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Water Inflow Prediction and Grouting Design for Tunnel considering Nonlinear Hydraulic Conductivity

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Abstract

Grouting prevents groundwater leakage into tunnels, based on the exponent model that expresses the nonlinear variation of the hydraulic conductivity of the surrounding rock, the formulas for predicting the magnitude of water inflow and outer water pressure of the lining after grouting are deduced. The parameter analysis shows how the water inflow decreases as the hydraulic conductivity of the grouting circle diminishes and the thickness of the grouting circle increases. When the parameter α (attenuation coefficient), which expresses the decreasing amplitude of the permeability coefficient of the surrounding rock with depth, is greater than 0, the water inflow increases until it reaches a maximum at a certain depth, and then the inflow decreases to 0 if deep enough. After considering the variation in the hydraulic conductivity of the surrounding rock, the thickness and hydraulic conductivity of the grouting circle may be designed to be too large to reduce the magnitude of the water inflow. Meanwhile, to reach the limited drainage criterion of the tunnel groundwater, the grouting circle thickness decreases gradually as α increases after the nonlinear variation of the hydraulic conductivity of the surrounding rock, which can reduce the cost for plugging the groundwater. Thus, it is critical to consider hydraulic conductivity variation during water inflow predicting and grouting when designing tunnels.

Keywords: *water inflow prediction, grouting design, heterogeneous and isotropic surrounding rock, nonlinear hydraulic conductivity, tunnel*

1. Introduction

There are many mountains in China, and mountainous terrain accounts for approximately 69.4% of the entire national land area. To develop traffic infrastructure under such terrain conditions, a large number of tunnels are necessary. Generally, all tunnels are buried underground and thus surrounded and affected by groundwater during tunnel construction and operational periods. Specifically, ground collapse and environmental impacts are caused by water leakage and gushing. The higher the hydraulic head is, the more harmful the water is to the tunnel (Wang *et al*., 2005; Zhang *et al*., 2012; Farhadian *et al*., 2016b). Therefore, it is essential to effectively address groundwater to guarantee the safety of the tunnel during its construction and operational periods.

Presently, grouting is one of the main and most effective approaches for groundwater inflow plugging (Zhang *et al*., 2012; 2015). Through grouting, the fractures of the surrounding rock are filled and cemented, and the hydraulic conductivity of the surrounding rock decreases. Hence, the water inflow is decreased, and groundwater drainage is controlled. Many researchers focus on predicting the water inflow magnitude and grouting design. Polubarinova-Kochina (1962) used the well flow theory to estimate groundwater ingresses into tunnels in a semi-infinite homogeneous saturated aquifer. Goodman *et al*. (1965) checked the method by Polubarinova-Kochina and proposed an analytical and numerical solution for transient flow. Based on methods developed by Goodman, Freeze and Cherry (1977), Domenico and Schwartz (1998) deduced an estimating method on the assumption that the groundwater table is below the ground surface, which was suitable for deep tunnels. Zhang and Franklin (1993) and Tani (1999) used the Fourier transform method to present an analytical solution for water inflow into a tunnel, which introduces a hydraulic conductivity gradient. Tani (2003) described a method to predict water inflow for a circular tunnel in a semi-infinite aquifer, which could consider the internal lining pressure, leakage and recharge infiltration. Wang *et al*. (2004; 2005), Gao (2005) and Zhang *et al*. (2007) used well theory and Darcy's law to estimate ground water seepage into tunnels with grouting and lining. Fernandez and Moon (2010) proposed an analytical solution for predicting water inflow in the

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presence of hydro-mechanical interaction within the joints in the surrounding rock mass, which considering the stress-induced rock-mass permeability reduction in the tunnel vicinity. Butscher (2012) used analytical and numerical methods to calculate tunnel water inflow under different types of tunnel support structures. Farhadian *et al*. (2016a) developed an empirical formula to predict water inflow by a discrete network flow model, which included the influence factors of the joint orientation, aperture, spacing and interconnectivity. Nevertheless, for predicting the water inflow and parameter design for grouting in this research, it was assumed that the rock mass was homogeneous and isotropic. However, this assumption is not consistent for all rock masses, and it may lead to the overestimation of the water inflow, causing the inappropriate parameters to be selected for grouting design. To precisely calculate the amount of water inflow and appropriately design the grouting parameters, the heterogeneity character implied among the surrounding rock needs to be considered.

The present study aims to establish a formula for calculating the amount of tunnel water inflow after grouting based on the hydraulic conductivity nonlinear variation with depth. Then, on the basis of a parameter analysis, the thickness and hydraulic conductivity ranges of the grouting circle are discussed, and an example illustrates the reasonability of the proposed approach.

2. Relationship between Hydraulic Conductivity and Depth

Hydraulic conductivity is an extremely important parameter for determining water inflow and grouting parameters. The accurate evaluation of hydraulic conductivity is a basis for the accurate and realistic prediction of water inflow. In general, the hydraulic conductivity changes with depth based on the site investigation and analysis (Zhang and Franklin, 1993; Tani, 1999; Giacomini *et al*., 2008; Jiang *et al*., 2009; Jiang *et al*., 2010; Wang *et al*., 2009); the greater the depth, the smaller the hydraulic conductivity, sometimes with a difference of several orders of magnitude. For instance, a group of survey data by Zhang and Franklin (1993) illustrated that the hydraulic conductivity of the rock is about 10^{-5} m/sec at the depth of 10 m below ground, while at 60 m, the hydraulic conductivity decreases to 10^{-8} m/sec, which showed the deference for several orders of magnitude between the varying depths. Another set of data showed similar results that the hydraulic conductivity at the depth of 10 m was about 10^{-7} m/sec, while this number reduced to 10^{-11} m/sec at 40 m. These results obviously indicate the change in hydraulic conductivity with depth. This phenomenon is related to various factors, e.g., the depth. Note that the stress of the surrounding rock increases with depth, which leads to a narrowing of the fractures and a decrease in hydraulic conductivity. Furthermore, at larger depths, the better the rock integrity, the less pores and cracks appear in the rock; therefore, the hydraulic conductivity is reduced (Fernandez and Moon, 2010).

Several empirical formulas are used to describe the relationship between hydraulic conductivity and depth, and the most common and simple formula is the exponential model proposed by Louis (1974), which is based on a large number of in situ tests. The empirical formula can be expressed as follows (Zhang and Franklin, 1993; Tani, 1999; Louis, 1974):

$$k = k_s e^{-\alpha D} \tag{1}$$

where

k (m/s) = is the hydraulic conductivity at depth D;
k_s (m/s) = is the hydraulic conductivity at $D = 0$;
α (m^{-1}) = is an attenuation coefficient.

Note that α may be different for different areas. For instance, Louis (1974) measured the data for the fractured amphibolites and gneisses of two dam foundations through in situ testing in France. The attenuation coefficients α of the foundations were determined to be 0.04/m and 0.12/m. Furthermore, the attenuation coefficient α for Canada shale was 0.27/m with a double embolization water pressure test, and a Northumberland Straits tunnel provided in mudstone and sandstone results in the range of 30 m with an α-value of 0.08/m (Tani, 1999). Wan *et al*. (2010) verified a common regularity of aquifers exists as follows:

Table 1. Attenuation Coefficient for Different Areas (Wan *et al*., 2010)

Depth order of magnitude	Site and lithology	Depth scope (m)	Attenuation coefficient α (m^{-1})
10^0	MADE aquifer in the USA (loose accumulation)	2 – 7	0.38
	Borden aquifer in Canada (loose accumulation)	2.5 – 4.5	0.51
$10^1 - 10^2$	Northumberland Straits tunnel in UK (mudstones and sandstones)	0 – 30	0.08
	Dam site of hydropower station in Southwest China (basalt)	30 – 160	0.0219
	Dam site of hydropower station in Southwest China (basalt containing a tuffite interlayer)	40 – 150	0.0324
	The pumped-storage power station in Shandong (granite)	0 – 85	0.0227
	Dam site of Xiaolangdi Reservoir in China (sand mudstone)	0 – 150	0.0368
10^3	Oregon Cascades in the USA (unspecified)	0 – 2,000	0.004
	Nuclear waste treatment sites in Sweden (unspecified)	0 – 1,000	0.00641
	Stripa area in Sweden (granite)	0 – 2,500	0.0039

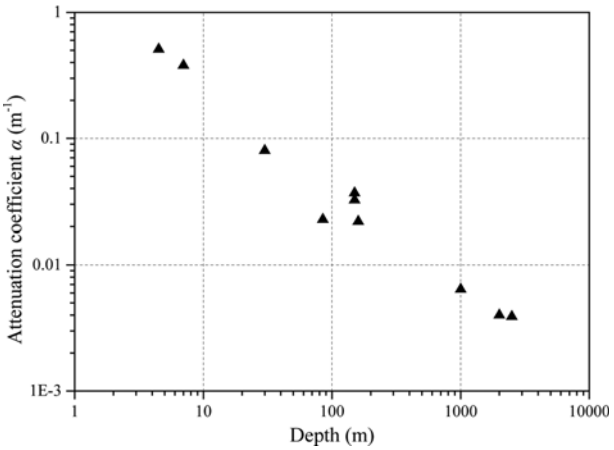

Fig. 1. Attenuation Coefficient for Different Areas (Note: Expressed in logarithmic coordinates)

the attenuation coefficient in hydraulic conductivity decreases with depth, as demonstrated by the collected and field investigation data in Table 1 and Fig. 1.

It can be seen that the hydraulic conductivity changes with depth are a generality based on the data from Table 1 and Fig. 1, and the hydraulic conductivity decreases with increasing depth. Therefore, it is necessary to consider the depth factor for issues regarding hydraulic conductivity, e.g., water inflow prediction and grouting parameter design.

The exponential model mentioned above is commonly used to account for the depth factor in calculations. However, the most important parameter in Eq. (1) is the attenuation coefficient α. The concrete procedures for determining the α value are as follows: measure the hydraulic conductivity of the rock at different depths, then obtain the α value based on a curve fitting method so as to obey the above exponential model. There are many methods used to determine the hydraulic conductivity (Sun *et al*., 2006), such as in-situ test methods, fracture measuring methods, the inversion analytic method and the rock quality design (*RQD*) estimating method (Jiang *et al*., 2009). In practice, the most common method is the in-situ method which includes pumping tests and packer injection tests (Ministry of Construction of the People's Republic of China, 2009). After the hydraulic conductivity of the rock is known, the attenuation coefficient α can be obtained by a curve fitting method.

3. Water Inflow Calculation after Grouting

Based on existing research (Cheng, 2014; Cheng *et al*., 2017), when the hydraulic conductivity changes with depth, the formula for calculating the water inflow of a tunnel may be expressed as follows:

$$q = \frac{2k_s e^{(-\alpha D)} \int_0^{\pi} e^{\alpha r \cos\theta} d\theta}{\ln(\frac{2H}{r})}(H - h_a) \tag{2}$$

where q ($m^3.m^{-1}.d^{-1}$) is the magnitude of water inflow of the tunnel, D (m) is the depth of the target point, k_s (m/s) is the hydraulic conductivity at the ground surface, α (m^{-1}) is the attenuation coefficient, r (m) is the radius of the tunnel, θ (°) is the clockwise angle between the lengthways axis and the radius of the point of interest, as shown in Fig. 1, H (m) is the distance from the groundwater table to the centerline of the tunnel, and h_a (m) is the hydraulic head at the tunnel circumference.

When the nonlinear variation of the hydraulic conductivity of the surrounding rock is not considered, i.e., the attenuation coefficient α is 0, and for a traffic tunnel, the hydraulic head h_a at the tunnel circumference is generally 0, therefore, Eq. (2) degenerates to Eq. (3), which is the empirical formula proposed by Goodman *et al*. (1965).

$$q = \frac{2\pi k_s H}{\ln(\frac{2H}{r})} \tag{3}$$

When a grouting measure is adopted to reduce the magnitude of groundwater inflow, the grouting circle is thin when compared to the surrounding rock in general; meanwhile, the grouting slurry is mixed uniformly; therefore, the grouting circle may be assumed to be homogeneous and isotropic, and the hydraulic conductivity is a constant. When the lining is molded in the field and the thickness is small, the hydraulic conductivity may also be assumed to be a constant. After grouting and lining, the calculated model of the tunnel is as illustrated in Fig. 2.

In Fig. 2, r_h (m) is the outer radius of the surrounding rock, r_j (m) is the outer radius of the grouting circle, r_l (m) is the outer radius of the lining, r_0 (m) is the inner radius of the lining, and h_0 (m) is the hydraulic head.

3.1 Basic Assumptions

1. The surrounding tunnel rock is heterogeneous and isotropic;
2. Without regard to the grouting injection process and it is

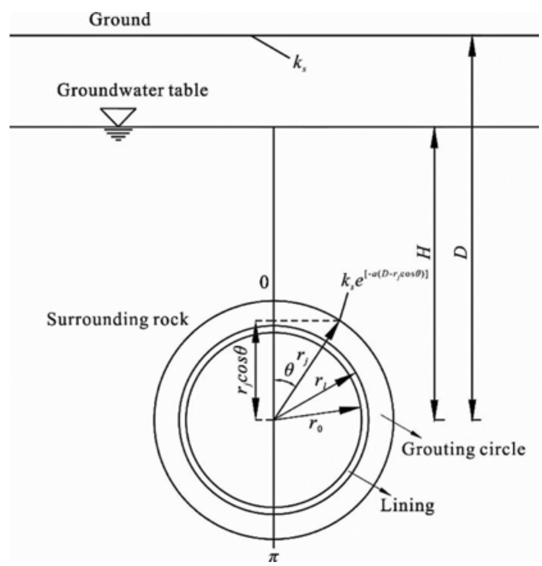

Fig. 2. Calculating Model of the Grouting Circle and Lining

assumed that the grouting circle and lining are both homogeneous and isotropic;

3. The groundwater flow is steady and assumed to follow Darcy's law;
4. The fluid is incompressible and can be released instantaneously;
5. The tunnel section is circular.

3.2 Formula Derivation

Equation (2) is the magnitude of the water inflow calculating equation with the heterogeneous and isotropic surrounding rock, and it can be transformed to the following:

$$H-h_a=\frac{q}{2k_s e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r}) \tag{4}$$

After grouting, the hydraulic head of the outer grouting circle can be expressed as

$$h_h-h_j=\frac{q}{2k_s e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j}) \tag{5}$$

where h_h (m) is the hydraulic head of the outer surrounding rock, and h_j (m) is the hydraulic head of the outer grouting circle.

Compared to the surrounding rock, the grouting circle and lining are thin and are assumed to be homogeneous and isotropic. According to Darcy's law, the water inflow through unit time and unit length may be described as follows:

$$q_j=2\pi r k_j dh/dr \tag{6}$$

The above equation can be rewritten as

$$dr/r=2\pi k_j dh/q_j \tag{7}$$

where k_j (m/s) is the hydraulic conductivity of the grouting circle. When the boundary condition is given as $r = r_j$, $h = h_j$, $r = r_l$, $h = h_l$, water inflow q_j for an arbitrary section that ranges from r_l to r_j is obtained by integrating Eq. (7); it is written as

$$q_j=\frac{2\pi k_j\left(h_j-h_l\right)}{\ln\left(r_j/r_l\right)} \tag{8}$$

where h_l (m) is the hydraulic head of the inner grouting circle.

Based on the same theory, the water inflow q_l for an arbitrary section in the lining is deduced as follows:

$$q_l=\frac{2\pi k_l\left(h_l-h_0\right)}{\ln\left(r_l/r_0\right)} \tag{9}$$

where h_0 (m) is the hydraulic head of the inner circle of the lining. When the inner grouting circle and outer lining circle are the same section, the hydraulic head of the outer lining circle is also h_l.

Equations (8) and (9) can be rewritten as

$$h_j-h_l=\frac{q_j}{2\pi k_j}\ln(\frac{r_j}{r_l}) \tag{10}$$

$$h_l-h_0=\frac{q_l}{2\pi k_l}\ln(\frac{r_l}{r_0}) \tag{11}$$

Equations (5), (10), and (11) are used in the model to calculate the hydraulic head of the surrounding rock, grouting circle and lining separately. Based on seepage continuity theory, the water inflow from the surrounding rock, grouting circle and lining are the same, i.e., $q = q_j = q_l$. Combining Eqs. (5), (10) and (11),

$$\begin{cases} h_h-h_j=\dfrac{q}{2k_s e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\dfrac{2H}{r_j}) \\ h_j-h_l=\dfrac{q_j}{2\pi k_j}\ln(\dfrac{r_j}{r_l}) \\ h_l-h_0=\dfrac{q_l}{2\pi k_l}\ln(\dfrac{r_l}{r_0}) \end{cases} \tag{12}$$

By solving Eq. (12), the following formula can be obtained for calculating the water inflow q:

$$q=\frac{2\pi\left(h_h-h_0\right)k_s}{\dfrac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\dfrac{2H}{r_j})+\dfrac{k_s}{k_j}\ln(\dfrac{r_j}{r_l})+\dfrac{k_s}{k_l}\ln(\dfrac{r_l}{r_0})} \tag{13}$$

Substituting Eq. (13) into Eq. (12), where the groundwater pressure is the product of the hydraulic head and the volume-weight of the water, the groundwater pressure on the outer lining and grouting circle may be described as

$$\begin{aligned} p_j &= r_w h_j = r_w\left(h_0+\frac{q}{2\pi k_j}\ln(\frac{r_j}{r_l})+\frac{q}{2\pi k_l}\ln(\frac{r_l}{r_0})\right) \\ &= r_w\left(h_0+\frac{\left(h_h-h_0\right)\ln(\dfrac{r_j}{r_l})}{\dfrac{k_j}{k_s}\dfrac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\dfrac{2H}{r_j})+\ln(\dfrac{r_j}{r_l})+\dfrac{k_j}{k_l}\ln(\dfrac{r_l}{r_0})}\right. \\ &\quad\left.+\frac{\left(h_h-h_0\right)\ln(\dfrac{r_l}{r_0})}{\dfrac{k_l}{k_s}\dfrac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\dfrac{2H}{r_j})+\dfrac{k_l}{k_j}\ln(\dfrac{r_j}{r_l})+\ln(\dfrac{r_l}{r_0})}\right) \end{aligned} \tag{14}$$

or

$$p_j=r_w h_j=r_w\left(h_h-\frac{\pi\left(h_h-h_0\right)}{\pi\ln(\dfrac{2H}{r_j})+\dfrac{k_s}{k_j}\ln(\dfrac{r_j}{r_l})e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta+\dfrac{k_s}{k_l}\ln(\dfrac{r_l}{r_0})e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j})\right) \tag{15}$$

where p_j is the groundwater pressure on the grouting circle.

$$p_l = r_w h_l = r_w \left(h_0 + \frac{(h_h - h_0)}{\frac{k_l}{k_s}\ln(\frac{2H}{r_j})\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta} + \frac{k_l}{k_j}\ln(\frac{r_j}{r_l}) + \ln(\frac{r_l}{r_0})} \ln(\frac{r_l}{r_0}) \right) \tag{16}$$

where p_l is the groundwater pressure on the lining.

The groundwater table can be determined at the early stage of the engineering investigation as h_w and $h_h = h_w$. In addition, the traffic tunnel does not have an inner pressure, $h_0 = 0$. In this case, Eqs. (13), (14), (15) and (16) can be simplified as

$$q = \frac{2\pi h_w k_s}{\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j}) + \frac{k_s}{k_j}\ln(\frac{r_j}{r_l}) + \frac{k_s}{k_l}\ln(\frac{r_l}{r_0})} \tag{17}$$

$$p_j = r_w \left(\frac{h_w \ln(\frac{r_j}{r_l})}{\frac{k_j}{k_s}\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j}) + \ln(\frac{r_j}{r_l}) + \frac{k_j}{k_l}\ln(\frac{r_l}{r_0})} + \frac{h_w \ln(\frac{r_l}{r_0})}{\frac{k_l}{k_s}\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j}) + \frac{k_l}{k_j}\ln(\frac{r_j}{r_l}) + \ln(\frac{r_l}{r_0})} \right) \tag{18}$$

$$p_j = r_w \left(h_w - \frac{\pi h_w}{\pi \ln(\frac{2H}{r_j}) + \frac{k_s}{k_j}\ln(\frac{r_j}{r_l})e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta + \frac{k_s}{k_l}\ln(\frac{r_l}{r_0})e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta} \ln(\frac{2H}{r_j}) \right) \tag{19}$$

$$p_l = r_w \left(\frac{h_w}{\frac{k_l}{k_s}\ln(\frac{2H}{r_j})\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta} + \frac{k_l}{k_j}\ln(\frac{r_j}{r_l}) + \ln(\frac{r_l}{r_0})} \ln(\frac{r_l}{r_0}) \right) \tag{20}$$

Equations (17) to (20) are formulas for calculating the water inflow, the groundwater pressure on the grouting circle, and the groundwater pressure on the lining separately after grouting and lining. If grouting and tunnel lining are not considered, k_j and k_l are implied to be infinity, and Eq. (17) can be simplified as Eq. (2), which degenerates to an equation for calculating the tunnel water inflow without grouting and lining. Based on Eqs. (17) to (20), the parameter for the water inflow and grouting can be calculated.

4. Discussions

4.1 Relationship between the Water Inflow and Hydraulic Conductivity of the Grouting Circle

Without lining, the hydraulic conductivity of the lining approaches positive infinity, i.e., $k_l \to \infty$. By simplifying Eq. (17),

$$q = \frac{2\pi h_w k_s}{\frac{\pi}{e^{(-\alpha D)}\int_0^{\pi} e^{\alpha r\cos\theta}d\theta}\ln(\frac{2H}{r_j}) + \frac{k_s}{k_j}\ln(\frac{r_j}{r_l})} \tag{21}$$

Following Eq. (21), r_0 is set to 5 m, the grouting circle is set to 5 m, i.e., r_j is 10 m, k_s is 9.26×10^{-7} m/s, the attenuation coefficient α is set to 0, 0.005, and 0.01, and the hydraulic conductivity of the grouting circle changes.

Figures 3 to 5 show the relationship between water inflow and depth, accounting for the hydraulic conductivity decrease of the grouting circle. The hydraulic conductivity of the grouting circle is expressed by the ratio of k_s/k_l.

It can be seen from Figs. 3 to 5 that water inflow decreases as the hydraulic conductivity of the grouting circle diminishes. This phenomenon implies that the hydraulic conductivity decrease of the grouting circle can achieve the purpose of reducing the water inflow. When α is not equal to 0, the water inflow increases first, and then it reduces until it approaches 0. At the same depth, the

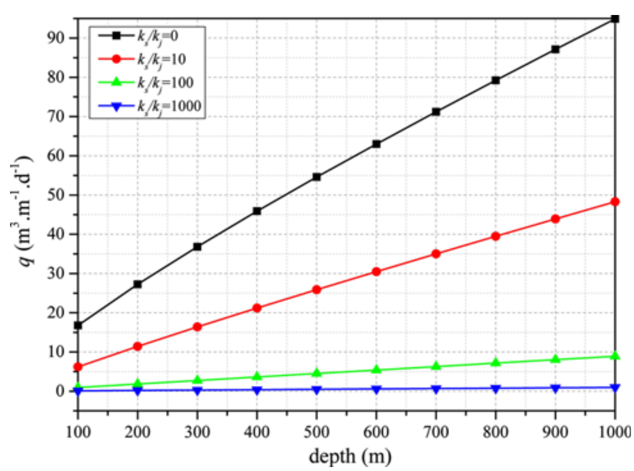

Fig. 3. Water Inflow Variation with Depth when α = 0

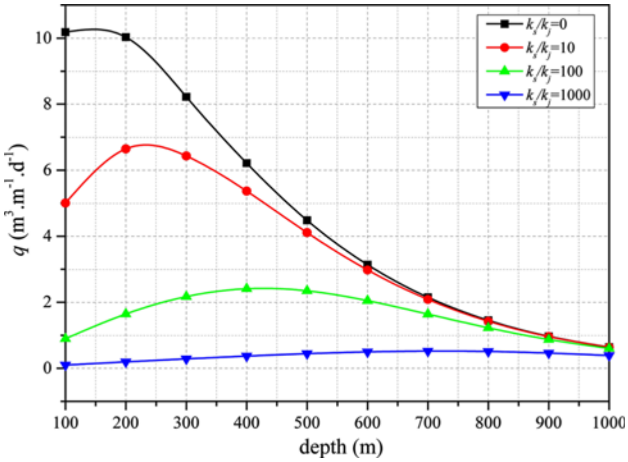

Fig. 4. Water Inflow Variation with Depth when α = 0.005

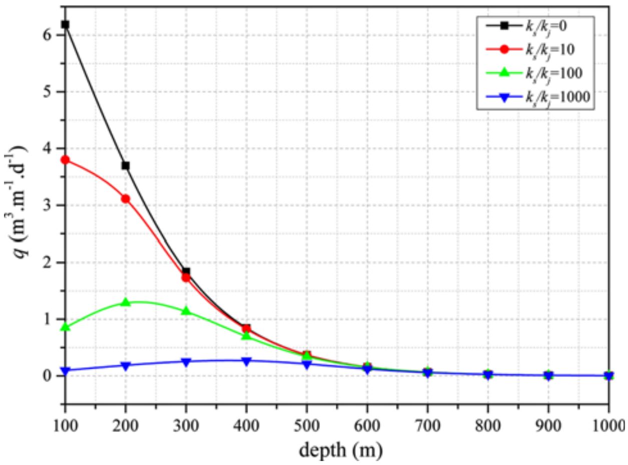

Fig. 5. Water Inflow Variation with Depth when α = 0.01

water inflow decreases as α increases.

In Fig. 3, the condition α = 0 implies that no consideration is given to the hydraulic conductivity variation of the surrounding rock. The water inflow increases with depth, and at the same depth, the water inflow decreases when the hydraulic conductivity of the grouting circle diminishes.

Figures 4 and 5 show other results. When $\alpha \neq 0$, the water inflow increases at the beginning, then decreases with depth; when it reaches a certain depth, the water inflow does not diminish any more although the hydraulic conductivity decreases continually.

As shown in Fig. 5 and Table 2, the curve for the case of k_s/k_j = 0 and the curve for the case of k_s/k_j = 10 are compared for the depth range from 0 to 500 m. The water inflow decreases with the hydraulic conductivity decreasing at the same depth. When the depth exceeds 500 m, the two curves coincide, indicating that if the hydraulic conductivity of the grouting circle is set to 10% of that of the surface rock, this does not achieve the purpose of plugging water. However, if the hydraulic conductivity of the grouting circle decreases to 1/100 of that of the surface rock, the water inflow diminishes. When the depth exceeds 700 m, the

Table 2. Water Inflow Variation with Depth when α = 0.01

Water inflow	h(m)	Parameter of grouting circle			
		k_s/k_j = 0 (without grouting)	k_s/k_j = 10	k_s/k_j = 100	k_s/k_j = 1,000
q (m^3.m^{-1}.d^{-1})	100	6.188	3.799	0.849	0.097
	300	1.838	1.731	1.133	0.254
	500	0.369	0.366	0.343	0.211
	700	0.065	0.065	0.065	0.060
	800	0.027	0.027	0.027	0.026
	900	0.011	0.011	0.011	0.011
	1,000	0.004	0.004	0.004	0.004

curves for the cases of k_s/k_j = 0, k_s/k_j = 10 and k_s/k_j = 100 coincide as in Fig. 5, which implies that even if the hydraulic conductivity of the grouting circle decreases to 1/100 of that of the surface rock, the purpose for plugging water is not achieved when the depth exceeds 700 m. When the hydraulic conductivity of the grouting circle is set to be 1/1,000 of that of the surface rock, the curves for four cases under k_s/k_j = 0, k_s/k_j = 10, k_s/k_j = 100 and k_s/k_j = 1,000 coincide when the depth exceeds 900 m. This implies that even if the hydraulic conductivity of the grouting circle achieves 1/1,000 of that of the surface rock, the conductivity would not achieve the purpose of the plugging water when the depth exceeds 900 m. Similar results can be seen in Fig. 4.

The reason for the above phenomenon is that when the variation of the hydraulic conductivity with depth is ignored, the hydraulic conductivity of the surrounding rock is constant, and the hydraulic conductivity of the grouting circle is always less than the hydraulic conductivity of the surrounding rock. As a consequence, the water inflow decreases as the hydraulic conductivity of the grouting circle diminishes at the same depth. However, when this relationship is taken into account, the hydraulic conductivity of the surrounding rock decreases gradually, while the hydraulic conductivity of the grouting circle is a constant as the ratio of k_s, thus the hydraulic conductivity of the surrounding rock will be less than that of the grouting circle at a certain depth. Using the results in Fig. 5 as an example, the curves for cases under k_s/k_j = 0 and k_s/k_j = 10 coincide at a depth of 500 m. This indicates that the hydraulic conductivity of the surrounding rock is less than k_s/10 when the depth exceeds 500 m, while the hydraulic conductivity of the grouting circle is k_s/10. Therefore, the grouting circle with a hydraulic conductivity of k_s/10 cannot achieve the purpose of plugging groundwater unless it reduces the hydraulic conductivity of the grouting circle. Nevertheless, despite reducing the hydraulic conductivity of the grouting circle to k_s/100, it is invalid to plug the groundwater when the depth exceeds 700 m. When the depth exceeds 900 m, the grouting circle cannot reduce the magnitude of the groundwater inflow even though the hydraulic conductivity reaches k_s/1,000. For these cases, grouting cannot reduce the magnitude of the groundwater inflow if the hydraulic conductivity of the grouting circle is greater than that of the surrounding rock.

Consequently, the nonlinear variation of the hydraulic conductivity of the surrounding rock should be considered during the scheme for water plugging to avoid plugging ineffectiveness. While estimating or testing the hydraulic conductivity around the tunnel circle, the minimum hydraulic conductivity criteria are applied to design of the grouting parameters.

It can be seen from Figs. 4 and 5 that, as the value of α (attenuation coefficient) increases, the curves coincide for the cases of k_s/k_j = 0, k_s/k_j = 10, k_s/k_j = 100, k_s/k_j = 1,000 at a shallower depth, which implies that the greater the value of α (attenuation coefficient) is, the faster the hydraulic conductivity of the surrounding rock diminishes with depth. When the value decreases to the designed hydraulic conductivity of the grouting circle, the grouting circle is ineffective for plugging the groundwater.

4.2 Relationship between Grouting Circle Thickness and Water Inflow

Following Eq. (21), r_0 is set to 5 m; k_s is 9.26 × 10^{-7} m/s; the grouting circle thickness is set to 1 m, 3 m, 5 m, and 7 m; the attenuation coefficients α are set to 0, 0.005, and 0.01; and the hydraulic conductivity of the grouting circle is set to a constant. The results between grouting circle thickness and water inflow are obtained as shown below.

Figure 6 shows the results for the case in which the variation of the hydraulic conductivity is not considered, i.e., α = 0. The water inflow increases with an increasing depth, and the inflow decreases when the grouting circle thickness increases. Fig. 7 shows similar results with Fig. 6, and the difference is only that the water inflow does not linearly increase with depth when α = 0.001.

Figure 8 illustrates that at the same depth, water inflow decreases when the grouting circle thickness increases within the depth range from 0 – 225 m; however, when the depth exceeds 225 m, the water inflow increases when the grouting circle thickness increases. This occurs because when the depth exceeds 225 m, the hydraulic conductivity of the surrounding rock decreases to k_s/10, which equals that of the grouting circle under the conditions of α = 0.01 and k_s/k_j = 10. The grouting circle is unable to plug the groundwater in this situation. However,

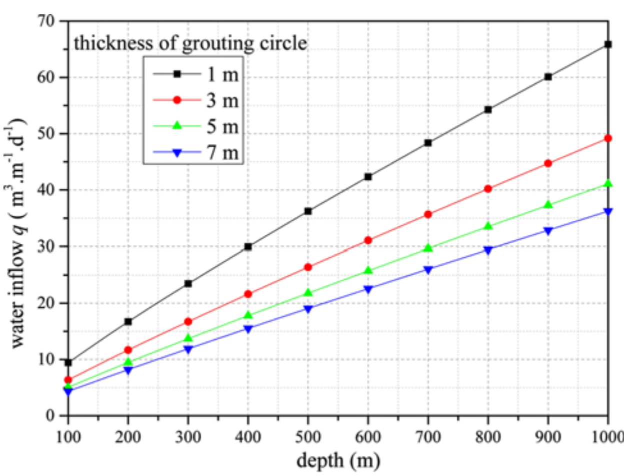

Fig. 6. Water Inflow Variation with the Grouting Circle Thickness when α = 0, k_s/k_j = 10

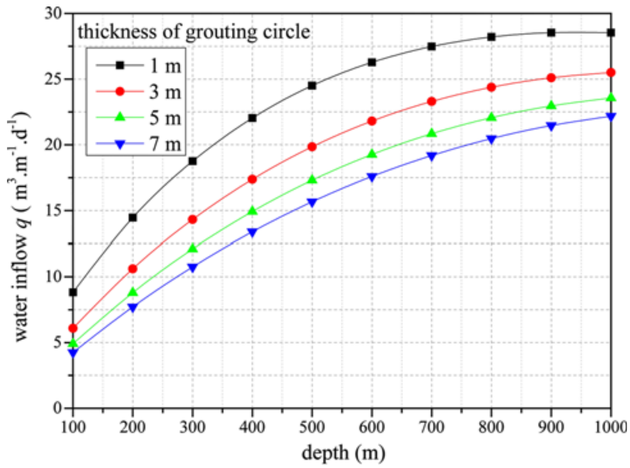

Fig. 7. Water Inflow Variation with the Grouting Circle Thickness when α = 0.001, k_s/k_j = 10

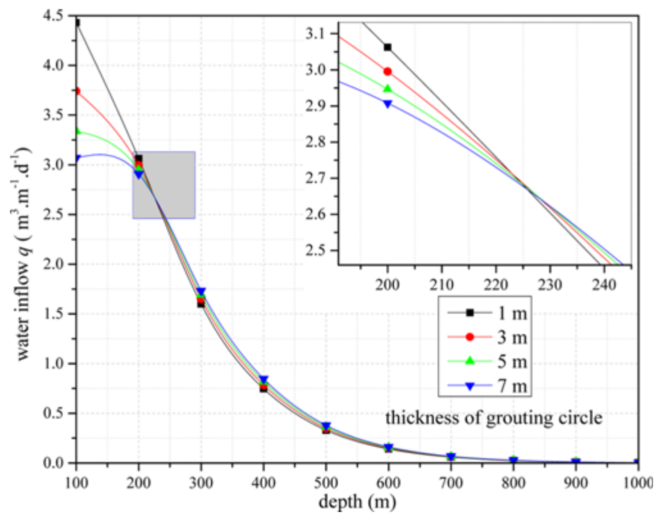

Fig. 8. Water Inflow Variation with the Grouting Circle Thickness when α = 0.01, k_s/k_j = 10

because the thickness of the surrounding rock is a constant, increasing the grouting circle thickness implies that the surrounding rock has been diminished, which also increases the hydraulic conductivity of the surrounding rock instead of grouting, thereby the water inflow increases.

Consequently, during grouting design phase, the hydraulic conductivity of the surrounding rock and the attenuation coefficient α must be determined precisely to avoid selecting an erroneous grouting parameter that is unable to plug groundwater. In addition, to present the optimal design scheme, the hydraulic conductivity and grouting circle thickness should be taken into consideration.

5. Case Study

The case of a highway tunnel in the research by Li and Zou (2013) is selected for study, and the tunnel parameters are as follows. The tunnel depth is 220 m, and the hydraulic head is 210 m, the hydraulic conductivity of surrounding rock before grouting is 6.2

Table 3. Water Inflow under the Condition of Different Values of α

α	0	0.0001	0.001	0.005	0.006	0.01
Q ($m^3.m^{-1}.d^{-1}$)	16.09	15.76	13.05	5.63	4.57	1.98

Table 4. Thickness of the Grouting Circle under Different α Values

α	0	0.0005	0.001	0.003	0.005
Thickness of grouting circle (m)	1.1	1.05	0.99	0.66	0.18

$\times$ 10^{-7} m/s, the conductivity decreases to 1.2 $\times$ 10^{-8} m/s after grouting, and the tunnel radius is 5.2 m. Following Eq. (3), the tunnel water inflow without grouting is 16.09 $m^3.m^{-1}.d^{-1}$, which is close to the measured value of 18.93 $m^3.m^{-1}.d^{-1}$.

The limited drainage criterion of the tunnel is 5 $m^3.m^{-1}.d^{-1}$ (Li and Zou, 2013). Following Eq. (21), the grouting circle thickness is calculated without considering the hydraulic conductivity variation with depth, thus, $\alpha = 0$. The grouting circle thickness is 1.1 m, which is approximately equal to the result of 1.22 m by Li and Zou (2013).

If the hydraulic conductivity variation with depth is taken into consideration, setting $k_s = 6.2 \times 10^{-7}$ m/s, on the basis of Eq. (17), the water inflow under the condition of different values of α can be obtained, as shown in Table 3.

Table 3 indicates that the water inflow Q decreases with increasing values of α. If $\alpha \geq 0.006$, the tunnel water inflow is less than the limited drainage criterion; however, when $\alpha \leq 0.006$, the grouting measures must be adopted to plug the groundwater. Since the hydraulic conductivity of the grouting circle is 1.2 $\times$ 10^{-8} m/s, when $0 \leq \alpha \leq 0.006$, the thickness of the grouting circle under the condition of different α values is calculated, and the result is given in Table 4.

It can be seen from Table 4 that, to reach the limited drainage criterion of the tunnel groundwater, the grouting circle thickness decreases gradually with increasing α. Thus, the cost of grouting for groundwater plugging can be decreased when the hydraulic conductivity variation of the surrounding rock is considered.

6. Conclusions

1. The nonlinear variation of hydraulic conductivity is considered by following the exponent model and Darcy's law to derive the equations for calculating the magnitude of water inflow and the grouting parameters. When the grouting and geological parameters are given, the amount of water inflow after grouting can be obtained.
2. When considering the hydraulic conductivity variation of the surrounding rock, the water inflow increases until it reaches a maximum at a certain depth; then, the inflow decreases and approaches 0 when the depth is large enough. At the same depth, the amount of water inflow decreases with α (attenuation coefficient) increasing. After grouting, the amount of water inflow decreases when the hydraulic conductivity of the grouting circle decreases and the grouting circle thickness increases.
3. During grouting design, the change of the hydraulic conductivity of the surrounding rock with depth must be considered carefully following the analysis; otherwise, the hydraulic conductivity of the grouting circle may be larger than that of the surrounding rock, which causes the grouting measures to fail to plug the water.
4. The case study illustrates the validity and correctness of the proposed approach. To reach the limited drainage criterion of tunnel groundwater, the grouting circle thickness decreases gradually with an increase in α after considering the hydraulic conductivity variation of the surrounding rock, which can reduce the cost for groundwater plugging.

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