hydraulic conductivity of the samples, which vary in age from 4 weeks to 11 years, decreases with time for the first 3 years but then remains constant. The results indicate that the long-term performance of these containment walls is influenced by various parameters such as aging, the type/duration of contaminant exposure, mixing of surrounding soil during construction, and wall depth. Piezocone tests, packer tests, and self-boring permeameter tests were carried out in the field to determine the suitability of different in situ techniques and compare with the laboratory results. The hydraulic conductivity is affected by the type of in situ technique used and the geometric scale of the test section.

# Chapter 3

# METHODS AND EQUIPMENTS

## 3.1. Introduction

This chapter discuss about methods and equipment's used during whole project analysis. This study is the combination of three different methods i.e. drilling operation, Packer test and finally curtain grouting operation.

## 3.2. Drilling Operation

Drilling through dams and their foundations is carried out with the help of water, compressed air and various drilling fluid. The risk will vary with the selected methods and the site conditions. Every drilling operation must be well thought out and must have benefits of successful completion that confidently outweigh the risk of potential negative impacts.

Excessive pressures from water, air, drilling fluid, or grout can fracture embankment and foundation materials. Hydraulic fracturing problems have occurred while drilling in embankments as evidenced by reports of loss of fluid circulation, blowouts into nearby borings, seepage of drilling fluids on the face of the embankment, and other similar geometry, piezometric surface, abutment configuration, foundation rock geometry, embedded structures, compaction stress, and settlement history all are significant and can influence situations. Hydraulic fracture can occur in both cohesive materials and cohesion less materials, and bedrock. It has been found that in soils, hydraulic fracturing can occur when the borehole pressure exceeds the lowest total confining stress (minimum principal stress, σ3) plus some additional strength (Sherard, 1986). The additional strength can be approximated by the untrained shear strength of the soil. The minor principal confining stress (σ3) in a normally consolidated soil with a level ground condition is typically the horizontal stress, which can be reasonably estimated. However, the minor principal confining stress in and under an embankment is difficult to determine and can vary significantly from idealized geostatic conditions. Effects from the side slope in-situ stress conditions. Typical drilling methods that use circulation fluids can quickly create induced fluid pressures that exceed the minimum confining stress. This often occurs when the return path for the fluid clogs or blocks off and the induced fluid pressures quickly increase. The use of non-pressurized stabilizing fluids is preferable, yet in some subsurface conditions, hydraulic fracture can occur under gravity pressure. Low stress zones may exist within and under embankments. It is possible for the confining stress in these locations to be much less than the gravity pressure exerted by a drilling fluid or grout.

Locations and conditions where hydraulic fracturing by drilling media is more likely to occur and have the higher potential of damaging the structure include the following:

- Near and over steep abutments that create low confining or tensile stress conditions.
- Adjacent to rock overhangs on abutments.
- Adjacent to buried structures or abrupt foundation geometry change that creates a differential settlement condition and a zone of lower soil stress transfer.
- Adjacent to conduits where narrow zones of soil backfill were placed between the structure and rock face.
- Dam cores that can experience more settlement than the adjacent shells.
- Dams in very narrow valleys. Arching keeps full confining stresses from developing.
- Near abutments where abrupt changes in geometry occur
- In areas where the embankment is subject to differential settlement due to large differences in thickness of adjacent compressible foundation or embankment soils.

### 3.3. Standard Method of Core Description

**Elevation:** The depth mark at every metre is shown, while every 3 metre is to be written in the description column. A horizontal line should be drawn and its R. L. written, corresponding to every significant entry in the subsequent columns, for example, at the, start and at the end of a litho unit or a major shear zone.

**Lithology:** After the cores have been examined carefully, the description in regard to their lithology should be entered in this column using accepted symbols. In cases where many subdivisions of a standard rock type have to be used suitable derivatives from the accepted symbol may be evolved and used but explained. A horizontal line should be drawn in the description and log column at every change of the lithology of the cores and thus only one symbol would be used in the log column between two horizontal lines. Corresponding to this entry in the log column, the name of the rock type should be entered in block letters against it in the description column of the lithology. Below this line in brackets and in small letters should be entered the depths of the hole between which the particular rock type is met, for example (10 to 15 m). If necessary a brief geological description of the rock type, such as colour, grain size or any other feature ( for example, greyish white, fine grained and calcareous) may be given in the column observation and interpretation. Care should be taken that all the entries are accommodated within the vertical space available against the respective log column.

**Systematic Rock Description:** It is considered that the qualifications are more important in core descriptions than the actual rock name and, for this reason; the name should be placed last. Such a system is appropriate to an engineering description where classification by mechanical properties is more significant than classification by mineralogy and texture. The description for each litho unit met with should be written under the columns 'Special Observations and Interpretations of the Performa'. In this project rock is considered highly weathered and comes under **Grade IV**.

The following system of weathering classification should be followed:

Table 3.1 Weathering classification of Rock (Source: IS: 4464-1985)

| Items | Description | Grade |
|---|---|---|
| Fresh | No visible sign of rock material weathering, it shows slight discoloration on major discontinuity surfaces. | I |
| Slightly weathered | Discoloration indicates weathering of rock material weathered and discontinuity surfaces. All the rock material may be discoloured by weathering and may be Somewhat weaker externally that in its fresh condition. | II |
| Moderately weathered | Less than half of the rock material is decomposed weathered and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as core stones. | III |
| **Highly weathered** | **More than half of the rock material is decomposed weathered and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as core stones.** | **IV** |
| Completely weathered | All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact. | V |
| Residual soil, | All rock material is converted to soil. The mass soil structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported | VI |

**Size of core pieces**: As the main interest on engineering projects is the evaluation of the physical condition of the rock type, this column is of great interest. A 100 per cent core recovery may on one hand be consisting of big rods of cores while on another may be composed of small broken pieces of less than 10 mm each. The entry in this column should be in a graphical manner to give a visual idea of the condition of the core. If the core pieces are bigger than 150 mm each, it means that the rock is massive to blocky and as such 150 mm and above core pieces have been included in one column. On the other hand core pieces of less than 10 mm size are included in one column. As in general the cores are broken in small pieces the lower ranges have been given more representation and the column is divided into IO mm, 10 to 25mm, 25 to 75 mm, 75 to 150 mm and above 150 mm groups. A horizontal line should be drawn at each change of the size of core entered and the zone from the zero size line to the size line recorded should be shaded by inclined pencil lines.

**Structural condition:** In this column the structural condition of the cores should be entered, for example, heavily sheared and crushed, moderately sheared. Blocky, etc. Suitable symbols
Should be entered in the log column with horizontal lines separating each entry and their description or name with the depths entered in the description column, in the same manner in which the lithology column is filled. Special features, like major joint planes, fractures, faults, etc., may be graphically plotted with their actual amount of dips in the log column here.

**Discontinuity spacing:** Considerations of discontinuity spacing should lead to an appraisal of rock mass structure; this may be assisted by field observation. In the case of sedimentary rock, where bedding may be the dominant discontinuity, it is possible to recognize and define a bedding spacing from borehole cores. This system has the advantage that the scale is related to that used in the mechanical analysis of soils. The following classification should be followed:

Table No 3.2 Classification of Bedding of Rocks (Source: IS: 4464-1985)

| Classification | Bedding plane spacing | Soil grading |
|---|---|---|
| Very thickly bedded | 2 m | Boulders |
| Thickly bedded | 0.6 to 2 m | Boulders |
| Medium bedded | 0.2 to 0.6 m | Boulders |
| Thinly bedded | 60mm to 0.2 m | Cobbles |
| Very thinly bedded | 20 to 60 mm | Course gravel |
| laminated | 6 to 20 mm | Medium gravel |
| **Thinly laminated** | **<6 mm** | **Sand and fine gravel** |

In this project rock is thinly laminated in nature and comes under sand and fine gravel stone.

For igneous and metamorphic rocks the separation of the integral rock discontinuities (such as foliation, flow-banding, etc.) may be described by adaptation of the bedding plane spacing scale given above, for example, medium foliated genesis. It is suggested that 'close' and 'very close' are applied to that part of the scale where a sedimentary rock would be described as 'laminated' or 'thinly laminated'. Terms such as blocky, intact, uniform, etc., may be used providing these terms are defined in the preamble to the bore-hole records and are based on in-situ inspection of the rock mass or by deduction from several boreholes.

**Core Recovery:** In this column the core recovery should be plotted in graphical form. A horizontal line should be drawn at the interval of each run of drilling and the line representing the percentage of core recovery entered should be shaded by inclined pencil lines. Here, the percentage of core recovery for concrete is **100 per cent** and for rock mass is considered **zero.**

**Rock quality designation (RQD)**: This classification is based on a modified core recovery procedure, which in turn, is based indirectly on the number of fractures and the amount of softening or alteration in the rock mass as observed in the rock cores from a drill hole.

**Fracture frequency**: RQD, given above, however, does not take into account the joint opening and condition; a further disadvantage being that with fracture spacing greater than 100 mm the quality is excellent irrespective of the actual spacing. This difficulty is overcome by using fracture frequency. The rock quality relation between RQD and Fracture Frequency is given as under:

Table No.3.3 Description of rock quality

| Description of rock quality | RQD% | Fracture frequency per m |
|---|---|---|
| **Very poor** | **0-25** | **Over 15** |
| Poor | 25-50 | 15-8 |
| Fair | 50-75 | 8-5 |
| Good | 75-90 | 5-1 |
| Excellent | 90-100 | <1 |

In this project rock quality is **very poor** and RQD is varies **from 0 to 10 per cent**.
**Size of the hole:** Size of drilling hole is considered 76 mm here. But size of hole may be different as per requirement of site condition. It should be recorded in the drilling report.

**Casing:** generally casing is embedded through the concrete section so that drilling cost reduced. Whenever the drilling is required for water percolation test and for grouting the drilling process is proceed after the concrete section through the casing.

Fig 3.1 Casing embedded in the concrete section and core recovery of concrete

**Depth of water level:** The depth of the water level observed during drilling should be recorded in the drilling report. In this report water level is consider at ground level.
**Drill water loss:** The loss of drilling water should be recorded in the drilling report in terms of complete, partial or no water loss, 100 per cent, 50 per cent, and 0 per cent.
**Permeability:** the permeability of the material of any section should be entered in terms of Lugeon value or cm/sec.
**Penetration Rate:** the rate of penetration is primarily dependent on the type of rock. It also depends on many factors such as intensity of joints, nature of infilling material, degree of weathering of rocks. It is noted in the drilling reports in terms of cm/minute. In our case it varies from **2 to 5 cm/minute.**

**Strength:** The strength characteristics of the rock mass are noted based on uniaxial compressive tests.

Table 3.4 Uniaxial compressive tests (Source: IS: 4464-1985)

| Strength ,$N/mm^2$ | Terms |
|---|---|
| **Up to 1.25** | **Very weak** |
| 1.25 to 5 | Weak |
| 5 to 12.5 | Moderately weak |
| 12.5 to 50 | Moderately strong |
| 50 to 100 | Strong |
| 100 to 200 | Very strong |
| >200 | Extremely strong |

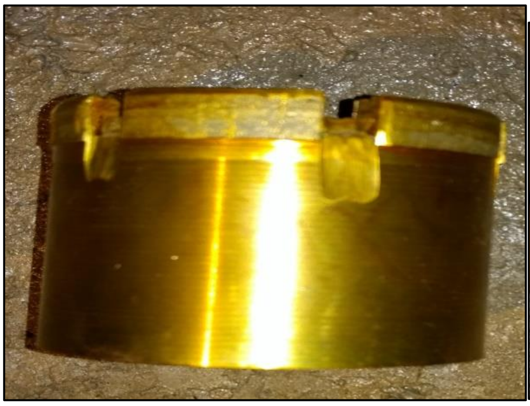



Fig 3.2 NX Diamond core Drilling bit.

Fig 3.3 NX,AW Drilling rod.

Fig 3.4 Drilling barrel

Fig 3.5 Drilling barrel and diamond drilling core bit.

### 3.4. Water percolation test or in-situ permeability test or Lugeon value test.

### 3.4.1. Introduction

The lugeon test (Packer test) is an in-situ permeability test which is widwly used to estimate the average hydraulic conductivity of rock formatipon. The test is named after **Maurice Lugeon(1933)** a Swiss geologist who first formulated the test. The lugeon test is a constant rate injection test carried out in a portion of a borehole isolated by inflated packers. Water is injected portion of borehole by using a slotted pipe. Water is injected at specific pressure and the resulting pressure is recorded when the flow has reached in the quasi-steady state condition. A water meter is also connected to record the total injected quantity of water.

### 3.4.2.Terminology

**(a) single packer method.**

In this method one packer is used in the drill hole. In this case the test section is between the bottom of the bore hole and the packer as shown in the fig. 3.3is considered.

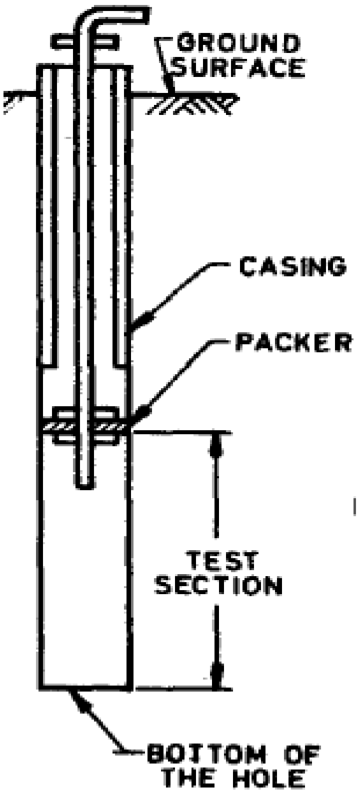



Fig 3.6 Permeability test in single packer method. (Source :IS 5529 Part2:2006**)**

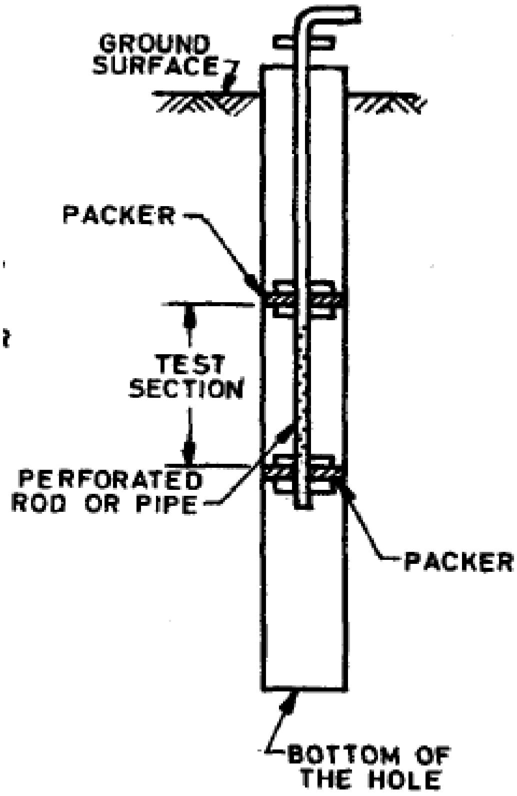



Fig 3.7 Permeability test in double packer method.(Source :IS 5529 Part2:2006)

**(b) Duuble packer method.**

In this method two packers are used in the drill hole. In this case the test section is considered between the two packers as shown in the fig.3.4.

### 3.4.3.Equipments

**(a) Drilling Equipment:**
A drilling bit and drilling rod is require to drill the hole before performing the WPT test.

**(b) Water meter**
A water meter having reading capability up to 0.5 litre in accuracy is must. The water meter should be periodically checked and calibrated.

**(c) Pressure Gauge**
A pressure gauge of range 4 kg/cm$^2$ to 20 kg/cm$^2$ should be chosen depending on the maximum pressure desired for the testing.

**(d) Pump**
A centrifugal pump of minimum capacity 500 litre/minute is used which is capable of producing pressure up to 30 kg/cm$^2$ . water pipes,connections and swivels,drill rods,perforated rods and other fittings are also must.

**(e) Packers**
The rock formation where drill holes retain their proper size learther cup packers are used. Mechanical packers are commonly used in moderately hard formations where the holes are drilled 20% over size. Pneumatic packers are used for all types of formations and are generally prefered for soft rock formations , in this case also holes are drilled 20 % over size.

**(f)Equipments for measuring water level in the drill hole and stop watch having measuring capacity up to 1hour and should have least count of 1 second.**

| | | |
|---|---|---|
|  |  |  |
| Fig 3.8 Piezometer device | Fig 3.9 Single packer | Fig.3.10 Water meter and pressure meter |

| Fig 3.11 Single and Double packer | Fig 3.12 Hydraulic Pump |
|---|---|

**3.4.4. Quality of water to be used for the test.**

The tests described are of the pumping-in type, that is, they are based on measuring the amount of water accepted by the ground through the open bottom of a pipe or through an uncased section of the hole. Unless clear water is used, these tests are invalid and can be Grossly misleading. The presence of even small amounts of silt or clay in the water used in the test will result in clogging of the test section and will give permeability results that are too low. Efforts should be made to assure supply of clear water by means of a settling tank or a
Filter. It is also desirable, where the climatic conditions demand, to raise the temperature of added water higher than ground temperature so as to preclude the creation of air bubbles in the test section that can greatly reduce the acceptance of water.

**3.4.5. General Procedure.**

The water percolation tests, covered by this standard, should be conducted in uncased and ungrouted sections of the drill holes. The procedure adopted consists of pumping water into the 'test section' and is therefore called 'pumping-in type'. Packers are employed for conducting these tests and depending upon the use of one packer or two packers the method is designated as single or double packer method respectively. Examination of the drill cores and the results of water tests obtained during drilling will usually indicate whether a double packer test in any isolated section or sections of the drill hole is required. In certain formations, it may not be possible to use the packer, or there is a danger of the packer being stuck in the hole. In such cases, a better method would be to grout the earlier stage, extend the bore hole -and carry out the test. The tests are based on-measuring the amount of water accepted by the 'test section' (of the hole) confined by a packer/packers while water is pumped into it. The layout of equipment for the test is as shown in Fig. 3.13
The single packer method as shown in Fig.3.6 is used where the full length of the hole cannot stand uncased/ungrouted in soft rocks, such as sand rock (soft sand stone), clay shale or due to highly fractured and sheared nature of the rocks or where it is considered necessary to have permeability values along with drilling (for example where multiple

aquifers are present). Double packer method may be adopted where the rocks are sound and the full length of the hole can stand without casing/grouting, as shown in Fig. 3.7. The specific advantage of double packer method is that critical rock zones can be tested by confining them with packers. The disadvantage of the double packer method is that leakage through the lower packer can go unnoticed and lead to over estimation of-water loss. Wherever time permits, single packer method would be preferable.

### 3.4.6. Specification of single packer method

The method used for performing the water percolation tests in a section of the drill hole using a single packer is as shown in Fig.3.6. In this method, the hole should be drilled to a particular depth desirable for the test. The core barrel should then be removed and the hole cleaned with water until clear water returns. The packer should be fixed at the desired level above the bottom of the hole and the test performed in accordance with the procedure laid down. After performing the test, the entire assembly should be removed. The drilling should then be proceeded with till the next test section has been drilled for performing the test. In this manner the entire depth should be tested alongside with the advancement of drilling.

### 3.4.7. Specification of double packer method

The method used for performing the water percolation tests in a section of the drill hole using a double packer is as shown in Fig. 3.7. In this method the hole should be drilled to the final depth desired and cleaned with water until clear water returns. Two packers connected to the ends of a perforated drill rod of a length equivalent to the test section should be fixed in the drill hole. The bottom of the perforated rod should be plugged before the double packer tests are proceeded with. The test may be done from bottom upwards or from top downwards. However, it is convenient and economical to start the tests -from the bottom of the hole and then work upwards.

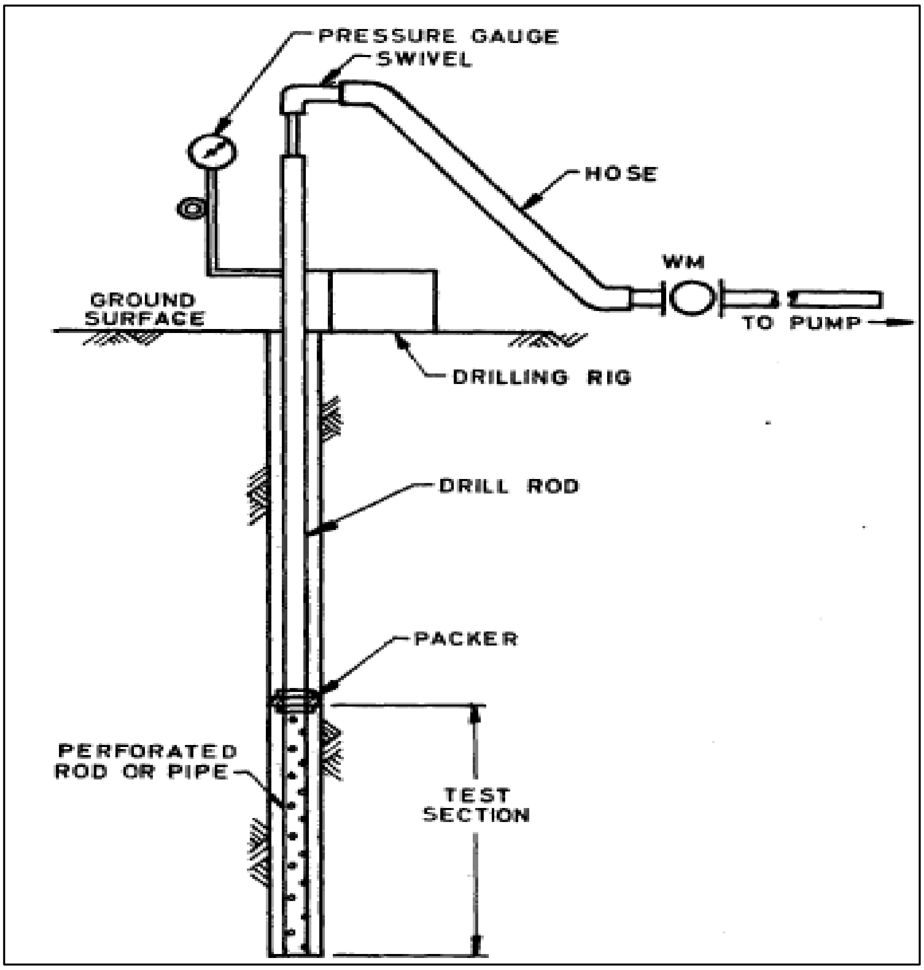



Fig 3.13. Sketch showing layout of equipment for permeability test in a drill hole (Single Packer Method), (Source: IS 5529:2006)

To verify the presence of ground water table, the water level in the hole should be depressed either by evacuation with compressed air or bailing out with stand shell. After this operation is completed, if three consecutive readings of the water level taken at 10 min to 15 min intervals are constant, then this water level may be taken as the ground water level. The time

Interval may have to be increased to 30 min in less permeable formations. This measurement

Is done for determining the hydrostatic pressure in the test zone and this value is used for calculating the permeability of the horizon. If these measurements indicate that there is no water table or piezometric head, this fact should be mentioned in the report.

The tests are recommended to be performed in1.5 m to 3 m test sections so that the entire hole is covered, depending upon the geological conditions; as for example in sections passing through a shear zone or a highly jointed zone a lesser length of section should be used. The test length should not, however, be less than 5 times the diameter of the borehole.

Under piezometric conditions, the piezometric head in separate horizons should be ascertained by measurement of water level after installation of packer in the hole.

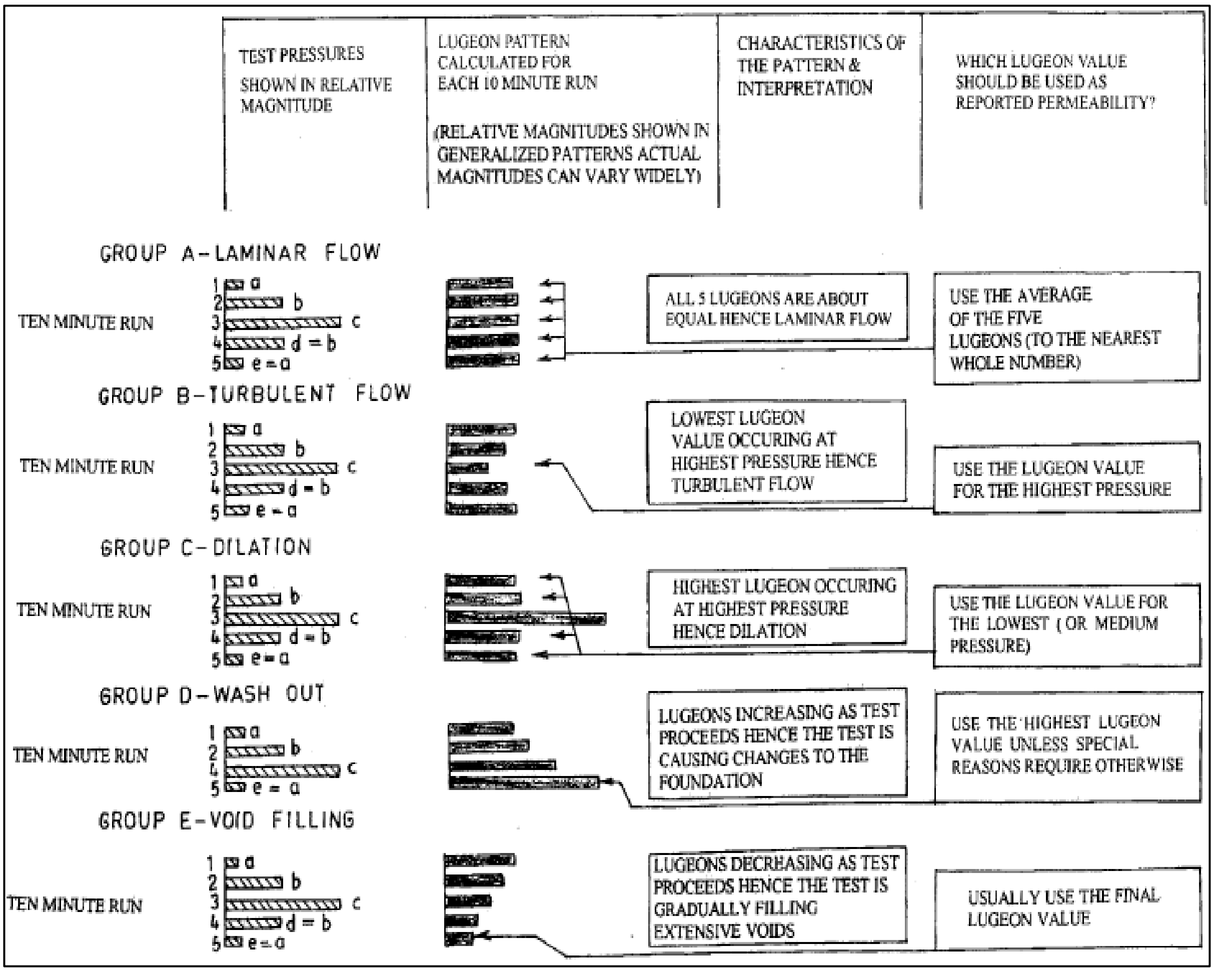
### 3.4.8 Cycle test.

In special circumstances cyclic tests are performed to evaluate the wash ability and grout ability of joints of rock or their extent. Cyclic tests for assessment of permeability are useful in computing Lugeon values. These tests are started at low pressures, the test pressures being built up to the maximum applicable pressure by increments and decreased in the same order
until the original pressure is reached. Generally, for the performance of the cyclic tests four ranges of pressure should be chosen which may-be fixed at 25 per cent, 50 per cent, 75 per cent and 100 per cent of the pressures selected for the test section based either on suitable rock cover or maximum equivalent of reservoir Head, whichever is applicable.

In addition to performing the permeability tests at regular intervals of the strata, it ‘is critical bedrock zones by confining them along with packers. Thickness of the permeable stratum and geological conditions varied locally to accommodate the packer(s) properly. Short test sections of 1.5 m would be preferred in thin bedded and heterogeneous strata.

When the intake of water in the test section is more than that which the pump can deliver, it is advisable to reduce Length of the section. Under normal circumstances test sections longer than 3 m are not recommended.

Table 3.5 Lugeon patterns for various occurrences during testing and their interpretation and percentage occurrences (Source: IS 5529: Part-2, 2006)



| | TEST PRESSURES SHOWN IN RELATIVE MAGNITUDE | LUGEON PATTERN CALCULATED FOR EACH 10 MINUTE RUN (RELATIVE MAGNITUDES SHOWN IN GENERALIZED PATTERNS ACTUAL MAGNITUDES CAN VARY WIDELY) | CHARACTERISTICS OF THE PATTERN & INTERPRETATION | WHICH LUGEON VALUE SHOULD BE USED AS REPORTED PERMEABILITY? |
|---|---|---|---|---|
| GROUP A-LAMINAR FLOW | | | | |
| TEN MINUTE RUN | 1 a; 2 b; 3 c; 4 d = b; 5 e = a | | ALL 5 LUGEONS ARE ABOUT EQUAL HENCE LAMINAR FLOW | USE THE AVERAGE OF THE FIVE LUGEONS (TO THE NEAREST WHOLE NUMBER) |
| GROUP B-TURBULENT FLOW | | | | |
| TEN MINUTE RUN | 1 a; 2 b; 3 c; 4 d = b; 5 e = a | | LOWEST LUGEON VALUE OCCURING AT HIGHEST PRESSURE HENCE TURBULENT FLOW | USE THE LUGEON VALUE FOR THE HIGHEST PRESSURE |
| GROUP C-DILATION | | | | |
| TEN MINUTE RUN | 1 a; 2 b; 3 c; 4 d = b; 5 e = a | | HIGHEST LUGEON OCCURING AT HIGHEST PRESSURE HENCE DILATION | USE THE LUGEON VALUE FOR THE LOWEST (OR MEDIUM PRESSURE) |
| GROUP D-WASH OUT | | | | |
| TEN MINUTE RUN | 1 a; 2 b; 4 c; 5 e = a | | LUGEONS INCREASING AS TEST PROCEEDS HENCE THE TEST IS CAUSING CHANGES TO THE FOUNDATION | USE THE HIGHEST LUGEON VALUE UNLESS SPECIAL REASONS REQUIRE OTHERWISE |
| GROUP E-VOID FILLING | | | | |
| TEN MINUTE RUN | 1 a; 2 b; 3 c; 4 d = b; 5 e = a | | LUGEONS DECREASING AS TEST PROCEEDS HENCE THE TEST IS GRADUALLY FILLING EXTENSIVE VOIDS | USUALLY USE THE FINAL LUGEON VALUE |

Table 3.6 interpretation of Lugeon value test (By Houlsby,1976)

| Behaviour | Lugeon Pattern | Flow vs. Pressure Pattern | Representative Lugeon Value |
|---|---|---|---|
| Laminar Flow |  |  | Average of Lugeon values for all steps |
| Turbulent Flow |  |  | Lugeon value corresponding to the highest water pressure (3rd step) |
| Dilation |  |  | Lowest Lugeon value recorded, corresponding either to low or medium water pressures (1st,2nd, 4th, 5th step) |
| Wash-out |  |  | Highest Lugeon value recorded (5th step) |
| Void Filling |  |  | Final Lugeon value (5th step) |

s

Table 3.7 Rock condition based on Lugeon values. ( source: IS 5529: Part-2, 2006)

| Lugeon Range | Classification | Hydraulic Conductivity Range (cm/sec) | Condition of Rock Mass Discontinuities | Reporting Precision (Lugeon) |
|---|---|---|---|---|
| **<1** | **Very low** | **$<1\times10^{-5}$** | **Very tight** | **<1** |
| **1-5** | **Low** | **$1\times10^{-5}$ -$6\times10^{-5}$** | **Tight** | **±0** |
| 5-15 | Moderate | $6\times10^{-5}$ -$2\times10^{-4}$ | Few partly open | ±1 |
| 50-50 | Medium | $2\times10^{-4}$-$6\times10^{-4}$ | Some open | ±5 |
| 50-100 | High | $6\times10^{-4}$-$1\times10^{-3}$ | Many open | ±10 |
| >100 | Very high | $>1\times10^{-3}$ | Open closely spaced or voids | >100 |

In our case generally Lugeon values come **less than 1 or come 1 to 5**. It means condition of rock mass discontinuities come under **very tight and tight** category.

## 3.5.Grouting

### 3.5.1 Introduction

Pressure grouting of rock foundations is normally carried out to fill discontinuities,cavities or voids in rock mass by suitable materials. The grouting programme should aim at satisfying the design requirements economically and in conformity with the rest of the construction schedule. The design requirements for adopting a srouting programme are as underThe parameters will depend on the type of structure.

### 3.5.2 Type of grouting

**(i) Curtain grouting**

(a) To safe guard the foundation against erodibihty hazard

(b) To reduce quantity of seepage

**(ii) Consolidation grouting**

(c) To reduce the deformability of jointed or shattered rock.

Even tboulb the overall objective is to reduce the permeability of the rock foundation,the relative emphasis between control of the rate of seepage and control of uplift depends .

On the value of the water stored and the nature of foundation strata. In cavernous and hiahly

jointed rocks the reduction of the rate of seepage may be an important safety consideration.

On the other hand in massive relatively unweathered rocks, the quantity of seepage may not be of consequence, as long as the desired reduction in uplift rressure is achieved. In such cases, uplift contro may be achieved primarily by drainage while the aim of grouting would be to ensure that local concentrations of seepage do not occur which are liable to impair the

efficiency of the drainage system.

Before arriving, at design requirements for any job the prrmary objectives should be defined, for example, reduction of rock deformability, etc. The depth, spacing and pattern of grout holes, the choice of method of grouting, materials injected and consumption limits as well as controls on pressure depend on the objectives. These controls and criteria would have to be established by trials and it is desirable to establish the programme of trials at the initial stales of the work.

The limiting lugeon values' given in Table 3.8 are recommended for deciding the necessity of grouting, Lugeon values in excess of those given in the table would indicate that grouting is desirable. In our case Lugeon valuegenerally lies less than 1 or 1to 5. It means it comes under **Group D, Washout and hydraulic fracturing.**

Table 3.8 Suggested limiting Lugeon values from erodibility considerations (source: IS 6066:1994)

| Items | Rock below cutoff trench | Rock below masonary dam |
|---|---|---|
| Group A, Laminar flow | 5-10 | 5-7 |
| Group B, Turbulent flow | 3-5 | 3-5 |
| Group C, Dilation | 1-3 | 1-3 |
| **Group D, washout and hydraulic fracturing** | **1-3** | **1-3** |
| Group E, void fill | 3-5 | 3-5 |

For dams exceeding 30 m height. Curtain grouting should be carried out when the water absorption exceeds one lugeon and For dams under 30 m height, curtain grouting should be carried out where the water absorption exceeds 3 lugeon.

### 3.5.3. Coordination with other construction activities

Grouting operations are generally nterdependent with other construction operations;for example, excavation and blastina in the vicinity of the area that is being grouted may cause leakage of grout and render the grouting operation ineffective or it may be necessary to complete the aroutina operations to enable a start to be made of other operations, such as concreting or masonry work. Sometimes it may be necessary to carry out grouting before removal of the overburden to obtain the necessary load of surcharge over the zone required to

be grouted. In other cases removal of the overburden may benecessary to facilitate sealing of the cracks prior to grouting. Draining boles should always be drilled only after grouting is completed within the expected distance of grout travel. Generally, It is preferable to complete blasting before taking up grouting operations. ]f blasting after grouting is unavoidable, thorough testing and regrouting is essential after blasting. A drainage and grouting gallery is a commonly used device to facilitate grouting after placing the masonry or concrete in the foundation and ensuring that the necessary cover of concrete is obtained. to enable the desired grouting pressures to be developed. Sometimes holes are dri lied in the foundation and GI pipes left in place, through the masonry or concrete. and the foundation grouted through these pipes later. It is difficult to make general stipulations regarding the coordination of grouting with other construction activities, but it would be evident from the above that careful planning of all associated construction activities, such as excavation, concreting, fill placement, drilling of drainage holes and their coordination with grouting is essential for successful execution of' the grouting programme.

### 3.5.4.Relation of geology to groutin, the importance of foundation exploration and initial experimentation.

Reliable geological interpretation of the type, distribution, approximate size and direction of discontinurties, voids, cavities, etc, in the foundation rock is necessary prior to grouting. The sub-surface conditions should be investigated by core drilling a number of holes in the foundation area. Percolation tests should be conducted in the holes within the open area of the foundat ion charted for use in planning the grout treatment. When investigation holes have served their purpose, they should be completely filled with grout. The grouting programme should be conducted in such a manner that the initial experimentation generally covers all the typical geological situations.

The depth, spacing and orientation of grout holes should be related to the geological features; for example, inclined holes should be preferred when the rock permeability is primarily due to closely spaced vertical/sub-vertical system of joints. It is sometimes necessary to evolve a pattern of holes consisting of different sets of holes appropriate to each type of discontinuity, such as bedding planes, system of joints and lava contacts.

### 3.5.5.Methods of grouting

Rock grouting consists essentially of drilling a series of grout holes in rock and injecting grout under pressure, which eventually sets in the openings and voids in the rock. The drilling
and grouting operations can be carried out either to the full depth in one operation or in successive depths either by stage grouting or by packer grouting.Grouting in the valley should proceed from river bed towards abutments.

#### (i)Full depth grouting

In the full depth method each hole is drilled to the full desired depth, washed, pressure tested
and grouted in one operation. This method is usually limited to short holes, 5m or less in depth, or boles up to 10m that have only small cracks and joints with no risk of surface leakage.In deep bore holes high grouting pressures have to be used to achieve proper penetration of the grout at an economic spacing of holes.

As full depth grouting involves the risk of disturbance in the upper elevations, it is not generally considered suitable for arouting deep holes. For grouting in heterogeneous strata, where the nature of rock discontinuities is subject to large variations in relation to the depth, full depth grouting is not recommended and stage grouting is preferred to packer groutig in such cases.

**(ii) Stage grouting**

Stage grouting is conducted to permit treatment of various zones individually, by grouting successively increasing depths. After sealing the upper zones. Stage grouting, in descending stages, can be carried out by adoptina the procedure given below.
Grouting is done by drilling the holes to a predetermined depth and grouting this initial depth at an appropriate pressure. Grout is then washed from the hole prior to its final set (within 2-4 hours) and the hole deepened for the next stage. Alternatively the grout is allowed to harden and redrilling is carried out through the hardened grout and the hole extended to the next stage. In another procedure called the one stage redrilled method, which is sometimes used, grout is washed out within a small depth of the top of the stage being grouted and only one stage is redrilled for proceeding to the next stale. In each of the above procedures the cycle of drilling-grouting-washing or redrilling is repeated until the required depth of the hole is reached.
For stage grouting. the connection at the top of the hole can be made directly to the header or by seating a packer at the top of the hole in the casing pipe. Alternatively, it is sometimes advantageous to install a packer immediately above the stage that is being grouted in order to isolate the upper portion of the hole. Higher pressures can then be permitted for grouting of the lower stage without causing upheaval in the higher stages. An alternative procedure would be to withdraw the grout pipe, after completing the grouting operation, by a distance equal to the depth of the stage grouted. After the initial set occurs, that is, about half an hour, the portion of the hole above the stage grouted may be washed. In this method the grout sets in the length of one stage, and it is necessary to redrill one stage before proceeding with further grouting. It is more convenient to install the packer at the top of the hole when one-stage redrilling procedure is adopted. Grouting with double packer is suitable where a few well defined seams or zones exist and the packers can be seated above and below such zones. Rotary drllling method is preferred when double packers are used. When packers can be seated and there is no risk of upheaval, grouting can be carried out with single packer in ascending stages. However, in many cases packers may function yet grout may overtravel and cause upheaval in the zones above the section being grouted. The method of stage grouting in descending order is therefore a more dependable method for badly jointed and fissured strata vulnerable to upheaval. In relatively compact rocks it may be more convenient to seat the packer at the top of the stale being grouted. The hole may then be washed, as soon as the period of initial set of cement is over to the entire depth of the hole up to the bottom of the stage in progress. On the other hand in strata vulnerable to upheaval, it may be necessary to allow the grout to set and form a sheath around the hole in order to enable high pressures to be used in the lower portions of the hole. In such cases, the washing and single stage redrilling procedure would have to be adopted.

**3.5.6. Pattern ,depth of holes and sequence of grouting**

The pattern and depth of holes is governed primarily by the design requirements and the nature. of. the rock. When the purpose is consolidation, the holes are arranged in a

regular pattern over the entire surface area required to be strengthened and the depth is determined by
the extent of broken rock as well as the structural requirements regarding the deformability
and strength of the foundation. When the purpose is impermeablllzation, the grout holes are arranged in a series of lines to form a curtain approximately perpendicular to tho direction of seepage. The depth of holes is dependent on design considerations as also on the depth of pervious rock and the configuration of zones of relatively impervious strata. The size of grout holes is generally less Important than the cost of drilling holes and the control of inclination. For grouting with cement, 38 mm holes are used. The advantage gained by drilling large holes does not often justify the increase in drilling costs. In long holes the diameter at the top of the holes may have to be larger than the final. diameter at the bottom of the hole to facilitate telescoping or to allow for the wear of the bit.

**3.5.7. Patterns of holes for curtain grouting**

**(i) single line grout curtains**

Single line. grout curtains are effective only in rocks having a fairly regular network of discontinuties with reasonably uniform size of openings. In such cases a curtain of adequate
Width can be achieved by grouting a single line of holes. In massive rocks with fine fissures
uplift control is primarily achieved by drainage and the grout curtain is used only as a supplementary measure to avoid concentrations of seepage which may. Exceed the the capacity of the drainage system.. single line curtain may serve this limited objective In comparatively tight rock formations.

In single line curtains it is costomary to drill a widely spaced system of primary holes, subscquently followed by secondary and tertiary holes at a progressively smaller spacing. Ihe
usual practice is to split the spacing from primary to the secondary and secondary to tertiary phase. One of the crtterra for deciding on the primary spacing is the length of expected intercommunication of grout between holes. The initial spacing usually varies between 6 m to 12m but the choice of spacing should be based on the geologlcal conditions and on experience. At every phase of . the grouting operation, the results of percolation tests and grout absorption data should be compared with the previous set of holes In order to decide
whether a further splitting of the spacing of holes is worthwhile. When no significant improvement is noticed either in terms of decrease of the grout absorption or water percolation, careful review should be made of the rock features, the nature of the rock and its relations to the patterns of holes. Sometimes it may be more advantageous to drill another line of holes at a different angle and orientation than to spht the spacing further. Spacings below one metre are rarely necessary and the requirement of a spacing

closer than one metre may often Indicate an unsuitable orientation and inclination of holes.

Possibly multiple line curtains may be.necessary.If the area is too limited, the setting time of the grout becomes important since it is not desirable to drill too close to a freshly grouted hole. Before pressure grouting is started, drilling of all the holes should be completed within a distance of 20 m of the hole to be grouted. Depending upon initial investigation and strata conditions the spacing of primary hole treatment should be decided. If the primary holes were spaced more than 6 m apart secondary holes should be drilled and grouted. On completion of primary holes spaced closer than 6 m or secondary holes ( when the primary holes are spaced more than 6 m ), should the percolation tests carried out in a few test holes indicate that further grouting of the area is necessary, secondary or tertiary treatment, as the case may be, should be carried out systematically thereafter in the whole area or In the particular section where the rock conditions are bad. Similarly tertiary holes should be taken over the whole area or the full length of the section which requires the treatment.

In addition to the systematic grouting of primary secondary or tertiary and subsequent boles it' may be necessary to drill and grout additional holes for treatment of peculiar geological features. such as faults, shear zones and weathered rock seams.

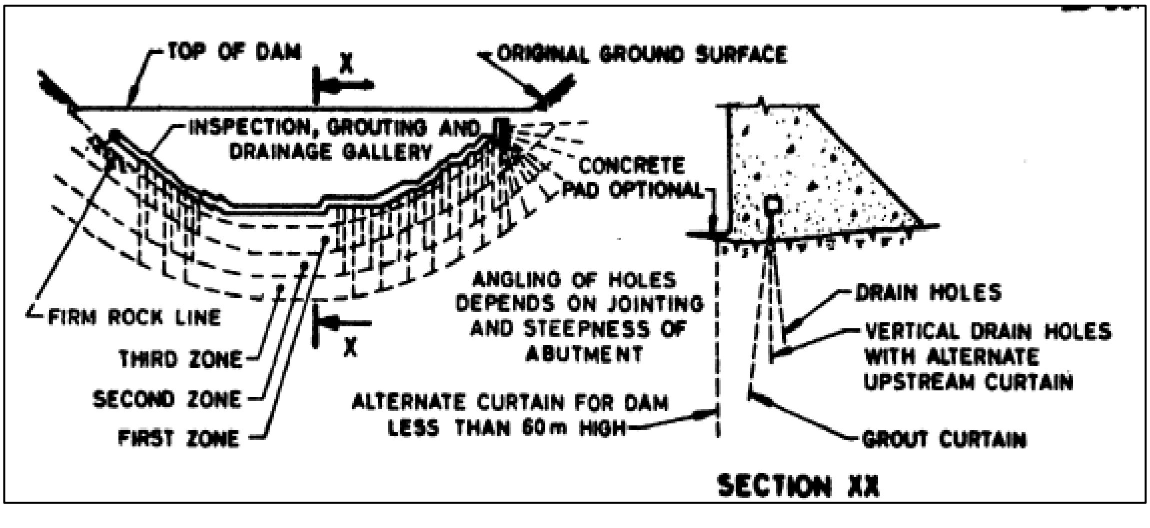



Fig 3.14 Profile of curtain grout holes in dam foundation. Source: IS 6066:1994

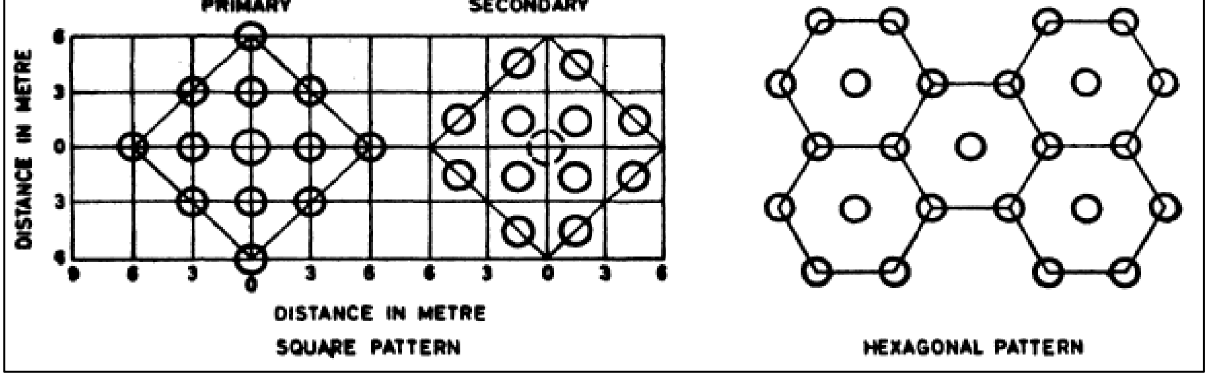



Fig 3.15 Profile of consolidation Grouting Source: IS 6066:1994

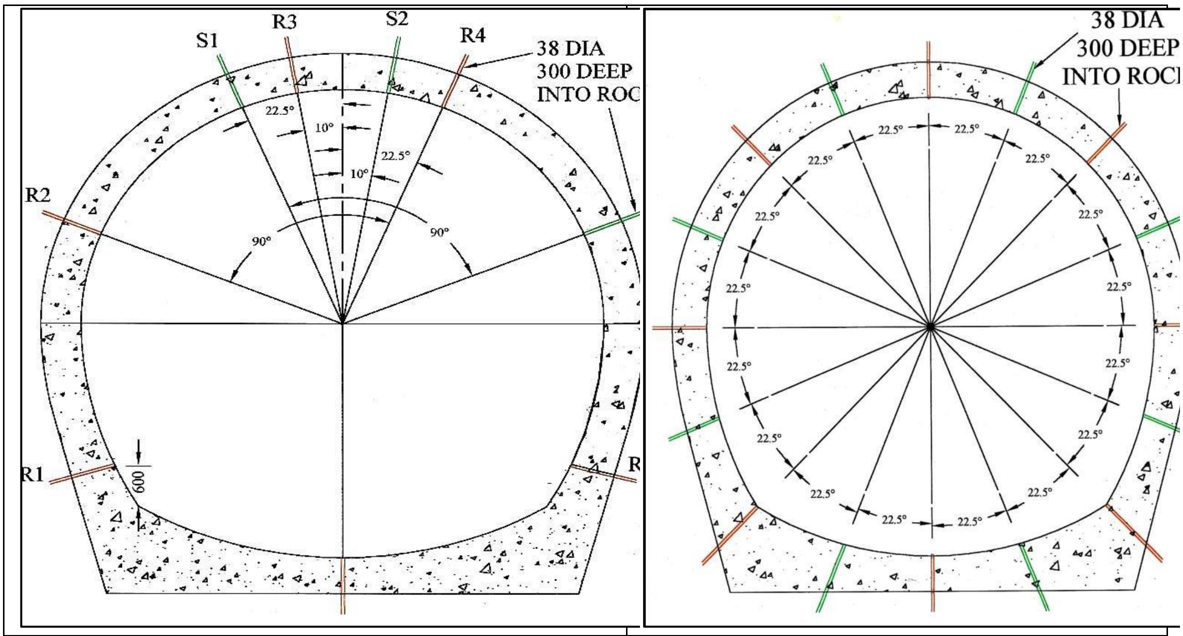



Fig 3.16 Patterns of contact grouting in Tunnel

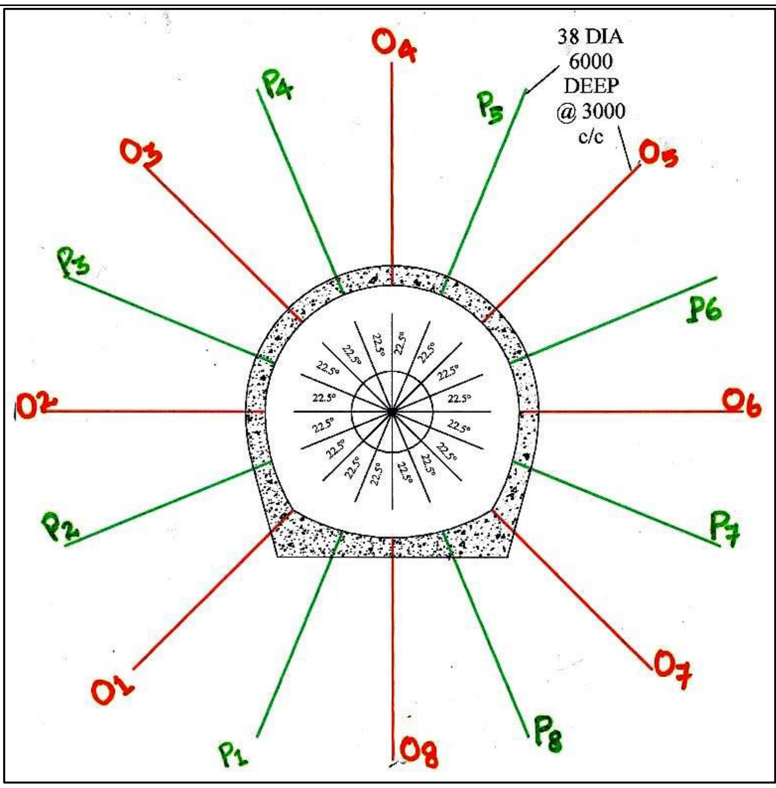



Fig 3.17 Patterns of consolidation grouting in Tunnel

### 3.5.8. Grouting Equipments

The grouting equipment should meet the Following requirements:

(a) It should be of sufficient size to meet the maximum demand of grout.
(b) It should be capable of prolonged operation at anticipated maximum pressure.
(c) It should be of sufficiently rugged construction to minimize delays from failure of essential parts.

(d) It should be permit quick cleaning by washing and provide to quick access to key parts in case of mechanical failure.

**(i) Grout mixers**

The mixers should have two tanks namely mixing tank and agitating tank. Mixers argenerally
cylindrical in shape, with the axis either horizontal or vertical and equipped with a system of power-driven paddles for mixing. Grout should be mixed in a mixer operating at 1500 r.p.m, or more. The higb speed of mixina serves the purpose of violently separating each cement grain from its neighbour thus permitting thorough wetting of every grain. This proves to be advantageous by chemically activating each grain to thorough hydration before reaching its
final resting place. Further individuul grains penetrate finer cracks more readily then ftocs.
Vertical, barrel-type mixers have proved satisfactory when small mixers are required for use
in confined or limited working spaces. This type of mixer consists essentially of a vertical barrel having a shaft with blades for mixing, driven by a motor mounted on top of the mixer above the barrel. Centrifugal pump mixers mix the grout by recirculating it through a high speed centrifugal pump. They are sometimes referred to as colloidal type mixers, but they do not achieve a true colloidal grout mix. However, they possess considerable merit and produce grout of excellent texture. When mixing sand-cement grouts, their action tends to guard against segregation.

**(ii) Agitator**

An agitator is a storage tank where the thoroughly mixed grout from the mixer is stirred by a slowly revolving paddle to keep the particles of unstable grout in suspension while awaiting injection.

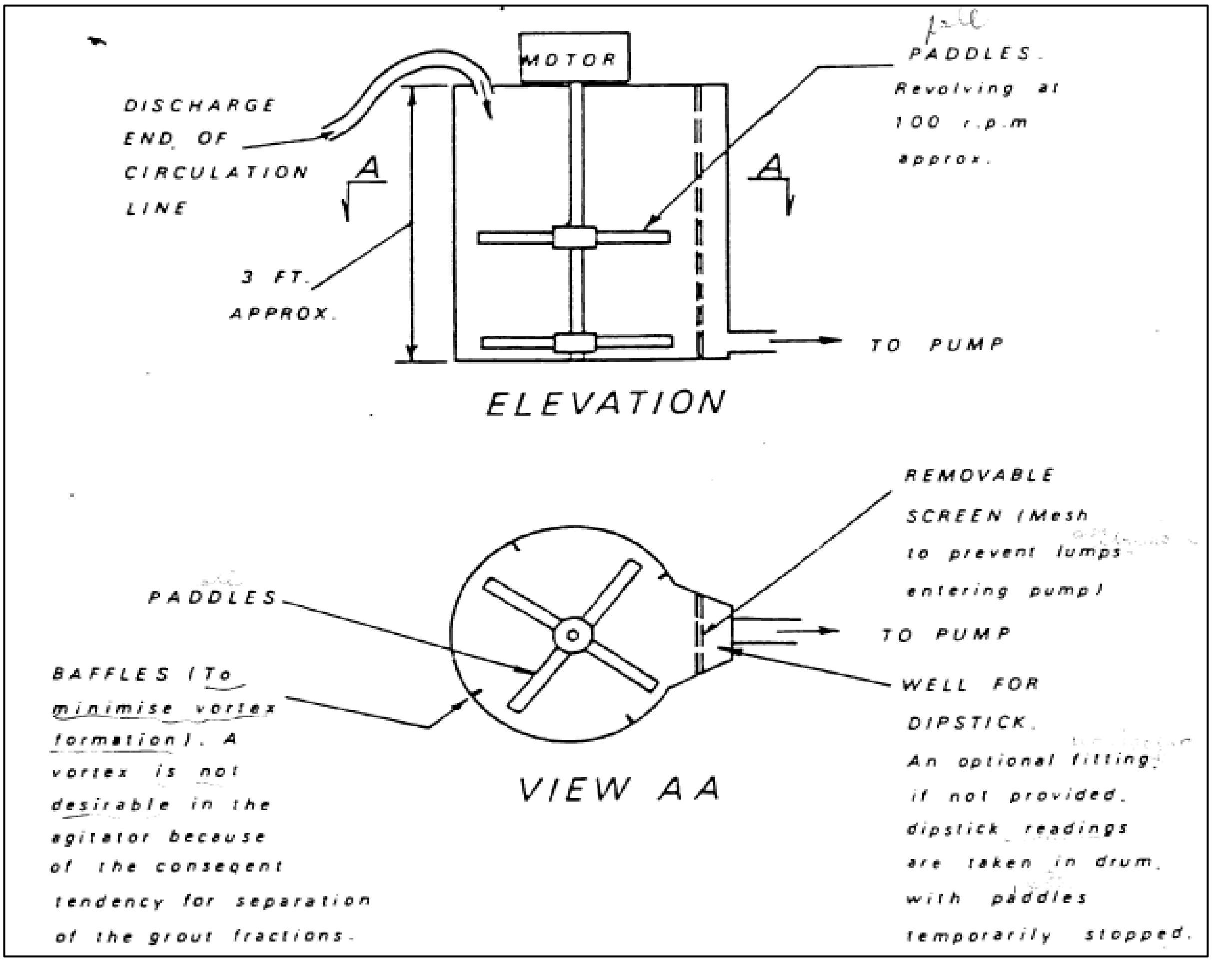



Fig 3.18 Typical view of agitator.source:Rock foundation grouting by Houlsby,1990

**(iii) Water meter**

Water meters for measuring the amount of water added to the grout mix should always be required by the specifications. Other methods, such as calibrated buckets and predetermined water level marking on the sides of the mixing tank should not be allowed. A water meter with a “reset-to-zero” feature is strongly recommended. This eliminates the risk of miscalculating the correct cumulative meter reading by the mixer operator.

**(iv) Pump**

Two common pumps used for grouting in underground structures are progressing helical cavity pump and piston pump. Progressing helical cavity pumps produce a continuous, uniform flow of grout into a hole at relatively constant pressure. These pumps are used primarily to pump grout mixtures of water, cement and bentonite. However, these are also capable of pumping sand grout mixes. The abrasiveness of the sand, however, increases the wear of the pump. A piston pump is better suited to pump sand mixes. Piston pumps are used predominantly to fill large voids, caverns, for backfill grouting behind precast concrete/steel liners and for contact grouting. One disadvantage of piston pump is that these deliver a pulsating pressure that makes pressure control difficult when constant or low pressures are required.

**(v) Pressure gauge**

Pressure gauges are used to monitor the injection pressure of the grout being delivered to the grout hole. The pressure gauge helps to ensure that the maximum allowable pressure is being applied to achieve the desired results of the grout programme design. It also helps to prevent the application of an injection pressure that is too high and could overstress the structures being grouted. The range of pressure reading of the gauge should be appropriate for the grout pressure being used. As gauges get easily damaged in the underground construction environment, they should be checked for calibrations very regularly.

**(vi) Gauge savers**

The grout mix should never be allowed to come into direct contact with the pressure gauge. Therefore, a protective medium must be used to separate the grout from the gauge. A gauge saver, also called a "diaphragm seal" can be used for this purpose. Within the body of a gauge saver, the upper portion is isolated from the grout by a diaphragm. The area between the top of the diaphragm and the pressure gauge is completely filled with a suitable fluid, usually oil.

**(vii) Packers**

Packers are used to seal off or isolate a portion of grout hole that allows grout to be injected under pressure into a specified section of the hole. Packers are installed either at the top of the hole, also called the hole collar or at other locations along the length of the hole. The packers are set at the top of the hole for contact grouting. It is also most often installed at the collar of the hole for consolidation grouting operations, when the hole is grouted in single stage. Holes are grouted using a single-packer or double-packer arrangements. In a single-packer arrangement the grout is discharged into the hole just below the location of the packers. In the double-packer arrangement, the grout is discharged into an isolated section of the grout hole located between an upper and lower packer. The single-packer arrangement is used in contact and most of the consolidation grouting operations. The double-packer arrangement is used to grout isolated sections in both consolidation and curtain grouting operations. Two common types of packers are the mechanically activated packer and the pneumatic/hydraulic inflatable packer.

Fig 3.19 Pneumatic packers for curtain gruoting

Fig 3.20 Mechanical packers for contact grouting

**(viii) Grout mterials**

Grout materials include cement, sand, water and admixtures, if any. The quality of these materials needs to be checked from time to time. The tests on the cement such as consistency, setting time and the compressive strength were performed regularly. Sand passing through 2.36 mm sieve was used in the grouting. Water to be used in grouting should be free from organic matter and deleterious materials. Generally water suitable for drinking purpose is also suitable for use in grouting. Water for use in the grouting operation was got tested from Q & C department of NEEPCO and it was found to be harmless for use.

Ordinary portland cement (OPC ,IS269:1989) is preferred in grouting the rock. However, admixtures such as flyash, silica fume, non-shrink compound and accelerators may also be added in the grout depending upon the requirement. Common additives such as bentonite, superplasticizers, fly ash, silica fumes, accelerators, sodium silicate and thixotropic modifier can be used in various situations (Table 3.9).

Table 3.9 Significance and proportioning of additives in grout

| Additive | purpose | Dosage |
|---|---|---|
| Bentonite | (i)Improving stability under pressure<br>(ii)Reducing shrinkage and bleeding of grout | 1 to 2% by weight of cement |
| Super plastisizers | Dispersing agent to reduce viscosity of grout | 0.5 to 1% by weight of cement |
| Fly ash | (i)As a pozzolana to replace cement<br>(ii)To produce a grout better resistant to aggressive groundwater | Up to 25 % |
| Silica fume | To produce a stronger, less permeable grout with enhanced stability and resistance to pressure filtration | Up to 10% |
| Accelerators | For faster setting of grout in cold weathers | - |

| | | |
|---|---|---|
| Sodium silicate | Used under high inflow/high pressure conditions | - |
| Thixotropic modifier | Used in flowing water conditions to produce a cohesive, water repellent grout that resists washout | - |

### 3.5.9. Selection of grout

Water cement ratio is the important factor, as lower the water cement ratio higher the strength
of grout mixture. However, for flowability and filling of crevices, water cement ratio is required to be adjusted. The choice of grout type is a function of the aperture of the rock joints and cost. For grouting that involve filling large fissures, the use of stable grouts formulated with locally available OPC is recommended. The various apertures that can be
grouted with OPC (Houlsby 1990) are as follows:
(i)500 microns without special care
(ii)400 microns with extra care using high quality grout
Snow (1968) proposed the following equation for estimating the hydraulic conductivity of a
rock mass with two sets of planar fractures:
Where,

$$K= \frac{\gamma_w N b^3}{6\upsilon} \qquad \text{(i)}$$

K = Hydraulic conductivity of rock (m/s), b = Joint aperture (m) (.001Nsec/$m^2$),
$\gamma_w$ = Unit weight of water (kN/$m^3$), ν = Dynamic viscosity of water,
N = Joints per metre.
Groutability of fine cracks is related to the width of the crack and the grain size of the grout
material, expressed as a groutability ratio of rock in the following formula (Weaver, 1991):
Groutability Ratio = Width of fissure/$D_{95}$ of Grout
For groutability ratio greater than 5, grouting is considered consistently possible. For groutability ratio less than 2, grouting is not considered possible. The $D_{95}$ and practical grouting range for various grouts are provided in Table 3.10

Table 3.10 Joint aperture range for various cement grouts

| Cement product | $D_{95}$ of grout | Practical Joint aperture range |
|---|---|---|
| Ordinary portland cement | 80-100 microns | >400 microns |
| High early strength cement | 40-60 microns | >200 microns |
| Microfine cement | 10-12 microns | >50 microns |

### 3.5.10 Control of grouting operation

**(i) Pressure**

The pressure should be adequate to achieve the desired grout and the pressure should be limited so as to avoid disturbance and upheaval of the ground and should take into account reservoir pressure.

For structures on rock foundations, it is a basic requirement that no disturbance should be caused to the surface zones of the foundation by the grouting operation. When grouting is undertaken below an existing structure no upheaval of the foundation can be allowed as it would have very harmful consequences on the structure and/or equipment.

In general, the disturbance caused by grouting is dependent more on the manner in which the pressure is developed and the nature of the rock than on the absolute magnitude of pressure. Relatively hiaher pressures can be sustained without damage to the foundations, when pressure is built up gradually, as resistance to flow is developed by deposition of grout. On the other hand, when pressures are raised hastily, damage could occur .even at relatively low pressures. In general, horizontal stratified or low dipping rocks are more vulnerable to disturbance by grouting pressure than fractured igneous or metamorpbic rocks or steeply dipped sedimentary rocks. Rocks previously subjected to folding and fracturing or rocks in the process of adjustment after removal of overburden load are also more vulnerable to disturbance.

It is always advisable to begin with a low initial pressure say 0.10 to 0.25 kg/cm$^2$/m of overburden, and build-up the pressure gradually. Initially the rate of intake may be 20 l/(min to 30 l/min. In order to avoid the premature build-up of high pressure a general guideline should be followed that the pressure should be raised only when the intake rate falls below 5 l/min. When surface leaks develop, pressure should be immediately reduced. Subsurface cracking may sometimes be indicated by an abrupt rise in the rate of intake after grouting at a constant value of pressure for a considerable period.

The true pressure at any depth should take into account the pressure head caused by the weight of the grout in the hole, this correction in kg/cm$^2$ may be computed by multiplying the depth of the hole in metres by factors relative to the water-cement ratio given in Table 3.11 and added to the pressure gauge reading at the top of the grout hole.

Grout mixture ( ratios by weight of water and cement) ranging from 5: 1 to 0·8 : 1 are recommended. It is only in exceptional circumstances that mixtures leaner than 10 : 1 need be used. The choice oC grout mixtures may be based on results of percolation tests conducted

prior to grouting. The ideal would be to conduct a percolation test in each hole.

Table 3.11 Multiplying factor for pressure correction (source IS6066:1994)

| Water-cement ratio | Factor | Water-cement ratio | factor |
|---|---|---|---|
| 0.75 | 0.151 | 2.50 | 0.118 |
| 1.00 | 0.140 | 2.75 | 0.117 |
| 1.25 | 0.131 | 3.00 | 0.112 |
| 1.50 | 0.127 | 4.00 | 0.110 |
| 1.75 | 0.123 | 5.00 | 0.107 |

| | | | |
|---|---|---|---|
| 2.00 | 0.121 | 10.00 | 0.102 |
| 2.25 | 0.119 | | |

for each stage. However, the number of percolation tests may be reduced if extent of zones of different types of rock and rock characteristics can be established on the basis of geological evidence and results of initial experimental grouting operations. It is inadvisable to relax the requirements of percolation testing in the initial stages of grouting and grout absorptions at low water-cement ratios are a poor substitute for water percolation tests.

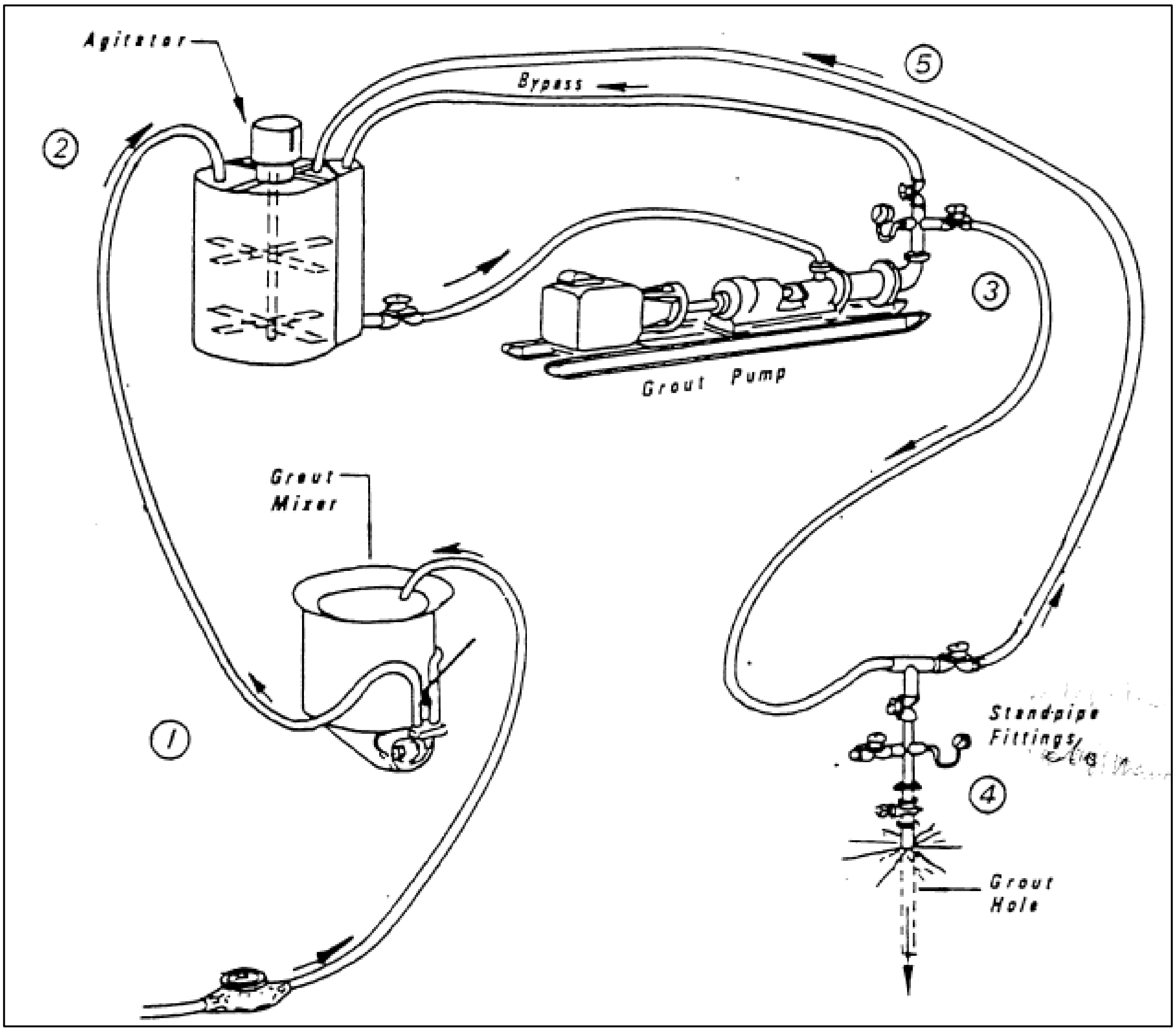



Fig 3.21 Grouting Arrangement ( source:Houlsby 1990)

Fig 3.22 Atlas Uni Comp Grout Pump

Fig 3.23 Pressure Gauge and Water Flow Meter

Fig 3.24 Lugeon values test with mechanical packer.

Fig 3.25 Lugeon values test with pneumatic packer.

# Chapter 4

# RESULTS & ANALYSIS

### 4.1 Introduction

This chapter deals with the various results obtained during my entire experimental works.

I am going to describes various results i.e drilling , Lugeon values test and curtain grouting one by one.

### 4.2 Results of drilling

The core rcovery in concrete section is lies between 80 to 100 percent. But core recovery in sedimentary rock which is mainly sand stone having greyish in colour,soft and friable in nature is zero. There is no core recovery in rock mass of dam foundation because the core barrel can not prevent falling down of the sand stone during pulling out process. Even triple tube barrel is used for core recovery but it is not able to recover the core of sand stone. Samples are wash out during the drilling process. In some hole core recovery is found in rock mass foundation but it is less than 10 percent.

**Core recovery (CR)** $=\frac{\textbf{total length of rock recoveed}}{\textbf{total core run length}}$

### 4.3 Results of Lugeon values test or WPT

The tests were conducted in five stages including increasing and decreasing pressure. The minimum pressure apllied during test is 1kg/cm$^2$ and maximum pressure is 10 kg/cm$^2$. At each stage a constant pressure is applied for an interval of 5 minutes while pumping water. Water pressure and flow rate are measured after every 5 minutes. Using the avearage values of water pressure and flow rate at each stage the average Lugeon value of the rockmass is determined. Lugeon is the conductivity required for a flow area of 1 litre per minute per meter of the borehole interval under constant pressure. The Lugeon value is calculated as follows and then the average representative value is selected for the tested rock mass. Durig entire test single packer method is used.

**Lugeon value (LV)** $= \frac{\mathbf{Q \times P_o}}{\mathbf{L \times P}}$ **litre/m/min.**

(ii)

Where,

Q= flow rate in litre/min.

L= length of the bore hole test interval

$\mathbf{P_o}$ = Reference pressure = 1MPa = 10.19 kg/cm$^2$

P = Pressure applied during test.

Lugeon test shall be performed in descending order by limiting the water pressure as follows:

Table 4.1Guide line for applying Gauge pressure

| S.N | Depth of hole | Water pressure in kg/cm$^2$ |
|---|---|---|
| 1 | 0m-5m, 5m-10m | 1-3-5-3-1 |
| 2 | 10m-15m, 15m- 20m | 1-3-5-7-5-3-1 |
| 3 | 20m-25m,25m-30m,30m-35m,35m-40m | 1-3-5-7-10-7-3-1 |

Table 4.2 Lugeon value test(BH-P32) Depth- 5m to 9m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth (m) | 5.00 to 9.00 | | | | | |
| | | | | | | |
| S No | Pressure(kg/cm$^2$) | Time (min) | Water meter reading(litres) | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3042.3 | 3053 | 10.70 | 4.36 |
| 2 | 3 | 5 | 3053 | 3071.1 | 18.10 | 2.46 |
| 3 | 5 | 5 | 3071.1 | 3087.4 | 16.30 | 1.33 |
| 4 | 3 | 5 | 3087.4 | 3092.2 | 4.80 | 0.65 |
| 5 | 1 | 5 | 3092.2 | 3099.7 | 7.50 | 1.02 |

Table 4.3 Lugeon value test(BH-P32) Depth- 9m to 14m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth (m) | 9.00 to 14.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading (litres) | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3110.6 | 3134.7 | 24.1 | 9.82 |
| 2 | 3 | 5 | 3134.7 | 3158.1 | 23.4 | 3.18 |
| 3 | 5 | 5 | 3158.1 | 3172.4 | 14.3 | 1.17 |
| 4 | 3 | 5 | 3172.4 | 3183.3 | 10.9 | 1.48 |
| 5 | 1 | 5 | 3183.3 | 3190.6 | 7.3 | 2.97 |

Table 4.4 Lugeon value test(BH-P32) Depth- 14m to 19m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 14.00 to 19.00 | | | | | |
| | | | | | | |
| S No | Pressure ($kg/cm^2$) | Time(min) | Water meter reading (litres) | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3198.4 | 3200.1 | 1.7 | 0.69 |
| 2 | 3 | 5 | 3200.1 | 3201.1 | 1.0 | 0.14 |
| 3 | 5 | 5 | 3201.1 | 3201.6 | 0.5 | 0.04 |
| 4 | 7 | 5 | 3201.6 | 3203.1 | 1.5 | 0.09 |
| 5 | 5 | 5 | 3203.1 | 3203.1 | 0.0 | 0.00 |
| 6 | 3 | 5 | 3203.1 | 3203.1 | 0.0 | 0.00 |
| 7 | 1 | 5 | 3203.1 | 3203.1 | 0.0 | 0.00 |

Table 4.5 Lugeon value test( BH-P32) Depth-19m to 24m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 19.00 to 24.00 | | | | | |
| | | | | | | |
| S No | Pressure($kg/cm^2$) | Time (min) | Water meter reading(litres) | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3203.6 | 3205.1 | 1.5 | 0.61 |
| 2 | 3 | 5 | 3205.1 | 3206.3 | 1.2 | 0.16 |
| 3 | 5 | 5 | 3206.3 | 3207.2 | 0.9 | 0.07 |
| 4 | 7 | 5 | 3207.2 | 3207.9 | 0.7 | 0.04 |
| 5 | 5 | 5 | 3207.9 | 3207.9 | 0.0 | 0.00 |
| 6 | 3 | 5 | 3207.9 | 3207.9 | 0.0 | 0.00 |
| 7 | 1 | 5 | 3207.9 | 3207.9 | 0.0 | 0.00 |

Table 4.6 Lugeon value test (BH-P32) Depth- 24m to 29m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 24.00 to 29.00 | | | | | |
| | | | | | | |
| S No | Pressure(kg/cm$^2$) | Time (min) | Water meter reading (litres) | | QTY (Litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3205.1 | 3205.9 | 0.8 | 0.33 |
| 2 | 3 | 5 | 3205.9 | 3207.2 | 1.3 | 0.18 |
| 3 | 5 | 5 | 3207.2 | 3208.2 | 1.0 | 0.08 |
| 4 | 7 | 5 | 3208.2 | 3208.5 | 0.3 | 0.02 |
| 5 | 5 | 5 | 3208.5 | 3208.5 | 0.0 | 0.00 |
| 6 | 3 | 5 | 3208.5 | 3208.5 | 0.0 | 0.00 |
| 7 | 1 | 5 | 3208.5 | 3208.5 | 0.0 | 0.00 |

Table 4.7 Lugeon value test(BH-P32) Depth- 29m to 34m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 29.00 to 34.00 | | | | | |
| | | | | | | |
| S No | Pressure(kg/cm$^2$) | Time (min) | Water meter reading(litres) | | QTY (Litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3211 | 3211.7 | 0.7 | 0.28 |
| 2 | 3 | 5 | 3211.7 | 3212.5 | 0.8 | 0.11 |
| 3 | 5 | 5 | 3212.5 | 3212.9 | 0.4 | 0.03 |
| 4 | 7 | 5 | 3212.9 | 3213.3 | 0.4 | 0.02 |
| 5 | 10 | 5 | 3213.3 | 3214 | 0.7 | 0.02 |
| 6 | 7 | 5 | 3214 | 3214 | 0.0 | 0.00 |
| 7 | 5 | 5 | 3214 | 3214 | 0.0 | 0.00 |
| 8 | 3 | 5 | 3214 | 3214 | 0.0 | 0.00 |
| 9 | 1 | 5 | 3214 | 3214 | 0.0 | 0.00 |

Table 4.8 Lugeon value test (BH-P32) Depth -34.0 m to 39.0 m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 34.00 to 39.00 | | | | | |
| | | | | | | |
| S No | Pressure(kg/cm$^2$) | Time (min) | Water meter reading(litres) | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3218.1 | 3219 | 0.9 | 0.37 |
| 2 | 3 | 5 | 3219 | 3219.6 | 0.6 | 0.08 |
| 3 | 5 | 5 | 3219.6 | 3220.2 | 0.6 | 0.05 |
| 4 | 7 | 5 | 3220.2 | 3221.1 | 0.9 | 0.05 |
| 5 | 10 | 5 | 3221.1 | 3221.6 | 0.5 | 0.02 |
| 6 | 7 | 5 | 3221.6 | 3221.6 | 0.0 | 0.00 |
| 7 | 5 | 5 | 3221.6 | 3221.6 | 0.0 | 0.00 |
| 8 | 3 | 5 | 3221.6 | 3221.6 | 0.0 | 0.00 |
| 9 | 1 | 5 | 3221.6 | 3221.6 | 0.0 | 0.00 |

Table 4.9 Lugeon value test (BH-P32) Depth- 39m to 43m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P32 | | | | | | |
| Location-B3 EL-209 | | | | | | |
| Depth(m) | 39.00 to 43.00 | | | | | |
| | | | | | | |
| S No | Pressure(kg/cm$^2$) | Time (min) | Water meter reading(litres) | | QTY (Litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 3234.2 | 3234.5 | 0.3 | 0.12 |
| 2 | 3 | 5 | 3234.5 | 3236.1 | 1.6 | 0.22 |
| 3 | 5 | 5 | 3236.1 | 3234.1 | 2.0 | 0.16 |
| 4 | 7 | 5 | 3234.1 | 3255.8 | 21.7 | 1.26 |
| 5 | 10 | 5 | 3255.8 | 3268.5 | 12.7 | 0.52 |
| 6 | 7 | 5 | 3268.5 | 3268.5 | 0.0 | 0.00 |
| 7 | 5 | 5 | 3268.5 | 3268.5 | 0.0 | 0.00 |
| 8 | 3 | 5 | 3268.5 | 3268.5 | 0.0 | 0.00 |
| 9 | 1 | 5 | 3268.5 | 3268.5 | 0.0 | 0.00 |

Table 4.10 Lugeon value test (BH-P4)Depth3m to 8m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 3.00 to 8.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY (litres) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 900 | 907 | 7.0 | 2.85 |
| 2 | 3 | 5 | 907 | 921 | 14.0 | 1.90 |
| 3 | 5 | 5 | 921 | 994 | 73.0 | 5.95 |
| 4 | 3 | 5 | 994 | 1014 | 20.0 | 2.72 |
| 5 | 1 | 5 | 1014 | 1019 | 5.0 | 2.04 |

Table 4.11 Lugeon value test (BH-P4)Depth- 8m to 13m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 8.00 to 13.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1029 | 1040.60 | 11.6 | 4.73 |
| 2 | 3 | 5 | 1040.60 | 1048 | 7.4 | 1.01 |
| 3 | 5 | 5 | 1048 | 1065 | 17.0 | 1.39 |
| 4 | 3 | 5 | 1065 | 1065 | 0.0 | 0.00 |
| 5 | 1 | 5 | 1065 | 1065 | 0.0 | 0.00 |

Table 4.12 Lugeon value test (BH-P4) Depth 13 m to 18m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 13.00 to 18.00 | | | | | |
| | | | | | | |
| S No | Pressure(Kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1019 | 1019.4 | 0.4 | 0.16 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2 | 3 | 5 | 1019.4 | 1019.9 | 0.5 | 0.07 |
| 3 | 5 | 5 | 1019.9 | 1026.8 | 6.9 | 0.56 |
| 4 | 7 | 5 | 1026.8 | 1028.9 | 2.1 | 0.12 |
| 5 | 5 | 5 | 1028.9 | 1028.9 | 0.0 | 0.00 |
| 6 | 3 | 5 | 1028.9 | 1028.9 | 0.0 | 0.00 |
| 7 | 1 | 5 | 1028.9 | 1028.9 | 0.0 | 0.00 |

Table 4.13 Lugeon value test (BH-P4) Depth 18 m to 23m

| Project-2*55 MW Pare HEP | | | | | | |
|---|---|---|---|---|---|---|
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth - | 18.00 to 23.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1031 | 1034 | 3.0 | 1.22 |
| 2 | 3 | 5 | 1034 | 1034.2 | 0.2 | 0.03 |
| 3 | 5 | 5 | 1034.2 | 1045 | 10.8 | 0.88 |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 5 | 5 | - | - | - | - |
| 6 | 3 | 5 | - | - | - | - |
| 7 | 1 | 5 | - | - | - | - |
| After 5 kg pressure packer not fixed due to increased size of bore hole. | | | | | | |

Table 4.14 Lugeon value test (BH-P4) Depth 23 m to 28m

| Project-2*55 MW Pare HEP | | | | | | |
|---|---|---|---|---|---|---|
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 23.00 to 28.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1050.4 | 1051.2 | 0.8 | 0.33 |
| 2 | 3 | 5 | 1051.2 | 1052.8 | 1.6 | 0.22 |
| 3 | 5 | 5 | 1052.8 | 1054.3 | 1.5 | 0.12 |
| 4 | 7 | 5 | 1054.3 | 1059.6 | 5.3 | 0.31 |
| 5 | 10 | 5 | 1059.6 | 1060.1 | 0.5 | 0.02 |
| 6 | 7 | 5 | 1060.1 | 1060.1 | 0.0 | 0.00 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 7 | 5 | 5 | 1060.1 | 1060.2 | 0.1 | 0.01 |
| 8 | 3 | 5 | 1060.2 | 1060.2 | 0.0 | 0.00 |
| 9 | 1 | 5 | 1060.2 | 1060.7 | 0.5 | 0.2 |

Table 4.15 Lugeon value test (BH-P4) Depth 28 m to 33m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth - | 28.00 TO 33.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1062.8 | 1063.8 | 1.0 | 0.41 |
| 2 | 3 | 5 | 1063.8 | 1064.9 | 1.1 | 0.15 |
| 3 | 5 | 5 | 1064.9 | 1065.9 | 1.0 | 0.08 |
| 4 | 7 | 5 | 1065.9 | 1066.8 | 0.9 | 0.05 |
| 5 | 10 | 5 | 1066.8 | 1067.4 | 0.6 | 0.02 |
| 6 | 7 | 5 | 1067.4 | 1067.5 | 0.1 | 0.01 |
| 7 | 5 | 5 | 1067.5 | 1067.5 | 0.0 | 0.00 |
| 8 | 3 | 5 | 1067.5 | 1067.5 | 0.0 | 0.00 |
| 9 | 1 | 5 | 1067.5 | 1067.5 | 0.0 | 0.00 |

Table 4.16 Lugeon value test (BH-P4) Depth 33 m to 38m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 33.00 TO 38.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1088 | 1088.2 | 0.2 | 0.08 |
| 2 | 3 | 5 | 1088.2 | 1090.9 | 2.7 | 0.37 |
| 3 | 5 | 5 | 1090.9 | 1093.7 | 2.8 | 0.23 |
| 4 | 7 | 5 | 1093.7 | 1099.3 | 5.6 | 0.33 |
| 5 | 10 | 5 | 1099.3 | 1106.2 | 6.9 | 0.28 |
| 6 | 7 | 5 | - | - | - | - |
| 7 | 5 | 5 | - | - | - | - |
| 8 | 3 | 5 | - | - | - | - |

| | | | | | | |
|---|---|---|---|---|---|---|
| 9 | 1 | 5 | - | - | - | - |
| 0.2Pressure apply 10kg but pressure not performed due to rock fractured. | | | | | | |

Table 4.17 Lugeon value test (BH-P4) Depth 38 m to 43m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P4 | | | | | | |
| Location-B6 EL-188 | | | | | | |
| Depth(m) | 38.00 TO 43.00 | | | | | |
| | | | | | | |
| S No | Pressure ($kg/cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1134.9 | 1135.5 | 0.6 | 0.24 |
| 2 | 3 | 5 | - | - | - | - |
| 3 | 5 | 5 | - | - | - | - |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 10 | 5 | - | - | - | - |
| 6 | 7 | 5 | - | - | - | - |
| 7 | 5 | 5 | - | - | - | - |
| 8 | 3 | 5 | - | - | - | - |
| 9 | 1 | 5 | - | - | - | - |
| Packer not fixed due to water leakage from the side of casing. | | | | | | |

Table 4.18 Lugeon value test (BH-P13) Depth 3 m to 8m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m) | 3.00 TO 8.00 | | | | | |
| | | | | | | |
| S No | Pressure ($kg/cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2615.6 | 2615.9 | 0.3 | 0.13 |
| 2 | 3 | 5 | 2615.9 | 2618.8 | 2.9 | 0.39 |
| 3 | 5 | 5 | 2618.8 | 2626.5 | 7.7 | 0.63 |
| 4 | 3 | 5 | 2626.5 | 2626.5 | 0.0 | 0.00 |
| 5 | 1 | 5 | 2626.5 | 2626.5 | 0.0 | 0.00 |

Table 4.19 Lugeon value test (BH-P13) Depth 8 m to 13m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m ) | 8.00 TO 13.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY (Lts) | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2629.8 | 2629.9 | 0.1 | 0.04 |
| 2 | 3 | 5 | 2629.9 | 2630.5 | 0.6 | 0.08 |
| 3 | 5 | 5 | 2630.5 | 2630.8 | 0.3 | 0.02 |
| 4 | 3 | 5 | 2630.8 | 2630.8 | 0.0 | 0.00 |
| 5 | 1 | 5 | 2630.8 | 2630.8 | 0.0 | 0.00 |

Table 4.20 Lugeon value test (BH-P13) Depth 13 m to 18m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m ) | 13.00 TO 18.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2630.5 | 2630.6 | 0.1 | 0.04 |
| 2 | 3 | 5 | 2630.6 | 2630.7 | 0.1 | 0.01 |
| 3 | 5 | 5 | 2630.7 | 2630.9 | 0.2 | 0.02 |
| 4 | 7 | 5 | 2630.9 | 2631 | 0.1 | 0.01 |
| 5 | 5 | 5 | 2631 | 2631 | 0.0 | 0.00 |
| 6 | 3 | 5 | 2631 | 2631 | 0.0 | 0.00 |
| 7 | 1 | 5 | 2631 | 2631 | 0.0 | 0.00 |

Table 4.21 Lugeon value test (BH-P13) Depth 18 m to 23m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m) | 18.00 TO 23.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/$cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2634.4 | 2634.9 | 0.5 | 0.20 |
| 2 | 3 | 5 | 2634.9 | 2635.5 | 0.6 | 0.08 |
| 3 | 5 | 5 | 2635.5 | 2635.6 | 0.1 | 0.04 |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 5 | 5 | - | - | - | - |
| 6 | 3 | 5 | - | - | - | - |
| 7 | 1 | 5 | - | - | - | - |
| Packer not fixed due to soft rock and water return from the side of the casing. | | | | | | |

Table 4.22 Lugeon value test (BH-P13) Depth 23 m to 28m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m) | 23.00 TO 28.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/$cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2766.9 | 2767.6 | 0.7 | 0.28 |
| 2 | 3 | 5 | 2767.6 | 2768.7 | 1.1 | 0.15 |
| 3 | 5 | 5 | 2768.7 | 2769.6 | 0.9 | 0.07 |
| 4 | 7 | 5 | 2769.6 | 2770.3 | 0.7 | 0.04 |
| 5 | 10 | 5 | 2770.3 | 2770.8 | 0.5 | 0.02 |
| 6 | 7 | 5 | 2770.8 | 2770.8 | 0.0 | 0.00 |
| 7 | 5 | 5 | 2770.8 | 2770.8 | 0.0 | 0.00 |
| 8 | 3 | 5 | 2770.8 | 2770.8 | 0.0 | 0.00 |
| 9 | 1 | 5 | 2770.8 | 2770.8 | 0.0 | 0.00 |

Table 4.23 Lugeon value test (BH-P13) Depth- 28 m to 33m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m) | 28.00 TO 33.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2817 | 2819.2 | 2.20 | 0.89 |
| 2 | 3 | 5 | 2819.2 | 2819.8 | 0.60 | 0.08 |
| 3 | 5 | 5 | 2819.8 | 2820.9 | 1.10 | 0.09 |
| 4 | 7 | 5 | 2820.9 | 2821.5 | 0.60 | 0.03 |
| 5 | 10 | 5 | 2821.5 | 2821.9 | 0.40 | 0.02 |
| 6 | 7 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 7 | 5 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 8 | 3 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 9 | 1 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |

Table 4.24 Lugeon value test (BH-P13) Depth 33 m to 38m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m ) | 33.00 TO 38.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | 0.90 | 0.37 |
| 1 | 1 | 5 | 2811.7 | 2812.6 | 0.40 | 0.05 |
| 2 | 3 | 5 | 2812.6 | 2813 | 0.50 | 0.04 |
| 3 | 5 | 5 | 2813 | 2813.5 | 0.60 | 0.03 |
| 4 | 7 | 5 | 2813.5 | 2814.1 | 3.10 | 0.13 |
| 5 | 10 | 5 | 2814.1 | 2817.2 | 2.70 | 0.16 |
| 6 | 7 | 5 | 2817.2 | 2819.9 | 0.00 | 0.00 |
| 7 | 5 | 5 | 2819.9 | 2819.9 | 0.00 | 0.00 |
| 8 | 3 | 5 | 2819.9 | 2819.9 | 0.00 | 0.00 |
| 9 | 1 | 5 | 2819.9 | 2819.9 | 0.00 | 0.00 |

Table 4.25 Lugeon value test (BH-P13) Depth 38 m to 43m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P13 | | | | | | |
| Location-B5 EL-188 | | | | | | |
| Depth(m) | 38.00 TO 43.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | 1.40 | 0.57 |
| 1 | 1 | 5 | 2818.9 | 2820.3 | 0.60 | 0.08 |
| 2 | 3 | 5 | 2820.3 | 2820.9 | 0.30 | 0.02 |
| 3 | 5 | 5 | 2820.9 | 2821.2 | 0.30 | 0.02 |
| 4 | 7 | 5 | 2821.2 | 2821.5 | 0.40 | 0.00 |
| 5 | 10 | 5 | 2821.5 | 2821.9 | 0.00 | 0.00 |
| 6 | 7 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 7 | 5 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 8 | 3 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |
| 9 | 1 | 5 | 2821.9 | 2821.9 | 0.00 | 0.00 |

Table 4.26 Lugeon value test (BH-P8) Depth 3 m to 8m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 3.00 TO 8.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1327.8 | 1328.2 | 0.40 | 0.16 |
| 2 | 3 | 5 | 1328.2 | 1329 | 0.80 | 0.11 |
| 3 | 5 | 5 | 1329 | 1329.7 | 0.70 | 0.06 |
| 4 | 3 | 5 | 1329.7 | 1329.8 | 0.10 | 0.01 |
| 5 | 1 | 5 | 1329.8 | 1329.8 | 0.00 | 0.00 |

Table 4.27 Lugeon value test (BH-P8) Depth 8 m to 13m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 8.00 TO 13.00 | | | | | |
| | | | | | | |
| S No | Pressure ($kg/cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1334.1 | 1334.6 | 0.50 | 0.20 |
| 2 | 3 | 5 | 1334.6 | 1335.1 | 0.50 | 0.07 |
| 3 | 5 | 5 | 1335.1 | 1335.8 | 0.70 | 0.06 |
| 4 | 3 | 5 | 1335.8 | 1335.8 | 0.00 | 0.00 |
| 5 | 1 | 5 | 1335.8 | 1335.8 | 0.0 | 0.00 |

Table 4.28 Lugeon value test (BH-P8) Depth 13 m to 18m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 13.00 TO 18.00 | | | | | |
| | | | | | | |
| S No | Pressure ($kg/cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1345.6 | 1346.1 | 0.50 | 0.20 |
| 2 | 3 | 5 | 1346.1 | 1346.9 | 0.80 | 0.11 |
| 3 | 5 | 5 | 1346.9 | 1348.5 | 1.60 | 0.13 |
| 4 | 7 | 5 | 1348.5 | 1349.4 | 0.90 | 0.05 |
| 5 | 5 | 5 | 1349.4 | 1349.5 | 0.10 | 0.01 |
| 6 | 3 | 5 | 1349.5 | 1349.5 | 0.00 | 0.00 |
| 7 | 1 | 5 | 1349.5 | 1349.5 | 0.00 | 0.00 |

Table 4.29 Lugeon value test (BH-P8) Depth 18 m to 23m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 18.00 TO 23.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1430.6 | 1431.4 | 0.80 | 0.33 |
| 2 | 3 | 5 | 1431.4 | 1431.9 | 0.50 | 0.07 |
| 3 | 5 | 5 | 1431.9 | 1432.6 | 0.70 | 0.06 |
| 4 | 7 | 5 | 1432.6 | 1433.1 | 0.50 | 0.03 |
| 5 | 5 | 5 | 1433.1 | 1433.1 | 0.00 | 0.00 |
| 6 | 3 | 5 | 1433.1 | 1433.2 | 0.10 | 0.01 |
| 7 | 1 | 5 | 1433.2 | 1433.2 | 0.00 | 0.00 |

Table 4.30 Lugeon value test (BH-P8) Depth 23 m to 28m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 23.00 TO 28.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 1468.4 | 1472.7 | 4.30 | 1.75 |
| 2 | 3 | 5 | 1472.7 | 1473.2 | 0.50 | 0.07 |
| 3 | 5 | 5 | 1473.2 | 1474.1 | 0.90 | 0.07 |
| 4 | 7 | 5 | 1474.1 | 1474.7 | 0.60 | 0.03 |
| 5 | 10 | 5 | 1474.7 | 1477.4 | 2.70 | 0.11 |
| 6 | 7 | 5 | 1477.4 | 1477.5 | 0.10 | 0.01 |
| 7 | 5 | 5 | 1477.5 | 1477.5 | 0.00 | 0.00 |
| 8 | 3 | 5 | 1477.5 | 1477.5 | 0.00 | 0.00 |
| 9 | 1 | 5 | 1477.5 | 1477.5 | 0.00 | 0.00 |

Table 4.31 Lugeon value test (BH-P8) Depth 28 m to 33m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P8 | | | | | | |
| Location-B7 EL-188 | | | | | | |
| Depth(m) | 28.00 TO 33.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2130 | 2131.5 | 1.50 | 0.61 |
| 2 | 3 | 5 | 2131.5 | 2132.6 | 1.10 | 0.15 |
| 3 | 5 | 5 | 2132.6 | 2133.2 | 0.60 | 0.05 |
| 4 | 7 | 5 | 2133.2 | 2134.6 | 1.40 | 0.08 |
| 5 | 10 | 5 | 2134.6 | 2135.3 | 0.70 | 0.03 |
| 6 | 7 | 5 | 2135.3 | 2135.4 | 0.10 | 0.01 |
| 7 | 5 | 5 | 2135.4 | 2135.5 | 0.10 | 0.01 |
| 8 | 3 | 5 | 2135.5 | 2135.5 | 0.00 | 0.00 |
| 9 | 1 | 5 | 2135.5 | 2135.5 | 0.00 | 0.00 |

Table 4.32 Lugeon value test (BH-P15) Depth 3 m to 8m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P15 | | | | | | |
| Location-B4 EL-192 | | | | | | |
| Depth(m) | 3.00 TO 8.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2824.6 | 2832.3 | 7.70 | 3.14 |
| 2 | 3 | 5 | 2832.3 | 2837.2 | 4.90 | 0.67 |
| 3 | 5 | 5 | 2837.2 | 2841.6 | 4.40 | 0.36 |
| 4 | 3 | 5 | 2841.6 | 2841.8 | 0.20 | 0.03 |
| 5 | 1 | 5 | 2841.8 | 2841.9 | 0.10 | 0.04 |

Table 4.33 Lugeon value test (BH-P15) Depth 8 m to 13m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P15 | | | | | | |
| Location-B4 EL-192 | | | | | | |
| Depth(m ) | 8.00 TO 13.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min ) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2931.5 | 2932.5 | 1.00 | 0.41 |
| 2 | 3 | 5 | 2932.5 | 2934 | 1.50 | 0.20 |
| 3 | 5 | 5 | 2934 | 2934.8 | 0.80 | 0.06 |
| 4 | 3 | 5 | 2934.8 | 2934.9 | 0.10 | 0.01 |
| 5 | 1 | 5 | 2934.9 | 2934.9 | 0.00 | 0.00 |

Table 4.34 Lugeon value test (BH-P15) Depth 13m to 18m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P15 | | | | | | |
| Location-B4 EL-192 | | | | | | |
| Depth(m ) | 13.00 TO 18.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min ) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2945.7 | 2946.5 | 0.80 | 0.33 |
| 2 | 3 | 5 | 2946.5 | 2947.7 | 1.20 | 0.16 |
| 3 | 5 | 5 | 2947.7 | 2949.2 | 1.50 | 0.12 |
| 4 | 7 | 5 | 2949.2 | 2949.6 | 0.40 | 0.02 |
| 5 | 5 | 5 | 2949.6 | 2949.6 | 0.00 | 0.00 |
| 6 | 3 | 5 | 2949.6 | 2949.6 | 0.00 | 0.00 |
| 7 | 1 | 5 | 2949.6 | 2949.6 | 0.00 | 0.00 |

Table 4.35 Lugeon value test (BH-P15) Depth 18 m to 23m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P15 | | | | | | |
| Location-B4 EL-192 | | | | | | |
| Depth(m) | 18.00 TO 23.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/$cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2952.1 | 2952.5 | 0.40 | 0.16 |
| 2 | 3 | 5 | 2952.5 | 2953.2 | 0.70 | 0.09 |
| 3 | 5 | 5 | 2953.2 | 2962.3 | 9.10 | 0.74 |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 5 | 5 | - | - | - | - |
| 6 | 3 | 5 | - | - | - | - |
| 7 | 1 | 5 | - | - | - | - |
| Packer not fix due to soft rock and water leakage from the side of the casing. | | | | | | |

Table 4.36 Lugeon value test (BH-P15) Depth 23 m to 28m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P15 | | | | | | |
| Location-B4 EL-192 | | | | | | |
| Depth(m) | 23.00 TO 28.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/$cm^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 2941.3 | 2941.9 | 0.60 | 0.24 |
| 2 | 3 | 5 | 2941.9 | 2942.4 | 0.50 | 0.07 |
| 3 | 5 | 5 | 2942.4 | 2942.8 | 0.40 | 0.03 |
| 4 | 7 | 5 | 2942.8 | 2943.3 | 0.50 | 0.03 |
| 5 | 10 | 5 | 2943.3 | 2943.7 | 0.40 | 0.02 |
| 6 | 7 | 5 | 2943.7 | 2943.7 | 0.00 | 0.00 |
| 7 | 5 | 5 | 2943.7 | 2943.7 | 0.00 | 0.00 |
| 8 | 3 | 5 | 2943.7 | 2943.7 | 0.00 | 0.00 |
| 9 | 1 | 5 | 2943.7 | 2943.7 | 0.00 | 0.00 |

Table 4.37 Lugeon value test (BH-P69) Depth 8 m to 13m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth(m) | 8.00 TO 13.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 7582.3 | 7652.2 | 69.90 | 28.49 |
| 2 | 3 | 5 | 7652.2 | 7781 | 128.80 | 17.50 |
| 3 | 5 | 5 | 7781 | 7935.1 | 154.10 | 12.50 |
| 4 | 3 | 5 | 7935.1 | 7966.4 | 31.30 | 4.25 |
| 5 | 1 | 5 | 7966.4 | 8001.2 | 34.80 | 14.18 |

Table 4.38 Lugeon value test (BH-P69) Depth 13 m to 18m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth(m) | 13.00 TO 18.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 8040 | 8173.4 | 133.40 | 54.37 |
| 2 | 3 | 5 | 8173.4 | 8211.6 | 38.20 | 5.19 |
| 3 | 5 | 5 | 8211.6 | 8260.7 | 49.30 | 4.00 |
| 4 | 3 | 5 | 8260.7 | 8299.6 | 38.90 | 5.28 |
| 5 | 1 | 5 | 8299.6 | 8313.3 | 13.70 | 5.28 |

Table 4.39 Lugeon value test (BH-P69) Depth 18 m to 23m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth( m | 18.00 TO 23.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min ) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 8312.8 | 8356.4 | 43.60 | 17.77 |
| 2 | 3 | 5 | 8356.4 | 8389.1 | 32.70 | 4.44 |
| 3 | 5 | 5 | 8389.1 | 8441.2 | 52.10 | 4.24 |
| 4 | 7 | 5 | 8441.2 | 8483.3 | 42.10 | 2.45 |
| 5 | 5 | 5 | 8483.3 | 8502.1 | 18.80 | 1.53 |
| 6 | 3 | 5 | 8502.1 | 8532.6 | 30.50 | 4.14 |
| 7 | 1 | 5 | 8532.6 | 8561.4 | 28.80 | 11.74 |

Table 4.40 Lugeon value test (BH-P69) Depth 23 m to 28m

| | | | | | | |
|---|---|---|---|---|---|---|
| Project-2*55 MW Pare HEP | | | | | | |
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth(m ) | 23.00 TO 28.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 8571.3 | - | - | - |
| 2 | 3 | 5 | - | - | - | - |
| 3 | 5 | 5 | - | - | - | - |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 5 | 5 | - | - | - | - |

| | | | | | | |
|---|---|---|---|---|---|---|
| 6 | 3 | 5 | - | - | - | - |
| 7 | 1 | 5 | - | - | - | - |

Table 4.41 Lugeon value test (BH-P69) Depth 28 m to 33m

| Project-2*55 MW Pare HEP | | | | | | |
|---|---|---|---|---|---|---|
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth(m) | 28.00 TO 33.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | - | - | - | - |
| 2 | 3 | 5 | - | - | - | - |
| 3 | 5 | 5 | - | - | - | - |
| 4 | 7 | 5 | - | - | - | - |
| 5 | 10 | 5 | - | - | - | - |
| 6 | 7 | 5 | - | - | - | - |
| 7 | 5 | 5 | - | - | - | - |
| 8 | 3 | 5 | - | - | - | - |
| 9 | 1 | 5 | - | - | - | - |
| Packer not fixed due to increased size of hole and water continuously leakage from the hole. | | | | | | |

Table 4.42 Lugeon value test (BH-P69) Depth 33 m to 38m

| Project-2*55 MW Pare HEP | | | | | | |
|---|---|---|---|---|---|---|
| BH NO-P69 | | | | | | |
| Location-B10 EL-242 | | | | | | |
| Depth(m) | 33.00 TO 38.00 | | | | | |
| | | | | | | |
| S No | Pressure (kg/cm$^2$ ) | Time (min) | Water meter reading | | QTY Lts | Lugeon value |
| | | | From | To | | |
| 1 | 1 | 5 | 8568.2 | 8569.1 | 0.90 | 0.37 |
| 2 | 3 | 5 | 8569.1 | 8571.3 | 2.20 | 0.30 |
| 3 | 5 | 5 | 8571.3 | 8572.8 | 1.50 | 0.12 |
| 4 | 7 | 5 | 8572.8 | 8573.4 | 0.60 | 0.03 |
| 5 | 10 | 5 | 8573.4 | 8574.1 | 0.70 | 0.03 |
| 6 | 7 | 5 | 8574.1 | 8574.1 | 0.00 | 0.00 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 7 | 5 | 5 | 8574.1 | 8574.1 | 0.00 | 0.00 |
| 8 | 3 | 5 | 8574.1 | 8574.1 | 0.00 | 0.00 |
| 9 | 1 | 5 | 8574.1 | 8574.1 | 0.00 | 0.00 |

### 4.4 Description of Packer test

Packer test or Lugeon value test was performed in galary of dam in all blocks at different elevation level according to drawing of curtain grout holes. All bore holes are devided in to three categories i.e. primary holes,secondary holes and tertiary holes as per their requirements. The specing of primary holes are 6 metre and apecing of secondary holes are 3 metre and specing of tertiary holes are 1metre.

The diameter of bore hole is kept 76 mm constant throughout and depth varies from 28 m to 43m. Pre- curtain grouting packer test are performed in primary holes to know the pressure at which grouting is to be carried out and grout intake. No tests are performed in secondary holes and tertiary holes because lugeon values obtained in primary holes are very less.

To perform the packer test packer is tight at pressure of 20 to 30 kg/cm$^2$ and test performed at cycle pressure i.e (1 ,3,5,7,10) kg/cm$^2$ depended on depth at which test is to be carried out.

During entire packer test minimum Lugeon value obtained is zero and maximum Lugeon value 54.4 at block -10 bore hole no-P69 at elevation level 242.

The Lugeon value obtained during entire test are represented in the tabular form in table no 4.2 to table no 4.42.

Some results are missing because packer is not able to fixed due to increased size of bore hole and water leakage from the side of casing.

Finaly average value of Lugeon value is calculated for every 5m test section and recoded in the tabular form in table no 4.4.1 to table no 4.4.6.

The variation of average Lugeon value is also shown in the graphical form in the fig.4.1 to fig 4.6.

### 4.5 Variation of averageLugeon value with the depth at different location

Table 4.5.1 Block-03 BH-P32 EL-209

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 05-09 | 1.96 |
| 09-14 | 3.72 |
| 14-19 | 0.13 |
| 19-24 | 0.12 |
| 24-29 | 0.08 |
| 29-34 | 0.05 |
| 34-39 | 0.06 |
| 39-43 | 0.14 |

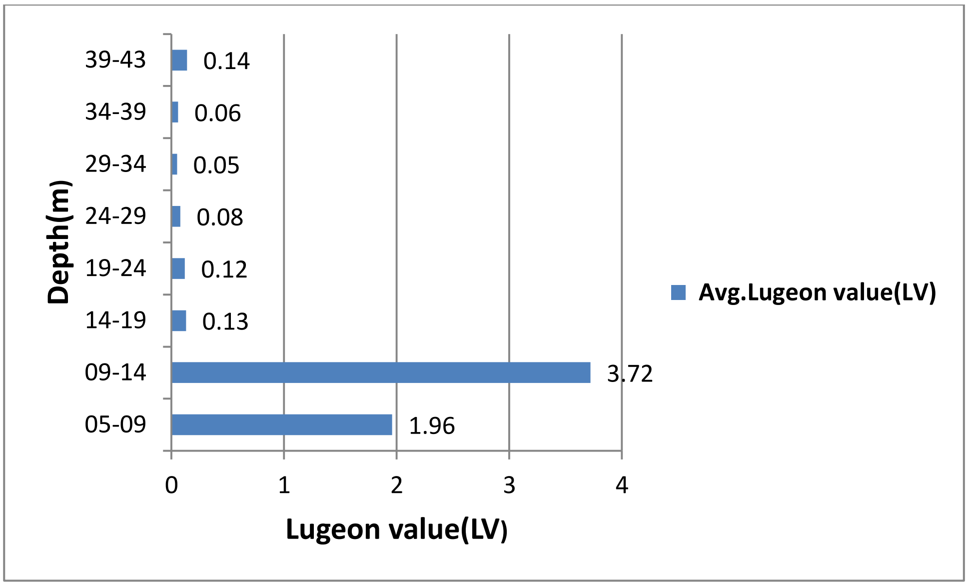



Fig 4.1 Depth Vs Lugeon value(LV) Block-03 ,BH-P32 ,EL-209

Table 4.5.2 Block-06 BH-P04 EL-188

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 03-08 | 3.09 |
| 08-13 | 1.42 |
| 13-18 | 0.13 |
| 18-23 | 0.31 |
| 23-28 | 0.13 |
| 28-33 | 0.08 |
| 33-38 | 0.14 |
| 38-43 | 0.03 |

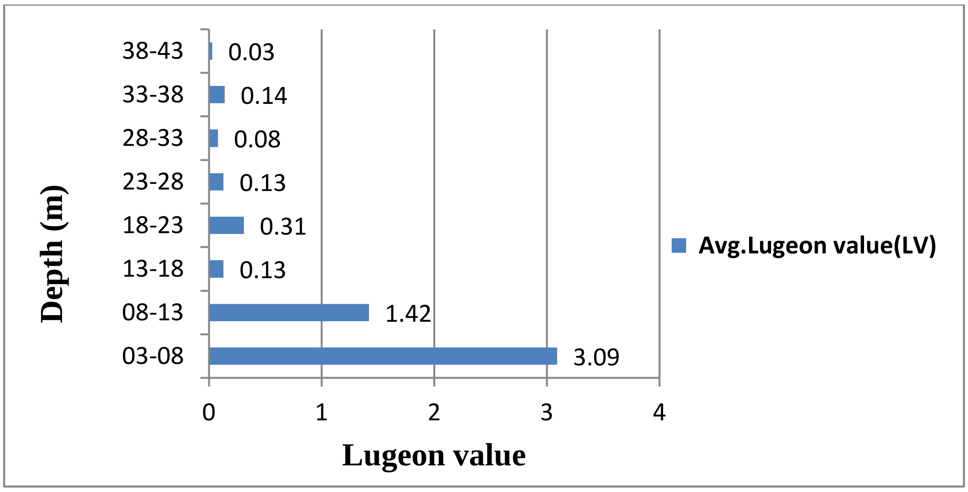



Fig 4.2 Depth vs Lugeon value(LV),Block-06 ,BH-P04,EL-188

Table 4.5.3 Block-05 BH-P13 EL-188

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 03-08 | 0.23 |
| 08-13 | 0.03 |
| 13-18 | 0.01 |
| 18-23 | 0.04 |
| 23-28 | 0.06 |
| 28-33 | 0.13 |
| 33-38 | 0.08 |
| 38-43 | 0.08 |

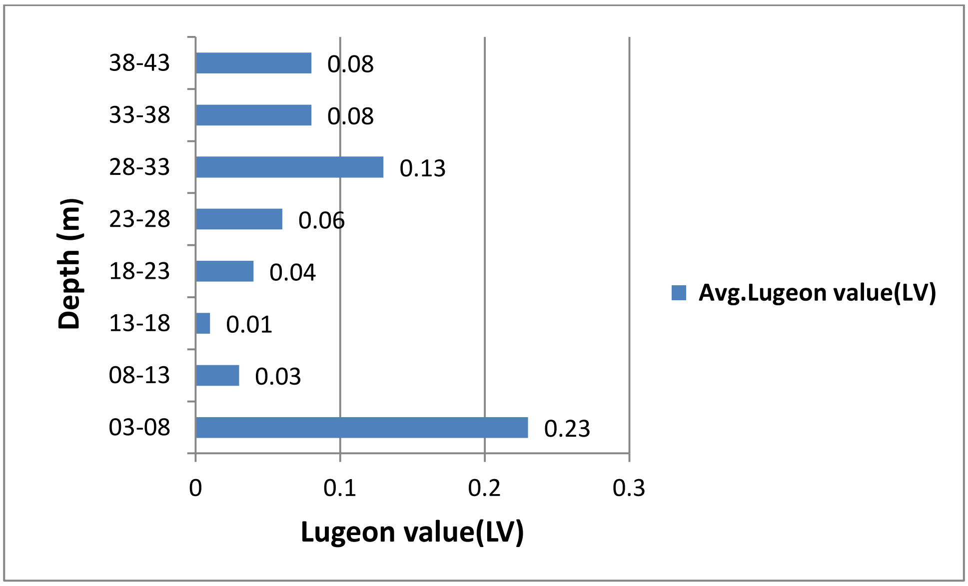



Fig 4.3 Depth vs Lugeon value(LV),Block-05,BH-P13,EL-188

Table 4.5.4 Block-07 BH-P03 EL-188

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 03-08 | 0.07 |
| 08-13 | 0.07 |
| 13-18 | 0.07 |
| 18-23 | 0.07 |
| 23-28 | 0.22 |
| 28-33 | 0.11 |

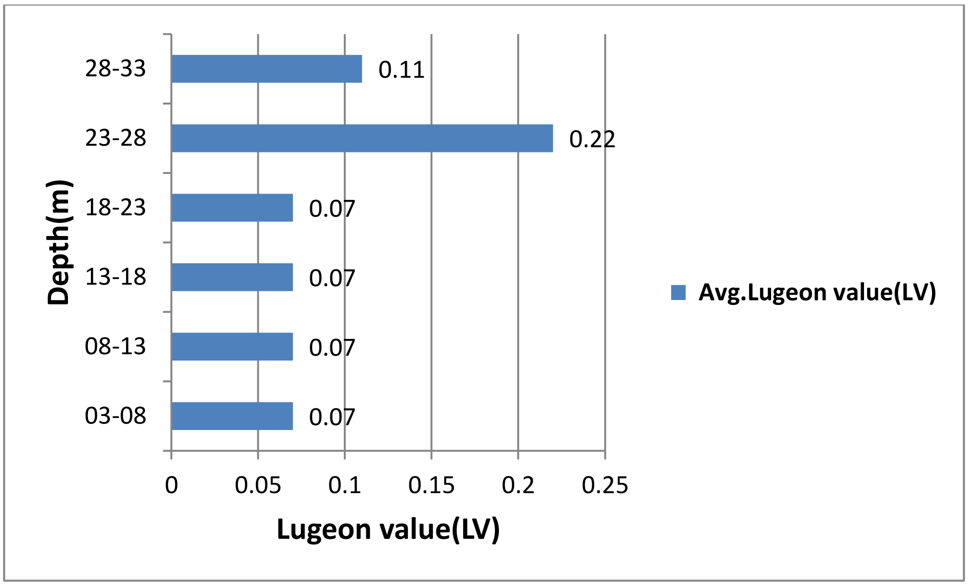



Fig 4.4 Depth vs Lugeon value(LV),Block-07,BH-P03,EL-188

Table 4.5.5 Block-04 BH-P13 EL-192

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 03-08 | 0.85 |
| 08-13 | 0.14 |
| 13-18 | 0.09 |
| 18-23 | 0.14 |
| 23-28 | 0.04 |

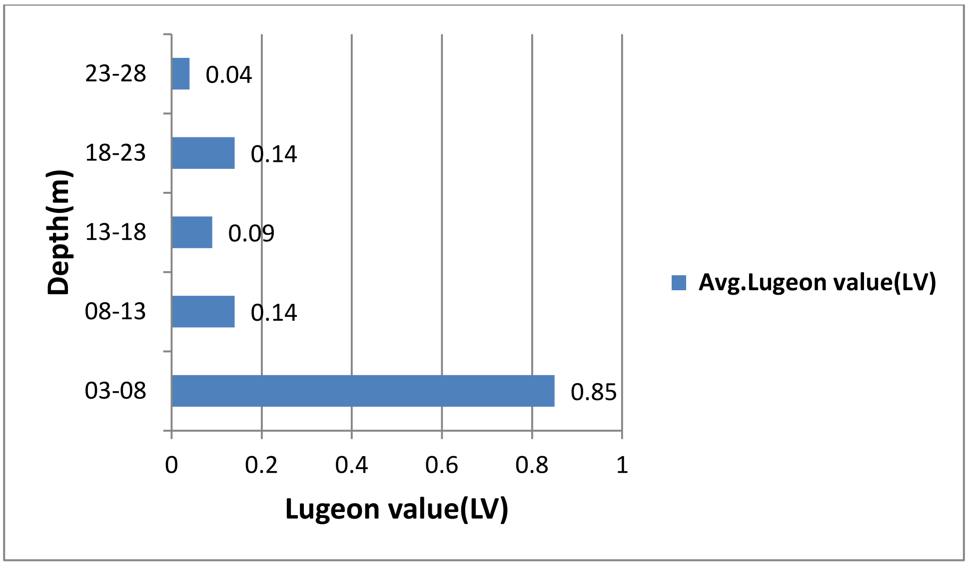



Fig 4.5 Depth vs Lugeon value(LV),Block-04,BH-P13,EL-192

Table 4.5.6 Block-10 ,BH-P63 ,EL-242

| Depth (m) | Avg.Lugeon value(LV) |
|---|---|
| 03-08 | 15.4 |
| 08-13 | 14.88 |
| 13-18 | 5.18 |

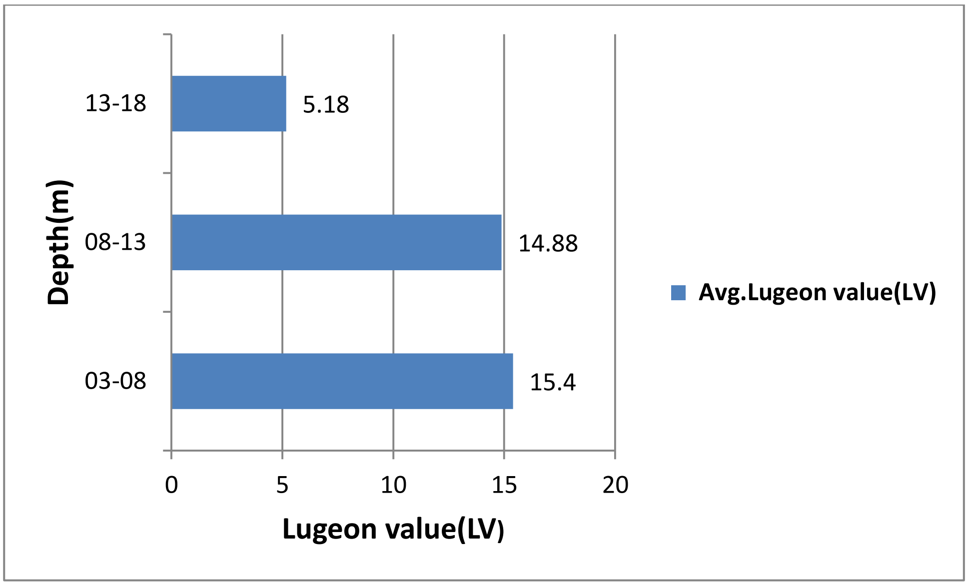



Fig 4.6 Depth vs Lugeon value(LV),Block-10,BH-P69,EL-242

## 4.6 Grouting details of different bore hole at different location in different Blocks.

Table 4.5.1 Bore hole grouting details.

| BORE HOLE GROUTING DETAIL | | | | | | | |
|---|---|---|---|---|---|---|---|
| S No | BH No | Block | Elevation | w/c | Cement QTY. | BH Plugging | Bentonite |
| 1 | P32 | B3 | EL-209 | 1:5 | 1 Bag | 3 Bag | 3kg |
| 2 | P4 | B6 | EL-188 | 1:5 | 1 Bag | 2 Bag | 2kg |
| 3 | P13 | B5 | EL-188 | 1:5 | 1.5 Bag | 2 Bag | 2kg |
| 4 | P8 | B7 | EL-188 | 1:5 | 1 Bag | 2 Bag | 2kg |
| 5 | P15 | B4 | EL-192 | 1:5 | 2.5 Bag | 2 Bag | 2kg |
| 6 | P69 | B10 | EL-242 | 1:2 | 160 Bag | 5 Bag | 5kg |

### 4.7 Description of curtain grouting

After performing the packer test in all primary holes curtain grouting is carried out for a depth equal to (2/3 of the maximum hydraulic head +8m ) measured from the rock surface subjected to a minimum of 15m.

The curtain grouting is carried out according to the results obtained of packer test. The curtain groutng is carried out in ascending order in stages not exceeding 5m.

The pressure applied during grouting varies from 1 $kg/cm^2$ to 10 $kg/cm^2$, and water cement ratio varies from 1:5 to 1:2. The cement used for grouting purpose is microfine cement and anf for plugging is ordinary port land cement. The minimum consumption of cement is 1 bag and maximum is 160 bag (B-10,BH-P69,EL-242).

After curtain grouting drainage holes are installed in downstream side of curtain grouting of diameter 76mm up to a depth equal to ¾ of the depth of grout curtain at different location having specing of 3m centre to centre.

# Chapter 5

# CONCLUSION

After all operations (i.e Drilling,Packer test and Curtain grouting) performed the following conclusion has been concluded-

(i) The single line grout curtain has been completed in all blocks.

(ii)The minimum lugeon value was found zero and maximum was 54.4.

(iii) There is no need of curtain grouting in secondary holes and tertiary holes.

(iv) Generally Lugeon values lies less than one which represent Condition of Rock Mass discontinuities are tight and very tight.

(v) The grout curtain is further supported by providing drainage hole in downstream of 76 mm diameter having spacing 3m c/c.

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