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# Common Approximations to the Water Inflow into Tunnels

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## 1. Introduction

Groundwater inflow into hard rock tunnels is very difficult to estimate accurately. In practice, estimates range from grossly low, which then results in large cost overruns and hazardous conditions in the workplace, to grossly high, which leads contractors to ignore them. This chapter looks at some of the reasons why inflow estimates are difficult to make. It also suggests a few practical ways to get past some of these problems.

There are two main problems with inflow estimates. The first is the lack of simple, realistic equations or models that can be readily applied to hard-rock tunnels. This difficulty may turn out to be unavoidable, and this chapter does not attempt to improve upon the situation. The second difficulty is that the practical range of permeability in fractured rock typically ranges over at least six orders of magnitude, and this range typically repeats again and again over the lengths of long tunnels. This range of permeability, combined with the length of the tunnels, makes hard-rock tunnels very different from well fields, major aquifers, and other applications of practical hydrology.

This chapter is concerned with hard-rock tunnels in fractured rock that are excavated below groundwater table. These tunnels are commonly constructed under atmospheric pressure using either drill and blast methods or main-beam tunnel boring machines. Lining in these tunnels is typically installed only after excavation is completed, and only where needed. During construction, groundwater flows freely into these tunnels through fractures in the rock. Where the rock is tight and the potentiometric head above the tunnel is low, the inflow will be small. Where the rock contains large, open fractures or where the head is high, the inflow will be substantial. Where the rock contains both large fractures and high head, the inflows can be catastrophic.

Tunnel designers must determine, and tell the contractor, how much water to expect over both the total length of the tunnel and in the heading area. The total flow is used to design appropriately sized pumping systems and water treatment plants. The inflows in the heading will affect the contractor's construction methods and schedule. Major delays can occur if either is underestimated. Excessive and unnecessary cost can result if either is grossly overestimated.

In hard-rock tunnels, most of the inflow comes from a few places, some of the inflow comes from many places, and much of the tunnel is dry. The total inflow accumulates over the

length of the tunnel and is the sum of all the inflows. This is the fundamental observation from hard rock tunnels, and the root of much of the trouble with estimating inflow using standard hydrologic methods.

There are several analytical expressions in literature to calculate groundwater discharges into tunnels, such as Goodman(1965), Lohman(1972), Zhang(1993), Heuer (1995), Lei(1999), Karlsrud(2001), Raymer(2001) and El Tani (2003). In addition them, Katibeh and Aalianvari (2010) proposed a new method to classified tunnel length with accordance to groundwater flow into tunnels. This chapter provides a summary of analytical methods to calculate groundwater discharges into tunnels.

## 2. Inflow equations

Groundwater inflow equations are based on Darcy's Law and conservation of mass. Darcy's Law holds that inflow equals the permeability times the gradient. Conservation of mass holds that the inflow equals the recharge plus the water released from storage.

### 2.1 Thiem equation

Darcy's Law is $Q = (KA)I$, where I is the hydraulic gradient, KA is the permeability (K) across an area (A), and Q is the inflow. In this version of the equation (KA) are taken together in parentheses as a single term to emphasize the fact that actual water flows through a finite volume of rock mass, rather than a theoretical unity.

If Darcy's Law is configured for cylindrical coordinates around a vertical well, then the Thiem equation from well hydraulics results (Figure 1):

$$Q = 2\pi T (H_2 - H_1) / \ln(r_2/r_1) \qquad (1)$$

where the gradient is expressed as $(H_2-H_1)/\ln(r_2/r_1)$. $H_1$ and $H_2$ are the potentiometric heads in the aquifer at two arbitrary points having radial distances $r_1$ and $r_2$ from the center of the well. The hydraulic conductivity (K) and unit area (A) are handled together in the term $2\pi T$, where T is the transmissivity. Provided that the rock is uniformly permeable, then T=Kb, where b is the vertical thickness of the aquifer. The Thiem equation (as shown above) is based on the following assumptions:

- Flow is radial toward the well and non-turbulent. The well produces from the full thickness of the aquifer.
- The well has reached steady-state flow, meaning that the cone of depression around the well has encountered a supply of water sufficient to replenish the water produced from the well. At this point, the cone of depression stops expanding and water is no longer being released from storage.

The Thiem equation can be applied to tunnels by turning the well on its side. The axis of the well is now the centerline of the tunnel. As with a well, the cone of depression is radial around the tunnel. Unlike a well, the potentiometric surface is a bit more abstract, and will look like a trough along the axis of the tunnel if measured at springline (or any constant elevation).
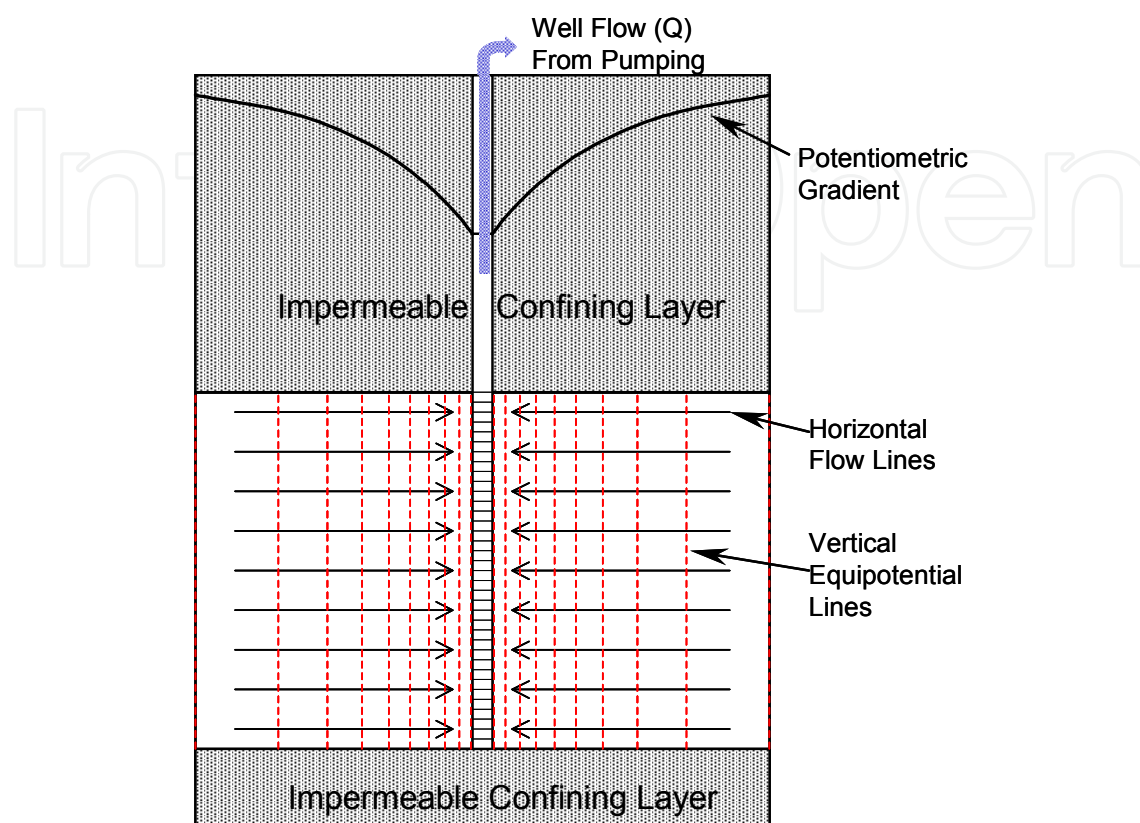
Fig. 1. Thiem equation for radial flow to wells. Equipotential lines are vertical and concentric about the well. Flow lines are horizontal and radial toward the well. The aquifer extends infinitely, and at infinity recharge equals well flow.

For convenience, $r_1$ can be the tunnel radius and $H_1$ the potentiometric head at the tunnel radius. If the tunnel is under construction at atmospheric pressure, $H_1$ is the elevation of the tunnel wall. $H_2$ is the head of water some distance $r_2$ from the centerline of the tunnel. Transmissivity, which is the coefficient of proportionality, now has to be oriented horizontally along the axis of the tunnel, becoming the "horizontal transmissivity."

Note that $H_1$ is not a specific point, but ranges between the crown and invert of the tunnel. This difference is trivial for deep tunnels. For large-diameter, shallow tunnels, however, this difference might become significant.

"Horizontal transmissivity" ($T_h$) is the coefficient of proportionality for the horizontal Thiem equation. It is placed in quotes because transmissivity is normally considered to represent the vertical thickness of the aquifer, rather than the horizontal length of the tunnel. The concept is important and worthy of indulgence to make a point. The fundamental observation shows that permeability varies over many orders of magnitude along the length of a hard-rock tunnel. This is similar to the horizontal stratification in a sedimentary aquifer

penetrated by a vertical well. The "horizontal transmissivity" requires that the average permeability ($K_{avg}$) be considered when calculating inflow, rather than the idealized value of hydraulic conductivity. The average hydraulic conductivity is $K_{avg} = T_h / L$, where L is the length of the tunnel.

The assumptions of the Thiem equation still have to apply for tunnels. Flow has to be radial toward the tunnel. The tunnel has to be long, such that that non-radial flow around the ends of the tunnel is negligible compared to the total inflow. The water table has to be high above the tunnel and not draw down close to the tunnel. The cone of depression has to have room to expand radially to a point where the water captured is large enough to provide for the inflow, even if an infinite supply of water is never encountered. These assumptions are reasonable for tunnels as long as the rock is nearly tight, the inflows very small, and the tunnel deep below the water table. In practice, tunnels that meet these conditions are less likely to have groundwater problems because the inflow rates will have to be very small.

## 2.2 Goodman's equation

Goodman (1965) considered the question of a tunnel lying beneath a lake or large river. He considered this lake or river to be an infinite source of water and applied the method of images (Lohman, 1972) to derive the following equation (Figure 2):

$$Q_L = 2\pi K H_0 \, / \, \ln(2z/r) \tag{2}$$

where $H_0$ is the head of water above the tunnel, and z is the distance from the tunnel to the bottom of the lake. Goodman also divided T by the length of the tunnel to put the equation in terms of hydraulic conductivity (K) and inflow per unit length of tunnel ($Q_L$). This equation only applies to steady-state inflow along the length of the tunnel. The inflow is steady state because the lake acts as an infinite recharge boundary, which causes the cone of depression to stop expanding.
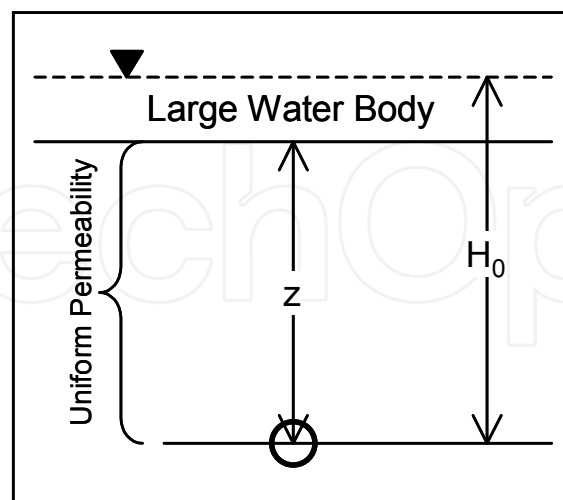


Fig. 2. Goodman's (1965) model for tunnels beneath a large water body. z is distance from centerline to top of rock. $H_0$ is initial head between water surface and centerline.

Workers who have tried to apply this equation commonly report that actual inflows deviate greatly from the inflows predicted by this equation. Zhang and Franklin (1993) report that

measured inflows range from 90 percent lower to 30 percent higher than predicted by Eq. 2. Heuer's (1995) work indicates that inflows from his projects have tended to be about one eighth (87.5 percent) lower than predicted by Eq. 2. My own experience from fairly shallow tunnels (typically less than 100 m) is in rough agreement with Heuer's. Other colleagues of mine report that inflows from deeper tunnels increase toward and then pass the inflows predicted by Eq. 2 as the tunnels become deeper.

Freeze and Cheery (1979) further modified Goodman's (1965) solution by replacing z with $H_0$. In this solution, the water table is modeled as an infinite recharge boundary. Freeze and Cherry's (1979) solution will give minimally lower estimates than Goodman's because $H_0$ is greater than z and both terms are in the logarithm. In a real tunnel, however, it is unlikely that recharge to the water table from precipitation could keep up with the yield from the larger fractures. This will serve to draw down the water table substantially around the tunnel and thus lower the head ($H_0$) to a point where recharge can keep up with the reduced inflow.

### 2.2.1 Assumptions

Goodman's model is based on some major simplifying assumptions. First, the tunnel is infinitely long and the water table will never be drawn down close to the crown of the tunnel. Second, at some height z above the tunnel, there is an infinite reservoir of water, such as a lake or river that cannot be depleted by inflow to the tunnel. The head ($H_0$) must be at least as high as the base of this reservoir (z) above the tunnel. Third, the hydraulic conductivity (K) of the ground between the tunnel and the base of the reservoir is the only factor limiting the rate at which this infinite supply of water drains into the tunnel. Fourth, the flow is non-turbulent and the ground is homogeneous and isotropic, such as a uniform sand or silt without fractures.

Goodman's assumption of an infinite reservoir above the tunnel is probably somewhat reasonable in many hard-rock tunnel situations in the eastern United States, including the Chattahoochee Tunnel. It is not necessary that this reservoir be a lake or a river: merely that it be a thick, saturated zone that is much more porous and permeable than the underlying bedrock that contains the tunnel. This overlying saturated zone must receive more than enough recharge from rivers, rainfall, or overlying formations to offset leakage through the bedrock zone and into the tunnel. The transition and soil zones of the Chattahoochee Tunnel are considered adequate to meet this requirement for the purpose of this analysis.

Goodman's assumption of a homogeneous, isotropic aquifer is unrealistic for hard-rock tunnels but is believed to be accounted for by Heuer's empirical reduction factor of 1/8. While Heuer's reduction factor has not been worked out mathematically, it seems reasonably consistent with the reduction in flow predicted by fracture-flow equations as opposed to porous-media equations, such as Goodman's. Further work needs to be done in this area.

### 2.2.2 Practical ranges of variables

The variables in Eq. 2 have practical ranges for tunnels. These practical ranges give insight into which are more important and which are less. In summary, K is the most important

term and hardest to estimate, $H_0$ is less important and easy to estimate, and $\ln(2z/r)$ is of minor importance and easy to estimate.

*$H_0$:* In most situations, the maximum for the static head ($H_0$) is the elevation difference between the tunnel and highest water table around the tunnel. The minimum is the elevation of the lowest water table around the tunnel. These values can be estimated readily from topographic maps and piezometers.

*$\ln(2z/r)$:* The rock cover above the tunnel should be known from borings. The tunnel diameter should be known from the design. Since both terms are in a logarithm, the practical range is quite small. For a 2 meter tunnel at 1000 meters deep, $\ln(2z/r) = 6.9$. For a 10 meter tunnel 30 meters deep, $\ln(2z/r) = 1.8$. This gives an extreme range between about 2 and 7.

*$K$:* In fractured rock, permeability ranges over many orders of magnitude within a given rock mass. This variability is difficult to predict. The practical minimum for tunnels is around $10^{-6}$ cm/s, because even with large heads over long distances, the amount of inflow is very small. The practical maximum, however, can range up to 0.1 cm/s or higher, depending on the rock conditions.

### 2.3 Heuer method

Heuer (1995, 2005) found that the actual water inflow into tunnel is generally significantly lower than the predicted water inflow using analytical equations and proposed an adjustment factor, on the basis of actual inflow measurements in various tunnels. The adjustment inflow rate is about one-eight of the inflow rate predicted from analytical solutions.

Although this factor is a significant improvement, it cannot be indiscriminately applied to tunnels under a range of conditions and needs to be modified to take into account the effect of depth, hydro mechanical interaction along rock mass discontinuities and other key geological features affecting the tunnel inflow rate

### 2.4 Lei method

Lei in 1999, derived analytical expression for hydraulic head, the stream function and the inflow rate of the two-dimensional, steady ground water flow near a horizontal tunnel in a fully saturated , homogeneous, isotropic and semi- infinite porous aquifer for a constant hydraulic head condition at the tunnel perimeter.

$$Q = \left( \frac{2\pi Kh}{\ln\left( \frac{h}{r} + \sqrt{\left(\frac{h}{r}\right)^2 - 1} \right)} \right) \tag{3}$$

Where Q is the groundwater flow, h is the head of water above the tunnel, K is the equivalent permeability and r is the radius of tunnel.(Fig.4)
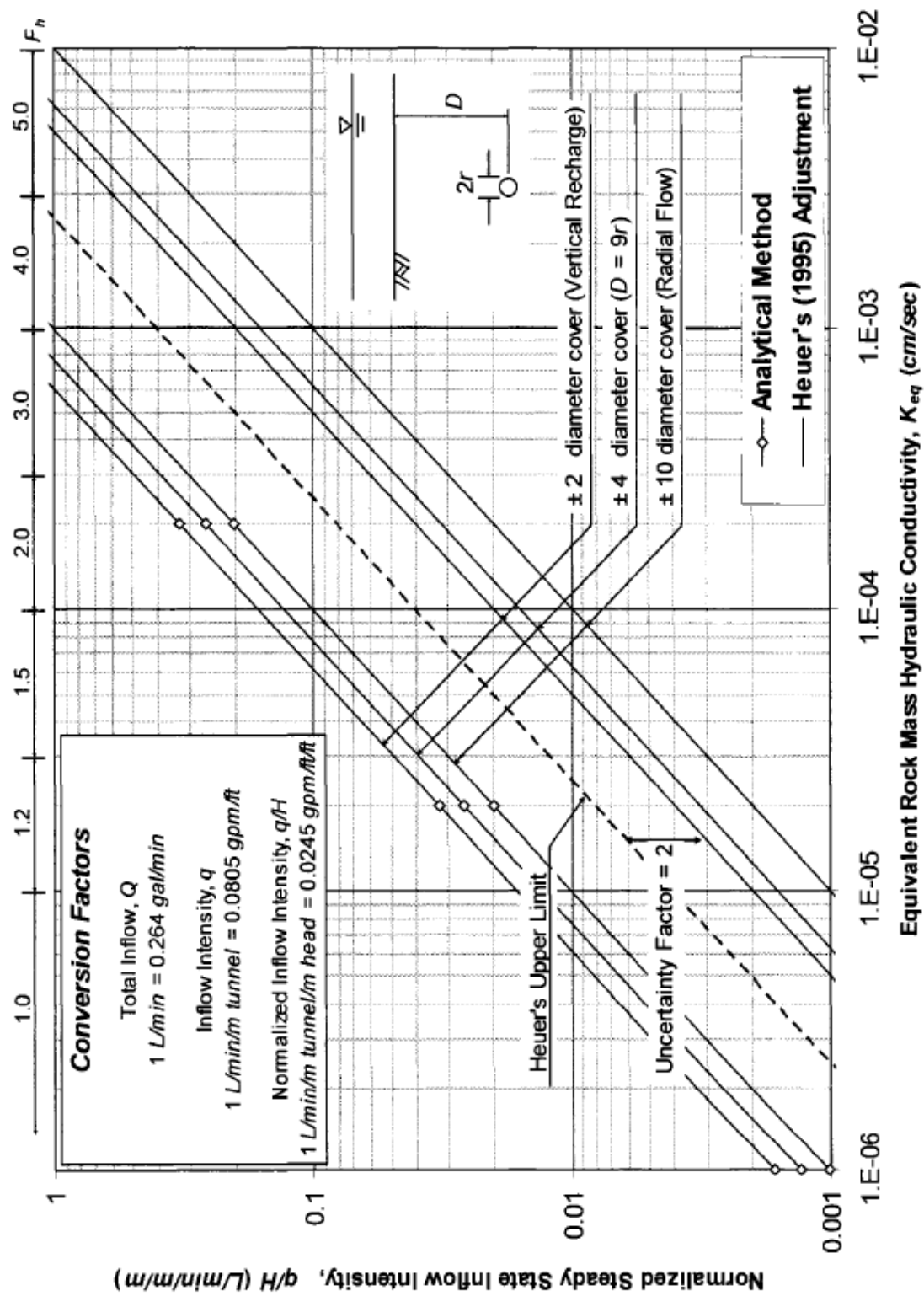
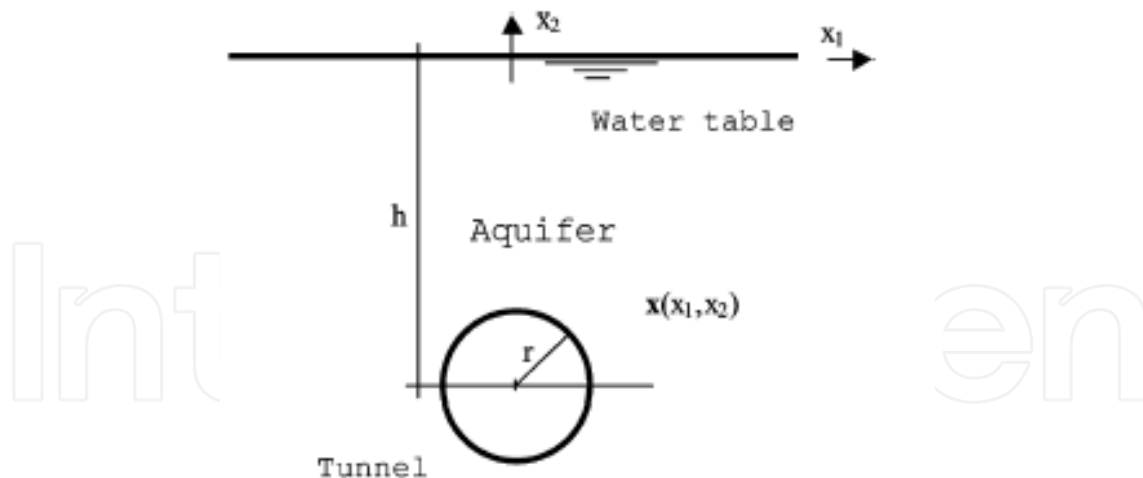Fig. 3. Relationship between steady state flow and equivalent permeability (Heuer1995).

Fig. 4. Circular tunnel in a semi-infinite aquifer with a horizontal water table.

This method deals with an ideal situation. And valid only for cases under the given assumptions. For more complicated scenarios, a numerical model would be more flexible.

### 2.5 El-Tani method

The steady gravity flow that is generated by a circular tunnel disturbing the hydrostatic state of a semi-infinite, homogeneous and isotropic aquifer is solved exactly. Many aspects of the flow are found in closed analytical forms such as the water inflow, pressure, leakage and recharging infiltration, which give a complete view of the aquifer in the drained steady state.

$$Q = 2\pi kh\left(\frac{1 - 3(\frac{r}{2h})^2}{\left[1 - (\frac{r}{2h})^2\right]\ln\frac{2h}{r} - (\frac{r}{2h})^2}\right) \tag{4}$$

Where Q is the groundwater flow, h is the head of water above the tunnel, K is the equivalent permeability and r is the radius of tunnel.

It is found that the maximum value of the recharging infiltration does not exceed the hydraulic conductivity allowing stating criteria for recharge intervention to ensure the stability of the aquifer. In addition to the main results, two aspects of the water inflow are treated. These are the necessary modifications that are to be considered in the case of an inclined water table and in the case of a lined tunnel that develops a constant internal pressure. It is also found that under an inclined water table a tunnel may cease to drain on its complete circumferential edge and a limiting condition is stated. Furthermore, the Muskat–Goodman and other water inflow predictions are compared to the exact gravity water inflow.

In this method The gravity flow that isgenerated by a circular tunnel is solved exactly. Other cases need to be obtained in closed forms such as the flow generated by non-circular tunnels in non-homogeneous and bounded aquifers. The integral formulation will probably be useful if it is adequately extended to these cases and to others in the three-dimensional space.

## 3. Tunnel site rating from groundwater hazard point of view(SGR)

Katibeh and Aalianvari(2009) using the experiments due to ten tunnels in Iran have been proposed a new qualitative and quantitative method for rating the tunnel sites in groundwater hazard point of view, named " Site Groundwater Rating" (SGR). In this method, the tunnel site, according to the preliminary investigations of engineering geological and hydrogeological properties, is categorized into six rates as follow: No Risk, Low Risk, Moderate Risk, Risky, High Risk and Critical. Considered parameters in this method are: joint frequency, joint aperture, karstification, crashed zone, schistosity, head of water above tunnel, soil permeability and annual raining.

In SGR method, after scoring each parameter, the SGR factor of the site is computed and according to this factor the tunnel site is divided into six categories. One of the advantages of this method is helping the engineers and contractors to design more suitable drainage systems and choosing the suitable drilling methods, according to the potential of the groundwater inflow, calculated in SGR.

In general, tunnel site are divided into two parts: tunnels in saturated zones and tunnels in unsaturated zones. Also with respect to the site lithology, tunnel sites are divided into rock sites and soil sites. The equation to compute the SGR factor is:

$$SGR = [(S_1 + S_2 + S_3 + S_4) + S_5]S_6\ S_7 \tag{5}$$

where parameters $S_1$ to $S_7$ respecting to the affecting parameters in groundwater inflow, will be described as below.

### 3.1 Score of frequency and aperture of joints, $S_1$

Massive rocks in tunnels alignment include one or more joint sets and tunnels cut them. Amount of water inflow into tunnels depends on joint frequency and joint aperture, so the representative parameter, $S_1$, is calculated using Eq.6:

$$S_1 = 25 \times (\sum_{i=1}^{n} \frac{\lambda_i\ g e_i^2}{12\nu}a) \tag{6}$$

where: $\lambda_i$, the joint frequency (1/m), $e_i$, the mean hydraulic joint aperture (m), g, the earth gravity (m/s²), $\nu$, the kinematic viscosity of water (m²/s), a, the unit factor (s/m) converting $S_1$ to dimensionless form.

The constant coefficient in Eq.6, 25, is obtained according to the experiments to normalize the parameter $S_1$.

The joint hydraulic aperture, $e_h$, in Eq.7, is different from the joint aperture estimated in surface. Cheng (1994) suggested the following equation to calculate joint hydraulic aperture in depth:

$$e_h = E - \Delta V_j \tag{7}$$

where, E is average joint hydraulic aperture in surface (mm) and $\Delta V_j$ is calculated using the following equation:

$$\Delta V_j = \frac{\sigma_n V_m}{K_{ni} V_m + \sigma_n V_m} \tag{8}$$

where, Cheng (1994) suggested $\sigma_n = 0.027Z$, according to his experiments, in which Z is overburden thickness. Moreover, analyses by Bandis et al. of experimental data indicated that the following relation was appropriate

$$V_m = A + B(JRC) + C(\frac{JCS}{E})^D \tag{9}$$

in which constants A, B, C, and D, according to the Priest's experimental researches are:
A= -0.2960, B= -0.0056, C= 2.241, D=-0.2450

JRC is joint roughness coefficient, JCS is joint wall strength and $K_{ni}$ is computed using Eq. 10.

$$K_{ni} = 0.02(\frac{JCS}{E}) + 1.75JRC - 7.15 \tag{10}$$

Finally, $S_1$ is calculated in dimensionless form. If the joint is filled with some materials such as clay, calcite, etc, then $e_i$ (Eq. 6) will be equal to zero. However, the caution must be taken for the joints filled with washable materials such as some clay types. Joints with washable materials can be identified in the supplementary site investigation during Lugeon tests.

### 3.2 Schistosity, $S_2$

Commonly, clay-base rocks are supposed to schistosity during tectonic processes, so that water can flow through schist planes. However, the relevant permeability is very less compared to the other discontinuities. In spite of low permeability, in SGR, the parameter $S_2$, representative of schistosity, is supposed in the range of 1 to 5, depending on the degree of schistosity.

### 3.3 Crashed zone, $S_3$

Crashed zones are the major path of groundwater flow through rock. Crashed zones considerably increase rock permeability; however this increase depends on the rock type. In clay-base rocks such as marl, shale, schist etc, clay minerals fill fractures and discontinuities resulting in considerable decrease in the permeability of crashed zone, but in the other rock types such as limestone, the permeability in crashed zone is very high. Moreover, the groundwater flow rate through crashed zones is related to the rock type and the crashed zone width. Considering rock type and crashed zone width, table 1 shows the equations to calculate $S_3$ in different rocks type in SGR.

| Type of rock | Crashed zone width | $S_3$ |
|---|---|---|
| Clay base rocks | Czw | $2 \times Log(10Czw \times b^*)$ |
| Other rock type | Czw | $100 \times Log(10Czw^{**} \times b)$ |

* b is the unit factor (1/m),
**Czw is the crash zone width (m)

Table.1. Method to estimate $S_3$ in crashed zones.

Crashed zones in both saturated and unsaturated zones are suitable paths for groundwater flow, thus, crashed zones are of most importance in SGR.

### 3.4 Karstification, $S_4$

Karstification is the geologic process of chemical and mechanical erosion by water on soluble bodies of rock, such as limestone, dolomite, gypsum, or salt, at or near the earth's surface. Karstification is exhibited best on thick, fractured, and pure limestone in a humid environment in which the subsurface and surface are being modified simultaneously. The resulting karst morphology is usually characterized by some types of cavities and a complex subsurface drainage system. So these cavities can conduct groundwater into tunnels. Groundwater inflow into tunnels can be very sudden and so dangerous. According to the degree of karstification, $S_4$ is estimated between 10 to 100.

### 3.5 Soil permeability, $S_5$

Parameters $S_1$ to $S_4$ are related to rock tunnels but if tunnel is excavated in soil, parameters $S_1$ to $S_4$ are automatically equal to zero. In SGR soil permeability is very important factor which is scored in $S_5$. The permeability of a clay layer can be as low as $10^{-10}$ m/s, of a weakly permeable layer $10^{-6}$ m/s and of a highly permeable layer $10^{-2}$ m/s.

Because of the direct relation between soil permeability and rate of groundwater inflow, in SGR, the score of soil permeability, $S_5$ is calculated as follow:

$$S_5 = K \times c \tag{11}$$

where, K is the soil permeability (m/day), c is the unit factor (day/m) converting $S_5$ to dimensionless form.

### 3.6 Water head above tunnel, $S_6$

Head of water (H) above tunnel is one of the most effective parameters on groundwater inflow into tunnels. The inflow equations such as Muskat–Goodman, Rat–Schleiss–Lei, Karlsrud and Lombardi indicate that groundwater inflow into tunnel has linear relation with H/Ln(H), so the representative parameter $S_6$, is calculated using Eq.12:

$$S_6 = \frac{H}{Ln(H \times d)} \times d \tag{12}$$

where, H is water head above tunnel and d is the unit factor (1/m) converting $S_6$ to dimensionless form. When tunnel is excavated above water table $S_6$ is equal to unit.

### 3.7 Annual raining, $S_7$

Just when tunnel is excavated in unsaturated zones, annual raining is effective on groundwater inflow into tunnels. In such case infiltrated water rain can seep into tunnel through fractures and faults. However, in such case groundwater inflow is not permanent like tunneling in saturated zones. Related to the tunnel depth, overburden permeability and length of water channel from discharging area up to tunnel, the time of reaching surface

water to tunnel is different. In unsaturated zones quantity and intensity of raining affect the groundwater inflow into tunnels, but here only annual raining is considered. $S_7$ for unsaturated tunnels is calculated using Eq. 13:

$$S_7 = \frac{P_y}{5000} \qquad S_7 \le 1 \tag{13}$$

where $P_y$ is annual raining (mm).

Maximum value of $S_7$ is when annual raining is equal to 5000 mm or more. When tunneling in saturated zone, $S_7 = 1$.

### 3.8 SGR factor

After calculating all the parameters, $S_1 - S_7$, SGR factor of the site is computed by means of Eq.5, then according to the value of SGR and using table 2, the tunnel site category can be found in six cases as: No Danger, Low Danger, Relatively Dangerous, Dangerous, Highly Dangerous, and Critical.

| SGR | Tunnel Rating | Class | Probable conditions for groundwater inflow into tunnel  (Lit/s/m) |
|---|---|---|---|
| 0-100 | No Danger | I | 0-0.04 |
| 100-300 | Low Danger | II | 0.04-0.1 |
| 300-500 | Relatively Dangerous | III | 0.1-0.16 |
| 500-700 | Dangerous | IV | 0.16-0.28 |
| 700-1000 | Highly Dangerous | V | Q>0.28 ،Inflow of groundwater and mud from crashed zones is probable. |
| 1000< | Critical | VI | Inflow of groundwater and mud is highly probable. |

Table 2. SGR rating for groundwater inflow into tunnels.

Experiments due to 10 tunnels show that there are direct correlations between SGR factor and groundwater inflow rate into tunnels. In case of high SGR factor (more than 700), mixture of mud and groundwater is probable to rush into tunnel, endangering persons and equipments.

In the other hand, with attention to SGR factor, groundwater inflow into tunnel can be predicted, which help to plan suitable drainage systems and even to choose the best drilling method.

## 4. Discussion

This chapter is concerned with groundwater flow approximation methods into hard-rock tunnels in fractured rock that are excavated below the water table. Several analytical equations to calculate groundwater discharges into tunnels, including Goodman(1965), Lohman(1972), Zhang(1993), Heuer (1995), Lei(1999), Karlsrud(2001), Raymer(2001) and El Tani (2003) were introduced here, along with SGR method.

El Tani (2003) compared the results of above mentioned methods with the observed seepage into tunnels, for different values of r/h. Table 1 contains a listing of diverse approximations of the water inflow including Muskat–Goodman, Rat–Schleiss–Lei, Karlsrud, Lombardi and El Tani methods. The relative differences of the diverse formula of table 1 with the exact (observed) water inflow (Q) are shown in Fig. 2 and are computed with:

$$\Delta = \frac{Q_{ap} - Q}{Q}$$

in which, $Q_{ap}$ is a water inflow approximation.

| | |
|---|---|
| **Goodman** | $Q = 2\pi K \dfrac{h}{\ln(\dfrac{2z}{r})}$ |
| **Karlsrud** | $Q = \dfrac{2\pi K H_o}{\ln(\dfrac{2H_o}{r})}$ |
| **Heuer** | $Q_L = \dfrac{2\pi K H_o}{\ln(\dfrac{2z}{r})} \times \dfrac{1}{8}$ |
| **Lee** | $Q = 2\pi K \dfrac{h}{\ln(\dfrac{h}{r} + \sqrt{(\dfrac{h}{r})^2 - 1})}$ |
| **El Tani** | $Q = 2\pi K \dfrac{1 - 3(\dfrac{r}{2h})^2}{[1 - (\dfrac{r}{2h})^2]\ln(\dfrac{2h}{r} - (\dfrac{r}{2h})^2}$ |

Table 3. Diverse approximations for the water inflow.
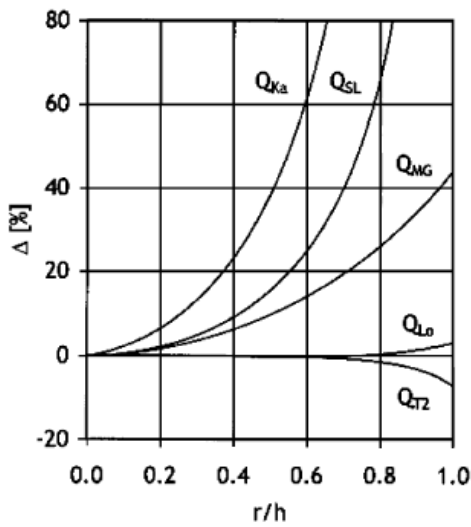


Fig. 5. Relative difference of the diverse approximations in Table 1with the exact water inflow.

As the Fig. 5 shows, with decreasing r/h, water inflow approximations converge each other and converge to the exact water inflow. For r/h less than 0.3, the relative differences are negligible (El Tani, 2003). For large values of r/h, means tunnels near to the Groundwater table, Lombardi and El Tani approximations are more close to the exact water inflow, while, the relative differences of other equations are considerable in this case (Fig. 5).

A new qualitative and quantitative method for tunnel site rating in groundwater hazard point of view (SGR), was introduced in this chapter. In this method, the tunnel site, according to the preliminary investigations of engineering geological and hydrogeological properties, is categorized into six rates as follow: No Danger, Low Danger, Relatively Dangerous, Dangerous, Highly Dangerous, and Critical. (Katibeh, Aalianvari, 2009). Using SGR factor, groundwater inflow into tunnel can be estimated, which helps to predict suitable drainage system and to choose the best drilling method.

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**Drainage Systems**

Edited by Prof. Muhammad Salik Javaid

The subject of 'drainage: draining the water off' is as important as 'irrigation: application of water', if not more. 'Drainage' has a deep impact on food security, agricultural activity, hygiene and sanitation, municipal usage, land reclamation and usage, flood and debris flow control, hydrological disaster management, ecological and environmental balance, and water resource management. 'Drainage Systems' provides the reader with a tri-dimensional expose of drainage in terms of sustainable systems, surface drainage and subsurface drainage. Ten eminent authors and their colleagues with varied technical backgrounds and experiences from around the world have dealt with extensive range of issues concerning the drainage phenomenon. Field engineers, hydrologists, academics and graduate students will find this book equally benefitting.

**How to reference**

In order to correctly reference this scholarly work, feel free to copy and paste the following:

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