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Effectiveness of Grouting at Hydropower Tunnel: A case Study of Headrace Tunnel at Super Dordi Hydropower Project, Nepal

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Abstract
With the growth of numerous hydroelectric projects in Nepal, tunneling activities have significantly grown. Large volumes of water need to be transported from headworks to powerhouses in medium and large hydropower plants. Many tunnel design decisions have a direct impact on the overall cost and time needed, including the choice of the tunnel alignment and estimation of rock mass quality and rock support need. Many of the important choices that must be made during the planning, design, and construction of a tunnel are heavily influenced by the geology along the tunnel line. Mainly with the headrace tunnel water inflow and leakage problem induces the instability along with sometimes the losses of the valuable water which makes the huge economical losses and create the safety issues. This paper mainly focuses on the leakage assessment and control measure which helps to identify possible water leakage from unlined or shotcrete lined tunnels and the method of control for it. Different methods of leakage control and effectiveness are also discussed. Effectiveness of post injection grouting in the headrace tunnel of Superdordi Hydropower project taken and concludes the role of grouting as control measure of leakage in tunnel.

Keywords
Leakage, inflow, estimation, effectiveness, and grouting.

1. Introduction

The Himalayan rock masses are typically severely fractured and extremely weathered. This requirement necessitates installing a lot of temporary rock support during tunnel excavation, though the Himalayas also employ full concrete lining after excavation is complete. As a result, excavating tunnels through the Himalayan rock mass is increasingly expensive, time-consuming, and occasionally economically undesirable for hydropower projects. Preinjection grouting is one of the greatest ways to reduce water leakage in water conveyance tunnels, and using temporary support as permanent support is one of the finest ways to handle this problem. Nepal is a mountainous country which has a width ranging from 150 to 250km north-south extending along 890km along east-west. In such limited area, it has the varying altitude 60m to 88848m from sea level. The abundant water flowing from the Himalayas, and the potential it carries along with total hydropower capacity of 83,280MW. Makes hydropower as one of the best possibly to develop nation. Development of hydropower using underground space is increasing day by day. The research and innovation in rock and tunnel engineering makes the hydropower production ease.

Normally, the rock mass is jointed aquifers and leakage/inflow from discontinuities of the permeable joints. Generally, the rock masses near the surface have highly jointed and loose joints with more weathered whereas moving in depth from surface joints are found to be tight and rarely jointed. From the visual inspection performed in many ungrouted tunnels maximum water leakage normally occurs near the surface [1].

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2. Objectives

The main objective of the current study is to analyze and demonstrate the Leakage assessment and Control measures at Headrace Tunnel of Super Dordi Hydropower Project-Kha. The specific objectives of the study are:

1. To carry out water leakage assessment using different approach
2. To evaluate the necessity of leakage control measures
3. Performance evaluation of grouting

3. Methodology

3.1 Methods of Water Inflow Measurement

3.1.1 Discharge vessels

Discharge vessels as shown in Figure 1, are important for the field measurement of leakage in the tunnel. Additionally helpful for springs with a medium-high or fluctuating inflow. We make use of the relationship between water level and the rate of leakage from the vessel's hole

Figure 1: Discharge vessels

The specific leakage (q) is expressed as:

$$q = v/(l \times t) \tag{1}$$

3.1.2 Tokheim and Janbu 1984

According to Tokheim & Janbu 1984, the following equation describes the water flow into a tunnel:

$$Q_w = \frac{2\pi \times K \times l \times P}{\mu_w \times G} \tag{2}$$

Where,
Q_w = inflow rate (m3/s)
K = specific permeability (m2)
L = length of tunnel/cavern (m)
P = potential (active head) (Pa)
μ_w = dynamic viscosity of water (kg/(m.s))
G = geometry factor

G depicts the flow pattern relative to the tunnel geometry which is calculated as::

$$G = \ln \frac{(2D - r)(L + 2r)}{r[L + 2(2D - r)]} \tag{3}$$

Where,
D = distance between the groundwater table and the length axis of the excavation
r = equivalent radius

The water inflow equation indicates that knowledge of permeability is necessary in order to implement the Tokheim and Lanbu 1984 approach. Therefore, permeability testing in boreholes must be done to provide accurate input. The main uncertainty when applying this method is generally represented by the geometry factor (G). The equations presented by Tokheim & Janbu were designed originally for a three-dimensional flow pattern. In rock mass, however, the water flow is frequently totally controlled by a single joint set. When assessing the geometry component G, a $L >> r$ should always be utilized to account for any potential anisotropy-related errors.

3.1.3 Panthi 2006 Approach (Empirical Approach)

Exploratory drilled holes as shown in Figure 3.4 with certain angle for grout holes have been taken into the study with the effect of static line in leakage. An unlined or shotcrete-lined tunnel's hydrostatic head (hstatic), degree of jointing, and discontinuity characteristics of the rock mass are the main factors regulating leakage [2]. Two of the most important components of the unlined or shotcrete-lined water tunnel concept are the management of water leakage when in operation at full hydrostatic pressure and the control of the leakage to an acceptable limit, according to [2]. For shotcrete-lined or unlined water tunnels, leakage limit may be set at a maximum of 1.5 l/min/m. However, estimating potential water loss before and during tunnel excavation is the main challenge in leakage assessment.[2] used thorough specific Q-value param and methodical water leakage testing conducted prior to the HRT excavation of the Khimti I HPP in Nepal to overcome this issue. The equation represents the semiempirical relationship between the param of rock quality index (Q) and particular tunnel leakage (qt) as:

$$q_t = \frac{f_a \times H \times (J_n \times J_r}{J_a} \tag{4}$$

Where, f_a is a permeability factor of joint (l/min/m2). This factor, value ranges from 0.001 to 0.25, is connected to the permeability state of joints in the rock mass. Joint alteration, joint roughness number, and Joint set number, respectively which are some Q-value characteristics denoted by J_a, J_r, and J_n. H is the static water head. Except for Joint Alteration Number (J_a), leads to minimize leakage as numerical value increases, all input factors in Equation 3-5 raise the leakage potential. Additionally, according to [3], the joint permeability factor (f_a), which is associated with joint spacing side tunnel roof, may be computed using joint spacing (J_s), joint persistence (J_p), and the topography of the rock slope with the shortest perpendicular distance (D) to the valley (Figure 2).

$$f_a = \frac{J_p}{D \times J_s} \tag{5}$$

In Panthi [2] jointing degree and the state of various joint sets, which are indicated by the joint infilling conditions and aperture, are the key determinants of the permeability condition of rock mass. Amount of leakage from the water tunnels will also be governed by the topographic surface distance, the joint persistence, the spacing of the worst joint set, and the hydrostatic water pressure that arises in the rock mass domain. Equations 4 & 5 described by Panthi (2006 and 2010) are utilized to assess the possibility of water leakage from the HRT of the UTHP in Nepal.

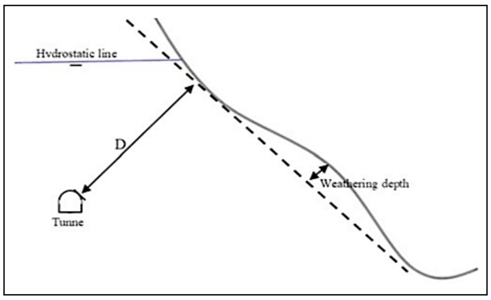

Figure 2: D explained by a typical topographic configuration [3]

3.2 Grouting Methods and evaluation

Numerous times, there have been major water input and leakage issues that have jeopardized project safety and resulted in significant financial losses. These issues also resulted in a reduction in the stability of the tunnel's surrounding rock formation. The actual problem, however, is a precise estimation of potential water loss before tunnel excavation, hence financial repercussions are considered properly in advance. water-related issues like unsafe and challenging tunneling conditions, a slow rate of progress, difficult blasting, instability brought on by water pressure, erosion, and swelling, a drop in water table, surface wells draining, subsidence, and foundation damage, inability to bond shotcrete to wet rock, and liner deterioration and corrosion of reinforcement steel by salt water in undersea tunnels. The performance of the Pre injection and Post injection grouting procedure is being assessed.

Pre-injection grouting

To determine if injection grouting was necessary or not, two basic criteria were used. These were (a) if hydrostatic pressure was less than 1.5 times the groundwater pressure entering the tunnel when it was in use, and (b) if the leakage through the rock mass surpassed a predetermined threshold (water loss after pumping the water through exploratory drill hole with 1.5 times hydrostatic pressure). Criterion (b) was heavily used for determining the pre-injection grouting at the headrace tunnel since, in general, criterion (a) is only valid in locations possible points of groundwater ingress during tunnel construction.

Post injection grouting

It is crucial to locate the regions where the greatest risk of leakage exists before performing the post-injection grouting performance on relevant parts of tunnel only. Early testing of the water filling in the HRT was done to confirm that.

Lugeon Test

The Lugeon test, sometimes known as the packet test on occasion, is a popular in-situ testing technique for determining the typical hydraulic conductivity of rock mass. In-situ testing of formation permeability is done by measuring the amount of water that is taken from a test hole segment when the interval is pressured at a specific pressure (10bars). It is primarily employed in variable permeability formations that are being evaluated for fracture.

4. Result and Discussion

The Super Dordi Hydropower Project site is located in Dordi Gaun Palika of Lamjung, Nepal as shown in Figure 4.1. Geographically, the project lies between the following boundary which is in Survey License provided by DOED.

Longitudes 84034’15” E and 84031’00” E

Latitudes 28018’43” N and 28016’20” N

The project area is in the physiographic zone of middle and higher mountains, with upper Dordi Khola catchments reaching an elevation of 7893 m above mean sea level.

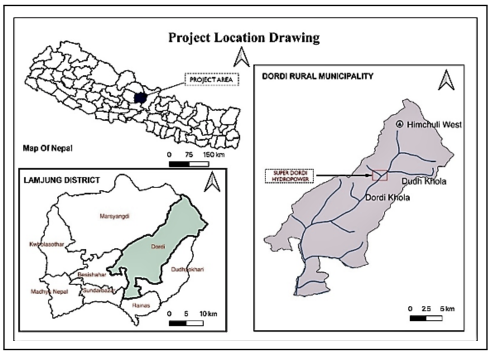

Figure 3: location map of SDHHP)

4.1 Geological Condition of Project

The project located at Higher Himalayan region mainly consists of banded gneiss with a number of fractures and weakness zone, Number of quartz veins and schist was identified in the project profile. Figure 4 displays the geological map of the project profile.

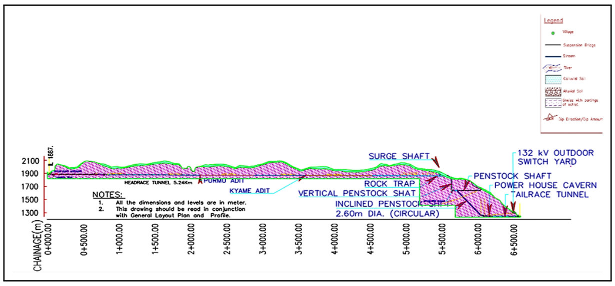

Figure 4: Geological map of SDHHP HRT)

4.2 Rock Mass Quality and Leakage Along HRT

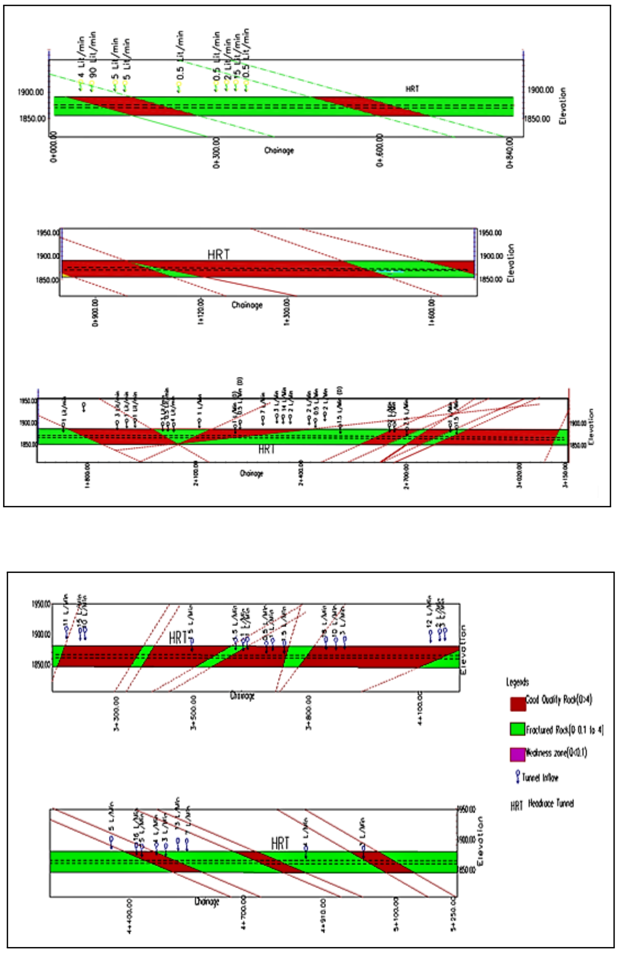

Figure 5: Rock Mass Quality along the HRT of SDHPP)

Figure 5 shows the overall rock mass quality along the HRT. The red color shows the good quality rocks ($Q > 4$), the green color shows the fractured rock (Q 0.1 to 4) and the pink color shows the weak zone ($Q < 0.1$) [4]. The amount of leakage at various chainage are shown along the headrace tunnel alignment. The critical sections having maximum leakage mainly focused on the interest of the study.

The maximum amount of leakage up to 90l/min occurs at the chainage 0+050m chainage is shown. Based on rock mass quality, shortest surface distance and maximum leakage the critical sections were selected.

4.3 Rock Mass Quality of the Critical Section of HRT

Table 1: Rock mass quality of the critical section

Chainage m	RQD	Q	Rock Mass Class
1+675	55	3.025	IV
2+015	60	7.5	III
2+157	70	21	II
2+878	70	7	III
3+105	70	11.55	II
4+106	60	6	III
4+411	60	6	III
4+516	65	16.52	II
4+682	40	1.32	IV

Table 1 shows the rock mass classification along the critical section. The rock mass class varies from rock class II to rock class IV was mainly observed. The value of Q was calculated using the Q system of rock mass classification relied on various rock mass parameter like joint spacing, joint discontinuities, roughness, water content, stress reduction factor etc.

4.4 Panthi 2006 Approach Result and Calculation

Table 2: Leakage along critical chainage of HRT by Panthi 2006 approach

Chainage m	fa l/min/m2	qt l/min/m
1+675	0.102	11.28
2+015	0.067	5.02
2+157	0.092	9.32
2+878	0.077	13.71
3+105	0.053	10.15
4+106	0.059	11.81
4+411	0.078	16.16
4+516	0.061	13.01
4+682	0.058	6.41

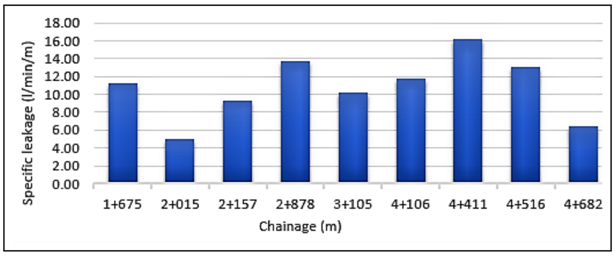

Figure 6: Assessment of leakage along chainage HRT by Panthi 2006 approach)

Table 2 illustrates the empirical approach for leakage assessment by Panthi 2006. The various input param like joint persistence, joint spacing, the shortest distance of the surface from tunnel section, permeability factor, hydrostatic head, joint alteration, joint roughness and joint set number were given and finally computed the leakage along that particular chainage. The highest amount of leakage 16.16 l/min occurs along chainage 4+411 m whereas the minimum leakage of 5.02 l/min occurs along the chainage of 2+015 m. Figure 6 represents the bar graph for leakage assessment along various chainages by Panthi 2006 approach conducted before grouting. The finding demonstrates that the permeability of the rock mass decreases as depth from the surface increases. As the number of joint sets decreases and tight joint results in a low permeability factor below the surface as compared to the surface where joints are loose due to weathering.

4.5 Tokheim and Janbu 1984 Method

Table 3: Inflow along critical chainage of HRT by Tokheim and Janbu 1984 method

Chainage m	k m/s	G	hstatic m	q l/min/m
1+675	3.98E-07	4.04	16.37	0.61
2+015	2.07E-07	5.00	16.71	0.26
2+157	1.17E-07	4.69	16.86	0.16
2+878	1.06E-06	4.86	19.67	1.61
3+105	2.07E-06	5.34	21.09	3.08
4+106	2.37E-07	5.12	22.20	0.39
4+411	1.91E-07	4.99	23.17	0.33
4+516	2.23E-07	5.09	23.78	0.39
4+682	7.66E-07	5.14	24.74	1.39

Table 3 illustrates the analytical approach for leakage assessment by Tokheim and Janbu 1984 method. The various input param like hydraulic conductivity, length of the tunnel, static head, dynamic viscosity, and geometry factor were given and finally computed the leakage along that chainage. The highest amount of leakage 3.08 l/min/m occurs along chainage 3+105 m whereas the minimum leakage of 0.16 l/min/m occurs along the chainage of 2+157 m. Figure 7 represents the bar graph for leakage assessment along various chainages by Tokheim and Janbu 1984 method conducted after grouting.

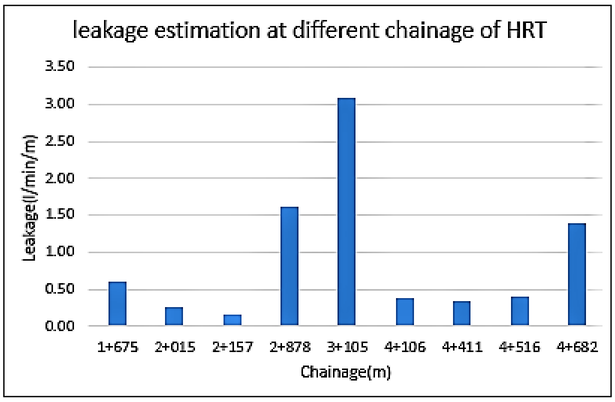

Figure 7: Bar graph for leakage assessment along various chainages by Tokheim and Janbu 1984 method)

Field measurement and leakage estimation from Tokheim and Janbu 1984 method in different chainage of HRT after grouting is carried out and found to be near from both method as shown in figure 8

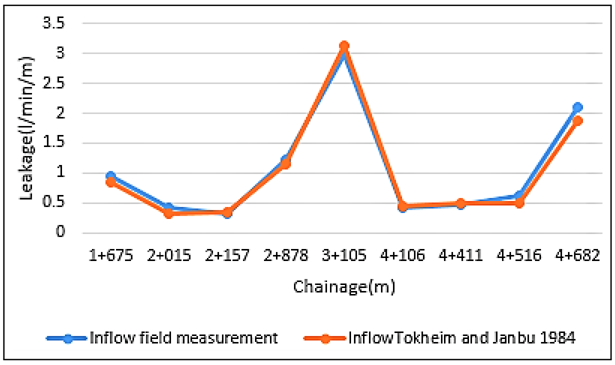

Figure 8: Inflow Comparison Between Field measurement and Tokheim Janbu 1984 Method)

4.6 Grouting Consumption

Table 4: Grout consumption at different chainage of HRT

S.N.	Chainage m	Grout Kg
1	1+675	305
2	2+015	455
3	2+157	620
4	2+878	400
5	3+105	260
6	4+106	720
7	4+411	110
8	4+516	905
9	4+682	820

Table 4 displays the grout consumption at the various chainage of HRT during post-injection grouting. Five holes are grouted taking one hole at the crown along with two holes each at valley and hill side at selected chainage. The highest amount of grout consumption is 905 kg along a chainage of 4+516 m whereas the minimum grout consumption is 110 kg along the a chainage of 4+411 m. Figure 9 represents the bar graph for grout consumption along various chainages during primary grouting.

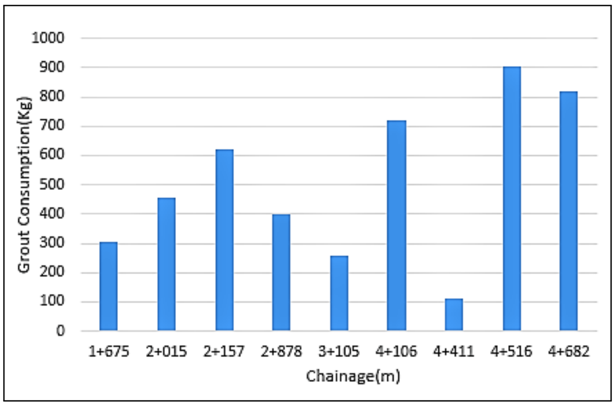

Figure 9: Grout consumption at chainage of HRT)

4.7 Performance Evaluation of Grouting on Leakage Control

Water inflow in the tunnel before and after the grout is calculated and the result found that the grouting plays an effective role to control the inflow as shown in figure 10.

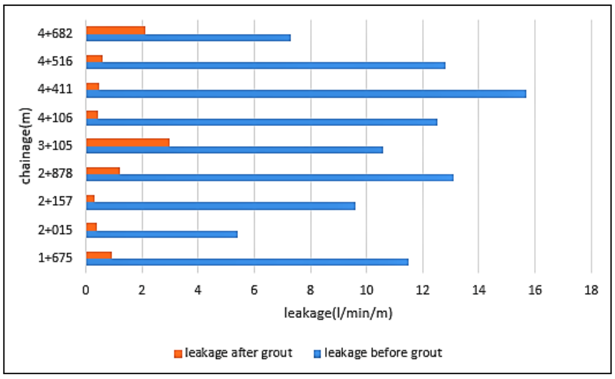

Figure 10: Performance of grouting on Inflow)

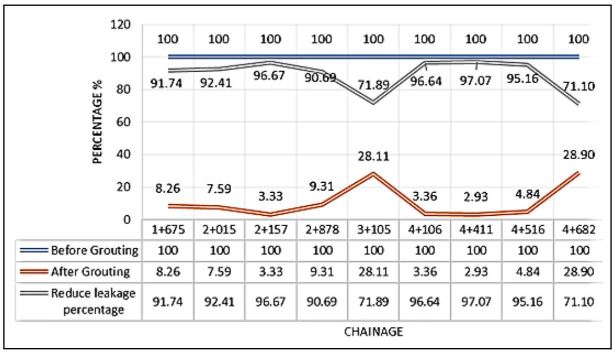

Figure 11: Inflow Percentage reduce after grouting)

Figure 11 represents the inflow comparison before and after grouting in the selected study chainage 1+675 m, 2+015 m, 2+157 m, 2+878 m, 3+105 m, 4+106 m, 4+411 m, 4+516 m, and 4+682 m. Among the chainages the highest percentage of 97.07% of leakage reduction is in chainage 4+411 m, whereas the lowest percentage is about 71.89% of reduction in inflow. It indicates that after post-grouting the significant percentage of inflow is reduced along the headrace tunnel chainage section.

4.8 Check for Secondary Grouting

The necessity of secondary grouting is mandatory for leakage assessment whether further leakage will occur or not. To verify the lugeon test is being carried out and check whether the lugeon value lies above or less than 1 [2]. Figure 12 is representing its verification.

Figure 12 shows five chainages 1+675 m, 2+878 m, 3+105 m, 4+106 m, and 4+682 m requires secondary grouting as lugeon value lies above 1. However, in other chainages secondary grouting is not necessary.

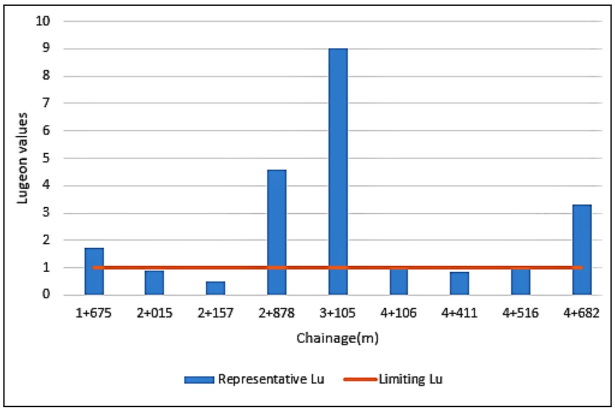

Figure 12: Lugeon values after the primary grouting at chainage of HRT)

5. Conclusion

1. The highest amount of leakage 16.16 l/min occurs along chainage 4+411 m whereas the minimum leakage of 5.02 l/min occurs along the chainage of 2+015 m by Panthi 2006 approach conducted before grouting. The leakage is maximum where the surface is minimum distance from tunnel section and is minimum where the surface distance from tunnel section is high.

2. After post grouting the significant percentage of inflow is reduced along the headrace tunnel chainage section as the highest percentage of 97.07% of leakage reduce in chainage 4+411 m, whereas the lowest percentage is about 71.89% of reduction in chainage 3+105 m. Inflow control is higher where the ground water pressure is minimum.

3. Five chainages 1+675 m, 2+878 m, 3+105 m, 4+106 m, and 4+682 m require secondary grouting as the lugeon value lies above 1 after primary grouting. The secondary grouting mainly required in those sections where there is high persistence, low spacing of joints, and low shortest distance of surface from tunnel section.

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