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Estimation of potential leakage and assessment of relationship between grouted values

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Abstract

Due to the tectonic movement, rock masses in Nepal Himalayas are sheared and highly fractured resulting in the loss of water from the valley side of the tunnel. This leakage plays a significant role in terms of stability, time, and cost for a successful operation throughout the lifetime. To control water-related problems like inflow and leakage treatment should be carried out to minimize the loss of water through unlined/shotcrete lined tunnels. Treatment like grouting has been widely used in the tunnel to limit the hydrogeological problems and eventually making it overtight. So, this paper focuses on estimating the potential leakage from the headrace tunnel of the Niligiri Hydro-electric project using a semi-empirical approach of Panthi (2006) and maximum grout take estimation has been carried out in single joint.

Keywords

leakage, grout, lugeon value, penetration length

1. Introduction

Due to tectonic movement, rock mass in the Himalayas are highly fractured and deeply weathered which might result in inflow and leakage out of the tunnel and the loss of water from fractured rock mass through unlined/shortcrete tunnel is is very common problem in the higher Himalaya [1]. Leakage in the tunnel is mainly governed by the hydraulic conductivity of rock mass and the transmissivity which can be calculated from the lugeon test value. Higher the lugeon value possibility of leakage out of the tunnel is high. Also while excavating the tunnel there is a high risk of encountering unwanted geological properties like crushed zone, shear zone, and unfavorable joints with respect to the tunnel axis which has the likelihood of leakage from respective chainage or section. So after the excavation early filling test should be carried out to locate the excessive leakage section and emphasis should be given to unwanted geological properties during grouting. In this article, Panthi (2006) proposed a semi-empirical approach has been used to estimate the potential leakage from the water tunnels in Nilgiri-II HEP.

2. Panthi 2006 Approach

Shotcrete-lined tunnels are semi-permeable so only shotcrete lined a tunnel is not able to withstand the force of leakage in a high-pressure tunnel. This approach has been derived through the study pg comprehensive data of Q-value at Khimti head race tunnel hand has been applied to study leakage potential in Khimti itself and Upper Tamakoshi Hydro Electric project. A semi-empirical relationship between specific leakage (q) and the parameter of rock quality index was established which is represented in the following equation:

$$q = fa \times H \times (Jn \times Jr)/Ja \tag{1}$$

Where fa is a joint permeability factor with unit l/min/m2. This factor is related to the permeability condition of joints in the rock mass and varies from 0.001 to 0.25. H is the static water head, and Jn, Jr, and Ja are some Q-value parameters represented by joint set number, joint roughness number, and joint alteration number, respectively. From equation 1 we can conclude that all inputs are directly proportional to specific leakage except joint alteration (Ja) which tends to increase or decrease the leakage with respect to infilling value. Panthi (2010) further suggested that the joint permeability factor (fa) can be quantified using Equation 2, which is related to joint spacing (Js), joint persistence (Jp), and the shortest perpendicular distance (D) from the rock slope topography to valley side tunnel roof:

$$fa = Jp/(D \times Js) \tag{2}$$

Equations 1 and 2 will be used to estimate the quantity of leakage from the tunnel in each section of the Nilgiri Hydroelectric project [2].

3. Nilgiri Hydro-Electric Project

Nilgiri Khola Hydropower is located in Annapurna Rural Municipality-4, Myagdi district. At about 2400m elevation from sea level. Physiographically it is situated in the Higher Himalaya region about 110 KM due north-west of Pokhara Valley, see Figure 1. It originates from the Nilgiri mountain of height 7061 m. Nilgiri Khola Hydropower has a total capacity of 117 MW and is divided into Nilgiri I and Nilgiri II which has the extent of 41 MW and 76 MW respectively but Nilgiri II is a cascade hydropower system. Scheme Type of Nilgiri-II is RoR (Cascade development) having a gross head of 789.75 m and a mean annual discharge of 17.15 m^3/s. One intake has been installed at the side with a number of openings one of which has the size of 1.5 m x 3 m. Desanding basin is an intermittent flushing system with a single bay and double hooper to trap

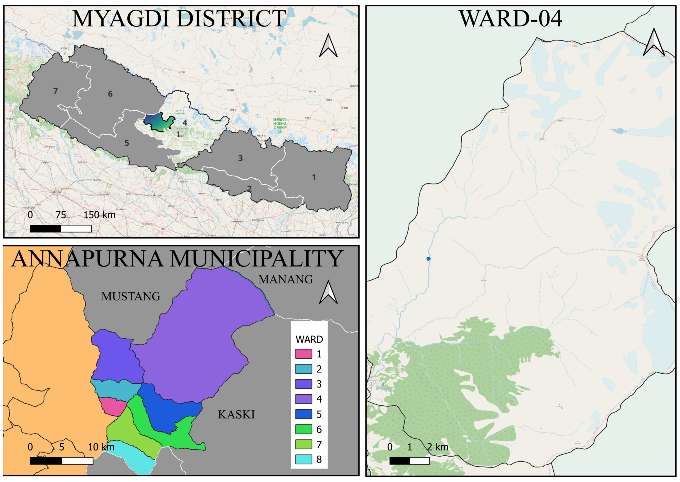

Figure 1: Location of Nilgiri HEP-II marked at ward-04

the silt and sediments flowing from the intake. The HRT is approximately 4.25 kilometers long with an inverted D-shape and a 10.5 square meters cross-section. A surge tank of height 36.25 m was built to avoid excessive water hammer pressure and five no.s of Adit tunnel were excavated of cross-section 10.5 square meters to facilitate the tunnel and shaft construction.

4. Project Geology

Nilgiri-II is located at the higher Himalayas region near the Main central Thrust (MCT). Rock mass in this region is mainly characterized by Jurassic metamorphic and sedimentary rocks consisting coarse coarse-grained mica gneiss with garnet, kyanite or silimanite, migatite, and quartzite around Nilgiri I. But from the tunnel face mapping main rock characterized is banded gneiss along the headrace tunnel. The rock mass at the project area has a foliation joints and two distinct cross joint and majority with random joints.

5. Rockmass Distribution

Tunnel excavation requires enough ground investigation, cost-effectiveness, and consideration of various constraints in a changing ground condition. In the case of Nilgiri-I and Nilgiri-II excavation was carried out with maximum use of the self-supporting capacity of rock mass except in some cases steel ribs were provided in a few sections. The Pre-planning phase investigation and predictions of the rock mass conditions along the tunnel in the Nilgiri Project was rather poor and planning involving the rock mass along the headrace tunnel would be of good quality and no measures such as pre-grouting control were considered at the initial phase of tunneling and the alignment was fixed assuming that the higher Himalaya rock would be of good quality. The figure below represents te predicted rock mass before excavation and the actual rock mass condition after excavation.

6. Specific Leakage

Tunnel construction in the Himalayas is not an easy task to carry out, not only during the excavation it involves risk but

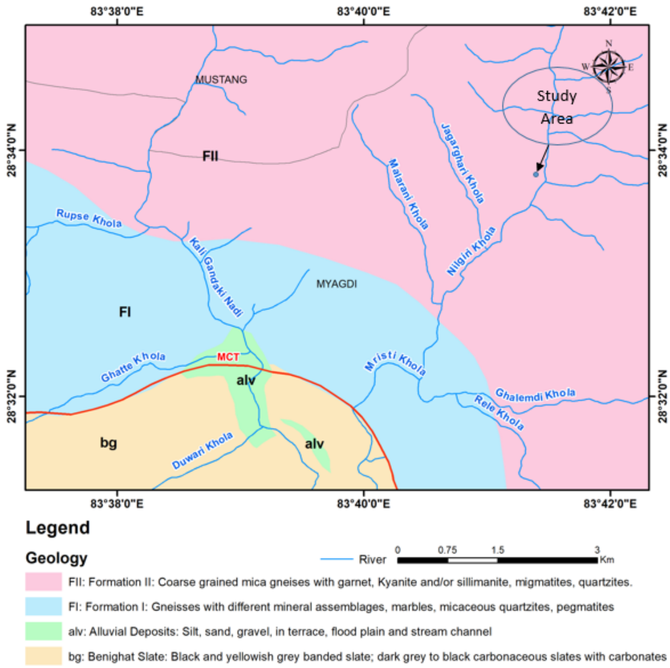

Figure 2: Geological Map of study area(Source: Department of mines and geology)

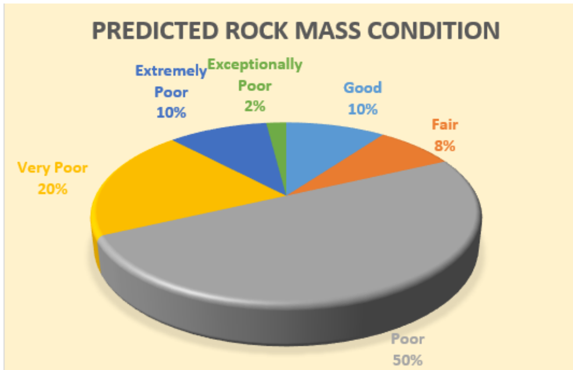

Figure 3: Predicted rock mass condition along HRT

after the breakthrough also. Stress-related uncertainities as well as water ingress and water leakage may cause the stability problem if not identified and treated. This topic will focus on the water leakage along the head race tunnel of 4.2 Km. Most of the tunnels in Himalayas for the hydropower are shotcrete lined which cannot seal the water to flow without leakage thus ground improvement techniques like pre-injection and post-injection grouting is needed to seal the rock mass. Specific leakage is quantified using the Panthi approach (2006, 2010) as described in the literature review. This theory was successfully used in the Upper Tamakoshi Project where the head of the project is almost similar to ours. Many researchers have provided various theories about the limit of leakage through shotcrete-lined tunnels in the Himalayas. Merritt (1999) recommends a specific leakage of 0.3 l/min/m tunnel for the unlined or shotcrete lined water tunnels, which is practically not feasible to achieve in shotcrete lined pressure tunnels constructed through the Himalayan rock mass. The cost related to leakage control will turn out to be much higher than a pressure tunnel fully lined with concrete or steel. Panthi (2006) recommends a leakage limit of a maximum of

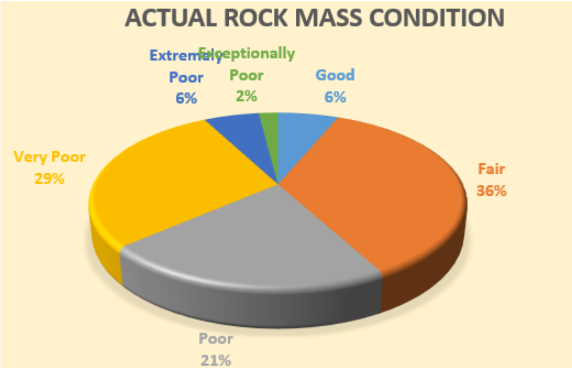

Figure 4: Actual rock mass condition along HRT

up to 1.5 l/min/m tunnel, which is achievable through ground improvements and is cost-effective as well.

During the analysis, it was assumed that during the dry season natural ground water table (GWT) lowers down to a minimum almost to the level of HRT and the water flowing in the HRT will function as the maximum ground water table which will produce hydrostatic head and will govern the extent of water leakage that may occur from the headrace tunnel out to the topographic surface. Leakage, assessment is carried out considering only dry season because, during the monsoon season, the water table reaches almost to the topographical surface. Therefore, the risk of potential water leakage from the headrace tunnel of Nilgiri-II will mainly be during the dry season of the year. (Fluid flow and leakage assessment through unlined/shortcrete lined tunnel: K.K Panthi and Chatra B.Basnet, 2021).

From the figure below we can conclude that among all the chainage, downstream chainage is the most vulnerable i.e. 3+300 m to 3+683 m. This section has a thin topographical cover and was marked as a critical section by the project previously. The concrete lining will be provided from 2+900 m to the end of the tunnel due to high leakage possibility.

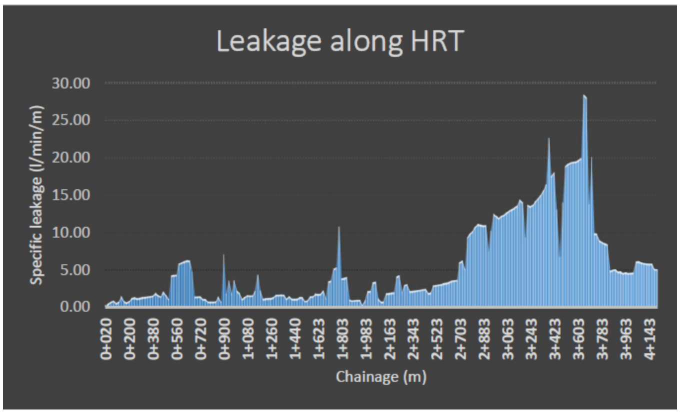

Figure 5: Potential leakage along HRT

7. Overall leakage Scenario

Mapped rock mass quality from the tunnel face map is used to calculate and demonstrate the overall scenario of different rock mass parameters number of joints (Jn), joint roughness(Jr), Joint alteration (Ja), Joint persistence (Jp), and Joint spacing (Js) with shortest distance to topographical cover (D) and permeability factor (fa) for six rock mass quality. As we move forward from inlet to outlet, the hydrostatic head increases, and the topographic covers decrease which is critical at 3+300 to 3+600 m. The maximum head is 35.8 m and the minimum is 0.251 at the beginning of the tunnel. The thickest topographical cover is 305.1 m and the thin is 75 m leakage was carried out at 20 m intervals throughout the tunnel. The average specific leakage through the tunnel having good quality rock mass is 4.14 l/min/m tunnel, which exceeds the acceptable limit. On the other hand, the average specific leakage from the headrace tunnel segments with fair and poor rock mass is 7.16 l/min/m and 5.19 l/min/m respectively.

8. Material and Methods for grouting

8.1 Water Pressure Test (WPT)

Water Pressure Test is the most common method to determine the permeability or opening of the joints in the rock mass which is conducted in six steps. At first low pressure is applied and pressure is increased gradually up to the third step having the maximum pressure value. When the pressure is reached to the maximum value pressure applied is decreased to medium and eventually to low again. This test is carried out for 10 minutes for each pressure step and then the lugeon value is calculated as follows:

$$Lu = 10Q/PL \tag{3}$$

Where Q is flow (ltr/min), L is length (m) of the test section and P is the pump pressure (bar). The obtained lugeon value is plotted in the graph to determine the pattern of laminar, turbulent, dilation, void filling, and wash-out flow, and lugeon is ascertained based on these flows.

8.2 Hydraulic Aperture

Hydraulic conductivity is calculated from the lugeon value following the transmissivity in the joints of the rock mass. When the lugeon value is multiplied by 1.6 × 10⁻7 and transmissivity is the product of hydraulic conductivity and the length of the borehole.[3]When these two parameters are determined hydraulic aperture is calculated by knowing Transmissivity (m^2/s), μ is dynamic viscosity (Pa.s), ρ is the density of water (Kg/m^3) as:

$$b_{hyD} = (T \times 12 \times \mu/\rho \times g)^{0.33} \tag{4}$$

8.3 Penetration Length

Maximum penetration length on the grout with ΔP, hydraulic aperture b_{hyd} and yield stress τ can be calculated based on Gustafson and Stille (1996) using following equation:[3]

$$I_{max} = \Delta P \times b_{hyd}/2 \times \tau \tag{5}$$

8.4 Volume of grout

Hassler et al. (1992), according to the property of the Bingham fluid, proposed the following equation for calculating the grout paste volume in one joint using:[3]

$$V_g = (I_{max})^2 \times \pi \times b_{hyd} \tag{6}$$

8.5 Statistical Analysis and Result

In this paper relationship between maximum grout volume and aperture was analyzed using different constant pump pressures for the 43 sections. If we see the following results for the pressure of 4, 5, and 6 pressure bar, the relation is linear and positive showing that there is a meaningful relationship between these two variables.

Table 1: Maximum grout volume and aperture in different constant pump pressures

$b_{hyd}(\mu m)$	V_{max}-4 bar	V_{max}-5bar	V_{max}-6 bar
121.67	5.28	36.71	96.42
99.18	1.3	15.5	45.05
255.030	21.95	259.09	756.65
253.48	21.55	254.40	742.95
200.2	22.53	160.9	425.46
291.79	31.33	382.68	1124.08
237.28	37.02	266.52	705.78
161.38	5.39	65.08	190.75
293.83	31.40	388.73	1144.34
275.96	24.59	316.96	939.28
217.98	20.62	183.82	509.64
178.67	1.66	63.70	215.30
184.00	6.83	92.30	275.58
310.02	72.05	565.60	1526.96
207.48	9.99	133.05	396.33
178.67	13.55	107.59	291.19
200.25	14.91	139.30	389.76
132.50	2.32	33.62	101.44
183.25	13.97	114.26	311.17
99.18	0.94	13.99	42.33
141.81	2.41	39.52	121.40
121.67	4.10	33.49	91.16
121.67	1.76	25.88	78.29
118.19	1.59	23.64	71.62
132.50	2.18	33.077	100.50
141.81	6.014	51.61	141.98
190.42	5.72	95.15	292.99
186.623	5.22	88.90	274.63
170.46	9.57	87.05	242.31
238.16	10.77	184.45	570.24
189.73	5.12	91.87	285.85
179.45	11.06	101.23	282.14
191.11	4.98	92.84	290.28
87.087	1.19	11.36	31.89
384.60	38.05	745.37	2345.91
292.96	16.25	326.92	1032.35
181.75	3.81	77.77	246.08
208.05	15.33	151.8	429.83
199	4.83	101.30	321.50
219.03	17.32	175.42	498.4
366.33	28.61	624.71	1992.64
327.96	20.16	446.54	1426.76
197.088	12.22	126.52	361.02

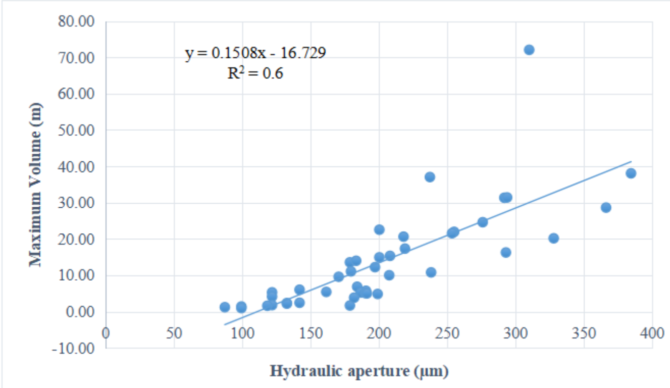

Figure 6: Relation between hydraulic aperture and maximum volume at 4 bar

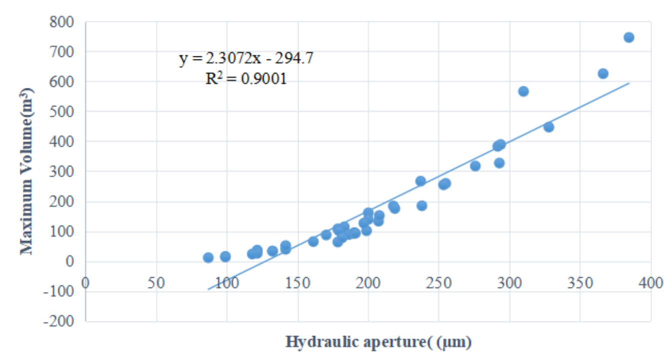

Figure 7: Relation between hydraulic aperture and maximum volume at 5 bar

The variables in Table 1 have been calculated based on the method mentioned earlier in the section. The above calculated maximum volume in different pressure bars is the value for one joint in a 2-D case and volume is calculated in liters. When these variables are plotted then we obtained a coefficient of determination R^2=0.6, R^2=0.90, and R^2=0.89 for the pressure bar of 4 5, and 6 respectively. Also when we plot the scatter diagram, analysis shows the result that the hydraulic aperture has also a meaningful relation to the actual and maximum penetration length of the grout in the joints. Maximum penetration length is the length that requires infinite time to reach the full penetration length.[4] So, while conducting grouting, the time variable must be defined to calculate the extent of the grout penetration which gives the actual penetration in respective time. This method is known as Real-time Grouting Control (RTGC) which is widely used in Sweden for the preliminary grout design to make the treatment safe and economical.

Quantified data was analyzed progressively and plotted on a scatter diagram between hydraulic aperture and maximum volume resulting in the equation of best fit as y = 0.1508x - 16.729 with coefficient of determination (R^2) = 0.6. Similarly, for the constant 5 bar and 6 bar pressure, the scatter diagram between hydraulic aperture and maximum volume results in the equation of best fit as y = 2.3072x - 294.7 with coefficient of determination (R^2) = 0.9 and y = 7.0629x - 294.7 with coefficient of determination (R^2) = 0.8993 . It is a strong indicator of the high degree of fit in regression analysis. It is highly efficient at describing and estimating the value of the

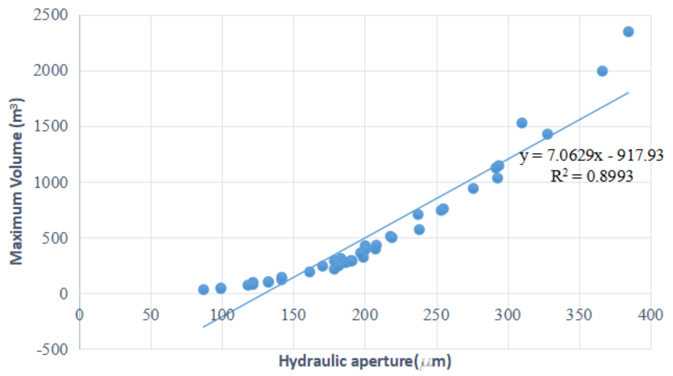

Figure 8: Relation between hydraulic aperture and maximum volume at 6 bar

dependent variable. Practically, independent variables in our model provide strong clarification for the observed variations in the dependent variable. Additionally, when we have new data with values of independent variables, we can expect the model to estimate the dependent variable with a high degree of accuracy.

Result and Conclusion

According to Panthi (2006), among the most important aspects of the unlined or shotcrete-lined water tunnel concept is control of water leakage while in operation at full hydrostatic pressure limiting the leakage to an acceptable limit. The leakage limit for unlined or shotcrete-lined water tunnels may be defined maximum of up to 1.5 liters per minute per meter tunnel. However, the main difficulty in leakage assessment is the quantification of possible water leakage prior to and during tunnel excavation.[2] While assessing the HRT of 4.2 Km, it was found, that leakage was high at downstream of the tunnel. Upstream of the tunnel shows leakage potential is quite low, as upstream of the tunnel has thick lateral cover compared to downstream. The most vulnerable section in HRT is 3+383 m, 3+643 m, 3+663 m, and 3+703 m having specific leakage of 22.64 l/min/m, 28.46 l/min/m, 28.08 l/min/m, and 20.13 l/min/m with permeability factor 0.05, 0.07, 0.07 and 0.06 l/min/m^2 respectively. Similarly, an overall leakage assessment was carried out for a specific rock mass class along HRT. Panthi (2006) recommends a leakage limit of a maximum of up to 1.5 l/min/m tunnel, which is achievable through ground improvements and is cost-effective as well. The average specific leakage for the good rock mass quality was found to be 4.14 l/min/m which exceeds the acceptable limit of our purpose and the maximum leakage for this rock is 10.68 l/min/m at chainage 1+783 m. Similarly, average specific leakage for other rock mass classes has also exceeded the acceptable limit of 1.4 l/min/m having 28.5 l/min/m as the maximum leakage for poor rock mass class. The study reveals that there is a need for effective water leakage treatment like consolidation grouting in the downstream or concrete lining to limit the leakage from the tunnel. In 4.2 Km HRT fair rock constitutes of highest percentage of 36 %, very poor rock mass is about 29 %, and poor rock mass of 21%. Similarly, extremely poor rock mass and good rock mass are of equal amount i.e. 6%. 2% of exceptionally poor rock mass was identified along HRT. These rock mass classes are distributed for the gneiss rock which is typically found in higher Himalayan region.

A scatter diagram was plotted to identify the relationship between hydraulic aperture, penetration length, and maximum volume at a constant pressure. The line of best fit between hydraulic aperture and penetration length was y= 0.0154 x+ 0.916 with coefficient of determination(R^2) = 0.50 and between hydraulic aperture and maximum volume y = 0.1508x+16.72 and coefficient of determination (R^2) = 0.6 at a constant pressure of 4 bar. The line of best fit between hydraulic aperture and penetration length was y= 0.0654 x+ 0.916 with coefficient of determination(R^2) =0.94 and between hydraulic aperture and maximum volume y = 2.3072 x+ 0.916 and coefficient of determination (R^2) = 0.9 at a constant pressure of 5 bar. Similarly, the line of best fit between hydraulic aperture and penetration length was y= 0.1154 x+ 0.916 with coefficient of determination(R^2) = 0.98 and between hydraulic aperture and maximum volume y = 7.0629 x+ 917.93 and coefficient of determination (R^2) = 0.89 at a constant pressure of 6 bar.

A water Pressure test was conducted between chainage 3+350 m and 3+360 m, and the experiment manifested the result that the chainage 3+600 m has the lowest lugeon value i.e. of 0.088 and the chainage 3+610 m and 3+650 m has the highest lugeon value i.e. 7.58 and 6.55 respectively among 43 sections where the grouting penetration assessment was conducted for the injection pressure bars 4, 5, and 6. Hydraulic conductivity and transmissivity are dependent on the lugeon value and value increases or decreases with the rise and fall of the lugeon value. Hydraulic aperture is calculated from the cubic law; the more opened this parameter more easily grout will flow. ,

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References

[1] Krishna Kanta Panthi. *Analysis of Engineering Geological Uncertainties Related to Tunneling in Himalaya Rockmass Condition*, 2006.

[2] K.K. Panthi and C.B. Basnet. *Leakage potential through shotcrete lined high-pressure tunnel- An analysis on a case from Nepal.* Rock Mechanics for Natural Resources and Infrastructure Development –Fontoura, Rocca & Pavon Mendoza (Eds)© 2020 ISRM, ISBN 978-0-367-42284-4, 2020.

[3] Cecilia Eliasson. Intensities as tools in grouting evaluations using data from the north link and stockholm city line. 2012.

[4] Håkan Stille and Kungliga Tekniska Högskolan. Real time grouting control method. 2008.