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Tunnel Inflow Monitoring in Permeable Rocks Nearby Cannero River, Piedmont (Northern Italy)

Loris Colombo[1], Daniele Cremonesi[1] & Vincenzo Francani[1]

[1] D.I.I.A.R Department of Environmental, Hydraulic, Infrastructures and Surveying Engineering, Politecnico di Milano, Milan, Italy

Correspondence: Loris Colombo, D.I.I.A.R Department of Environmental, Hydraulic, Infrastructures and Surveying Engineering, Politecnico di Milano, Piazza Leonardo da Vinci 32, Milan 20133, Italy. Tel: 39-223-996-669. E-mail: loris.colombo@polimi.it

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Abstract

Analytical methods used in engineering practice for a preliminary tunneling design do not adequately account for the domain limitation due to a river. In order to forecast the flow rate that has to be extracted to reach a sustainable water head in tunnel, the design must consider the changes of river hydrometric level and their consequences on piezometric head. The river is taken in account and the constraints of analytical solutions are discussed assuming isotropic and infinite domain. In particular, the study develops a formulation considering the variation of hydrograph in the river and different location of tunnel in the aquifer nearby the stream. Then, this equation was applied to a preliminary tunneling design in Cannero Valley, Northern Italy (Piedmont). The results, compared with analytical model's ones show that the analytical approach leads to a good estimation of the tunnel inflow and, the analytical to a preliminary evaluation of its most suitable location.

Keywords: analytical solutions, analytical simulation modeling, groundwater flow, tunnel inflow

1. Introduction

Several mathematical relationships have been developed to describe the drainage effects of tunnels on rivers consisting of the withdrawal of river and massive inflow of large volumes of water in the tunnel. For this reason heavy changes of piezometric head can occur opening a tunnel in saturated aquifer. The shrinkage of rivers discharge may even lead to their disappearance or to exceed the minimum vital flow.

Moreover, usual mathematical solutions are not suitable for practical cases least of all when occur complex geological settings.

Then, it is analyzed the suitability of a solving technique that can allow to predict tunneling extraction flow nearby river even if it is a first approximation.

For this purpose, the study is based on the project of a small sized tunnel in an alpine valley. The tunnel is located nearby Verbano Lake (Maggiore Lake) and crossed by Cannero River (Piedmont District, Northern Italy), affected by sudden floods that can propagate about some meters remarkable fluctuation to groundwater.

Two cases have been considered:

- Different tunnel locations near river with the same distance but different depths
- Different tunnel located at very low depths but with increasing distances from the river.

The results obtained with the application of the proposed technique, have been compared to ones with an analytical model such as GFlow (Haitjema, 2005). This calculation code is been subsequently used for the evaluation of loss of river, in order to identify the most suitable location for the tunneling design. There is a good overlap by comparing analytical solution developed and analytical model solution even if the present work is a sort of preliminarily solution for the most complex settings such as heterogeneous aquifer.

2. Definition of the Problem and Actually Knowledge

The construction of a tunnel, even if small sized, produces a draining effect which causes water-table drawdown in a different way, extinction of springs or wells (Gisotti & Pazzagli, 2001), changes in groundwater quality (Civita et al., 2002), changes in vegetation and slope stability conditions (Picarelli et al., 2002), and in the hydrological balance at basin scale (Gattinoni & Scesi, 2006). For this reason it needs to propose a forecasting method in order to predict, in approximate way, the interference of a tunnel on the shallow aquifers.

Kawecki (2000), Odeh and Babu (1993), Goode and Thambynayagam (1987), Dunning et al. (2004) obtained analytical solutions for the transient state for horizontal drainage system similar to the tunnels but in an infinite domain without boundary conditions. Therefore, the relationships are not useful for direct application in reality.

Analytical solutions account for an important role in the first and quick estimation of tunnel-inflow. Recently, numerical codes such as Modflow (EMS – Environmental Modeling System, McDonald & Harbaugh, 1984) have substituted analytical ones; they can help to analyze a more detailed situation, as the fact that analytical solutions cannot take in account of the randomness of the main hydrogeological parameters.

A large number of studies in recent years have underlined the usefulness of the hydrogeological survey during tunneling design also for the prediction of water inflows (Goodman et al., 1965; Ribacchi et al., 2002).

For the purposes of the study, the conceptual model must consider also a river that can greatly affect the flow arriving at the tunnel. This importance has already been demonstrated in numerical modeling studies of Dunning et al. (2004), about the Menomonee Valley tunnel. The Authors try to explain two cases: in the first the tunnel does not interfere with the river (Figure 1A), for the reason that it is located at a huge depth, in the second (Figure 1B) where the tunnel causes a river drainage, which draws water.

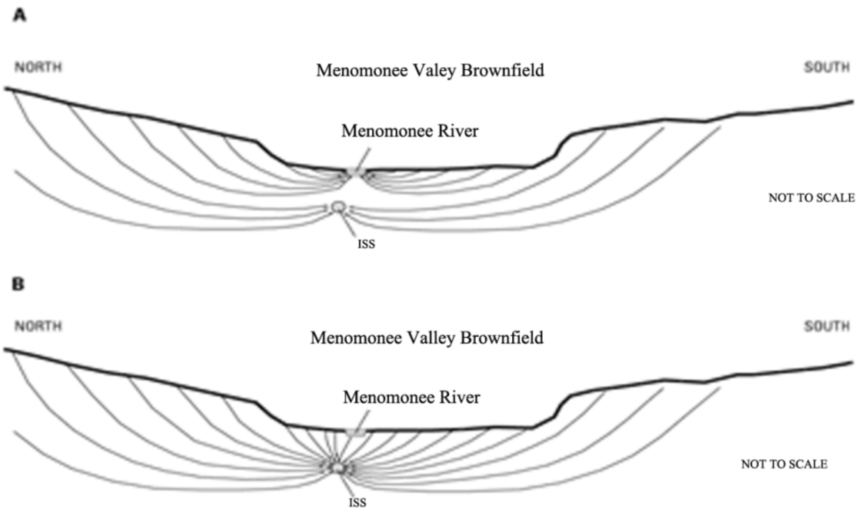

Figure 1. Menomonee Valley Brownfield, Milwaukee County, Wisconsin conceptual model (by Dunning C.P. et al., 2004)

In the present study, first of all, an analytical solution has been found and tested for the prediction of drawdown induced by a tunnel of a small diameter in the adjacent aquifer, even in presence of a water river. The analytical solution can also give an estimation of the flow extracted in order to maintain waterless the tunnel. Secondly, it is underlined that the flow rate taken from the river surface is influenced by the distance of tunnel from the river and by its depth from the water surface. An analytical model has been used.

In order to describe the method which was used to achieve these two goals, the effects of a shallow tunnel nearby Cannero river useful for a water system have been provided to predict. Two different configurations have been considered: a tunnel located at the same distance and at different depths (Figure 2, case 1) and a tunnel located at different distances from the stream with the same depth (Figure 2, case 2). The analytical solution must be able to provide the entity of flow rate and drawdown in the center line of the tunnel. Analytical models (Haitjema, 2005) are used for the comparison of the relationships modified by the authors (case 1-2).

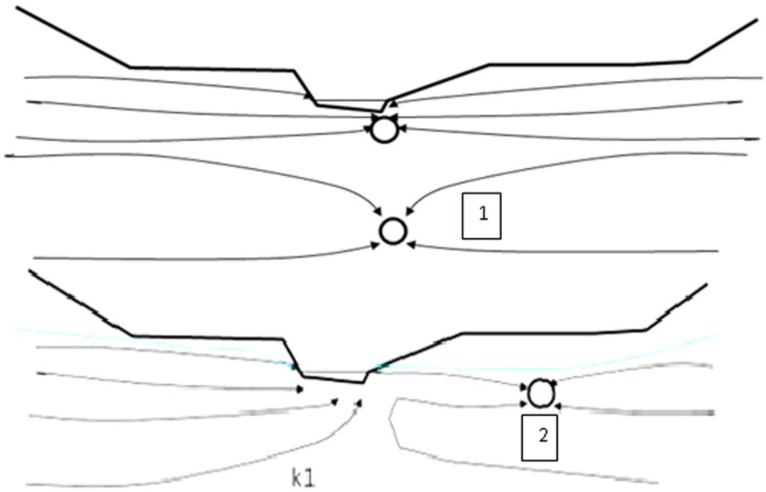

Figure 2. The two cases studied in this note: 1) tunnel located at a short distance from the Cannero river and at different depths respect to the bedrock; 2) tunnel located at a low depth with increasing distances from the Cannero river

For the first case, the Goode et al. (1987)'s relationship, valid for a tunnel located at a distance z_w from the bedrock partially penetrating a confined aquifer (Figure 3) has been used.

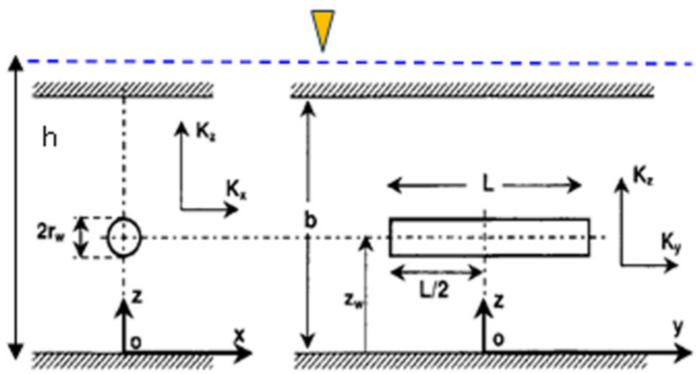

Figure 3. Partially penetrating tunnel modeling in a confined aquifer (Goode & Thambynayagam, 1987). The piezometric level is represented in a blue line

This relationship (1.1) depends also on the location of tunnel due to the different depth (Goode & Thambynayagam, 1987)

$$s = \frac{Q}{2\pi T}\ln\left(\frac{1.5\sqrt{\frac{Tt}{S}}}{x}\right) + \frac{Q}{2\pi LK}(\sigma_z + \sigma) \tag{1.1}$$

where *b [m]* is the thickness of the aquifer, , *S [-]* is the storage coefficient, *L [m]* is the tunnel length, *K [m/s]* is the aquifer hydraulic conductivity, *T* $[m^2/s]$ is the transmissivity, σ *[-]*is the skin factor (about one value) that depends on any change of the hydraulic conductivity nearby the tunnel, σ_z *[-]*is the skin tunnel factor due to the loss of the head pressure in the vertical flux. The last one is expressed by

$$\sigma_z = ln\left[\frac{b}{2\pi r}\frac{1}{sin(\frac{\pi z_w}{b})}\right] \tag{1.2}$$

The (1.2) is in function of the radius r *[m]* of the tunnel. It must be also a function of the distance from the river and of its variable rate in time.

3. Cannero Pre-Alpine Valley: A Description of Study Area

In order to show the necessity to adapt theoretical relationships to the geological complex settings, it is useful to propose an example of application of the (1.1) to the real design of a small tunnel (radius about 1.5 m), but large enough to avoid situations of critical speed around the tunnel cable. The (1.1) is of great interest theoretically, but can be observed that its application for the real case of Cannero valley has some problems. Indeed:

a) It cannot represent a groundwater subjected to piezometric fluctuations due to the proximity of the river, that under condition of flash floods may influence the regime of flow in the tunnel;

b) It cannot represent the variability of flow rate also due to the fluctuations in the stream.

In the tunnel installation area, the bedrock consists of metamorphic rocks such as mica schist and paragneiss rather sheared in surface and covered with a layer of debris consisting mainly of sandy silt with pebbles and stones, whose behavior can be approximated to a granular soil. The debris' thickness varies from a few centimeters to several meters.

It is therefore in presence of a homogeneous confined aquifer whose parameters are summarized in Table 1 as a result of a monitoring survey of the hydrogeological values. The effect of the tunnel has evaluated, at depth ranging between 6 and 14 m paying attention to the position of the bedrock respect to the tunnel base. The arrangement of the tunnel is oriented so that it is parallel to the river and iso-piezometric lines as shown in Figure 4.

Table 1. Parameters used for the computation of flow rate Q *[m³/s]* and hydraulic head h *[m]*

T aquifer *[m²/s]*	0.02
S *[--]*	0.2
Aquifer porosity *[--]*	20%
Aquifer thickness b *[m]*	20
R tunnel *[m]*	1.5
Tunnel length L *[m]*	100

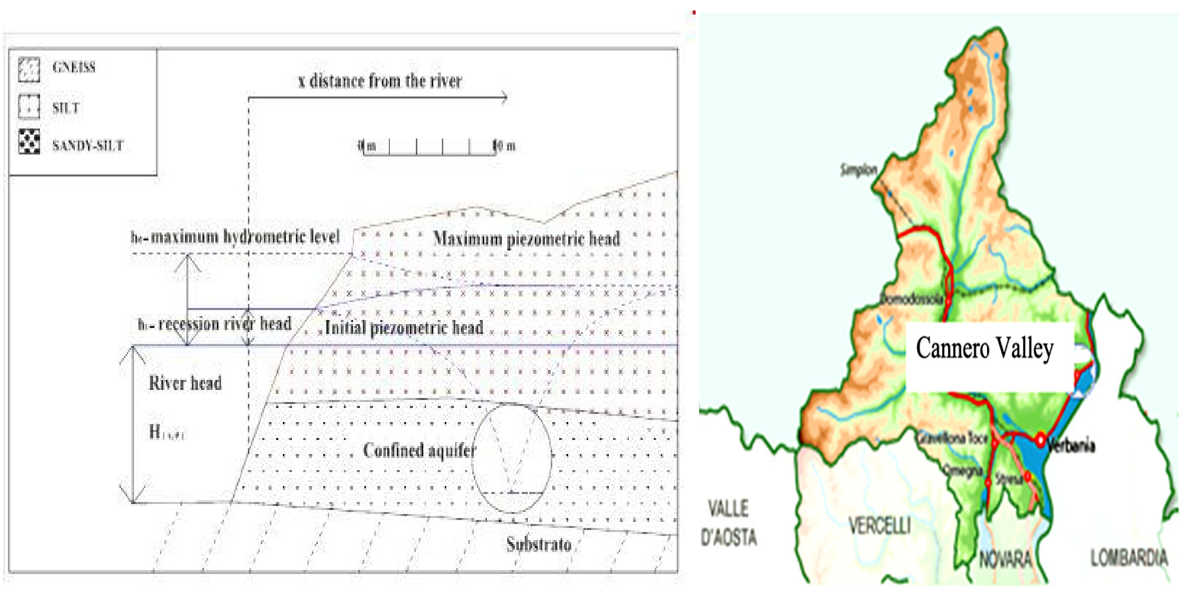

Figure 4. Piezometric behavior due to a tunneling design at a distance x from the river (a) and localization of Cannero valley (b)

4. Analytical Modeling of the Tunnel Inflow

The distance *x* between the tunnel parallel to the river and the river itself, following the indication of Milojevic (1963) and Raghunath (1987), has been taken in account in order to obtain a finite domain and consider the presence of river; also, the method of images has been applied, considering a symmetric tunnel respect to the river This symmetric tunnel produces a drawdown named *s'*. The difference between the drawdown, obtained by the original drain and its specular one, is obtained applying the following relationship (1.3):

$$s - s' = y - y' + \frac{Q}{2\pi T} ln\left(\frac{x'}{x}\right) \tag{1.3}$$

The head change causes for each time step a volume variation as a flow rate in input to the tunnel relationship where *H [m]* is the hydrometric level on the river during flood; *h [m]* depends on hydraulic head aquifer fluctuation at the distances x from the river without tunnel for each time step. This last one is obtained with the Pinder et al. (1969) relationship:

$$h = \sum_{m=1}^{p} \Delta H_m \left\{ erfc \frac{u}{2\sqrt{p-m}} \right\} \tag{1.5}$$

where h *[m]*is the head at a x distances from the river at *pΔt* time, *pΔt* is the total time from the beginning of the flood and *u* is

$$u = \frac{x}{\sqrt{\frac{T}{S}\Delta t}} \tag{1.6}$$

Considering a real impulsive and very fast hydrograph, typical for the regions of pre-Alpine valleys, the piezometric level is as higher as nearer the water river. Increasing distance from the river the groundwater level becomes about some meters lower as shown in Figure 5 . This fluctuation depends on variation of river floods during time.

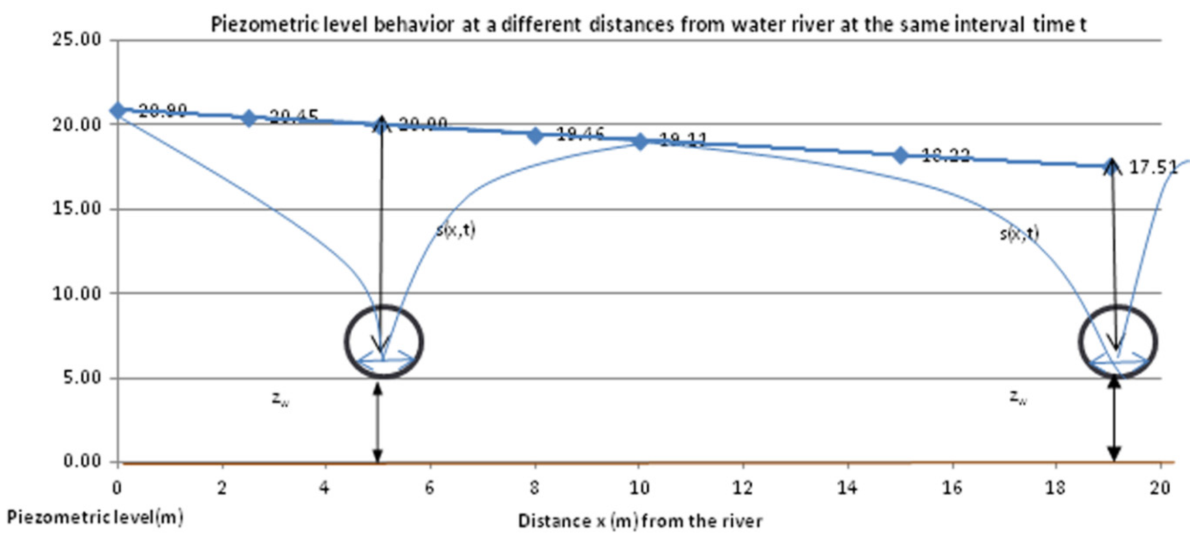

Figure 5. Piezometric level at the same interval time t at different distances from the river during flood. The constrained drawdown in the tunneling centerline becomes lower and lower when the tunnel is located at a higher distances from the river, or at different piezometric head

If, for simplicity, it is assumed that this flow is totally drained by the system of the tunnel and that maximum drawdown occurs to the tunneling centerline, it is possible to compute the drawdown induced to the groundwater level during transient state on the tunneling centerline.

First applying (1.4) to (1.1)

$$s = \frac{Q(t)}{2\pi T} \ln\left(\frac{1.5\sqrt{\frac{Tt}{S}}}{x} \right) + \frac{Q(t)}{2\pi LK}(\sigma_z + \sigma) \tag{1.7}$$

The (1.7) shows that the flow rate depends not only on the hydrograph variation but also on fluctuation of the connected groundwater computed with the (1.5). Applying the (1.7) to (1.3) the drawdown is due to

$$s(x,t) = \frac{Q(t)}{2\pi K}\left(\frac{1}{b} + \Lambda \right) ln\left(\frac{x'}{x} \right) \tag{1.8}$$

The correction factor Λ *[m^{-1}]* introduced into (1.8) is similar to σ_z skin factor divided into length of tunnel L. This last relationship leads to obtain the tunnel drawdown in function only of piezometric oscillations due to the river flood located at a x distance.

In the real case, it must be necessary to know the outflow of a sufficiently high flow rate which allows the tunnel not to be under the water level. In practical situation, then it should have a safety height approximately 0.5 m from the bed of the tunnel. The flow rate is determined by a simple procedure in reverse, forcing drawdown wanted in the center line of a tunnel. From this condition it is possible to obtain the drained flow which is always a function of oscillation of the river. The condition imposed at the symmetric axis of the tunnel can be obtained using the following equation

$$s(t) = h(t) - z_w - f \tag{1.9}$$

where the drawdown is dependent not only on time but also on a hydraulic head (1.5) computed in the tunnel center line by Pinder et alii, 1969. There are also on the relationship the aquifer thickness b *[m],* the height difference z_w *[m]* of the tunnel respect to the bedrock and the value f *[m]* of water height in tunnel. This drawdown depends on Pinder's computed hydraulic head and it is as lower as farther by river as shown in Figure 5.

The linearity of (1.8) allows to a simple inversion for computation of flow rate in function to the drawdown and to the time for having a lowering hydraulic head similar to the submerged level. Applying the 1.8 any interesting consequences can be obtained, as shown in Figure 6 and Figure 7

5. Results and Discussion

5.1 Analytical Formulation between Depth and Flow Rate Inflow

It is useful to extract the relationship between depth and flow rate to extract. Then, through the use of the above relationships, a computation highlighting the way it has a substantial reduction of the inflows to the tunnel by reducing the depth of the tunnel has been made. In this regard, two different effects have been observed: the first one is related to the difference in height of the tunnel with respect to the bedrock, the second one is related to the variation of the distance from the river.

The Figure 6 shows the extract flow rate in function to the depth in a time interval and for different distances from the Cannero river.

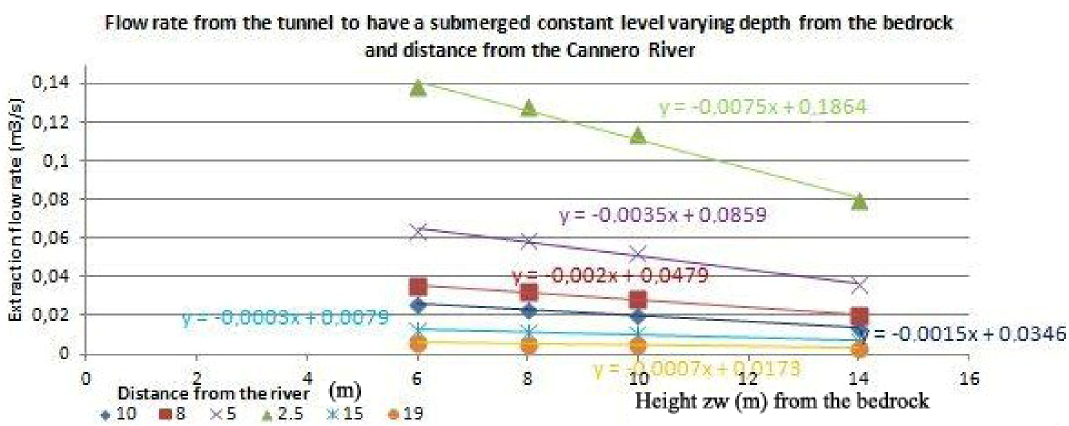

Figure 6. Flow rate behavior in function to the depth (zero values means the bedrock position) at the same interval time t (Figure 2, case 1)

Figure 6 shows that flow rate decreases about two magnitude order due to the river distances because imposed drawdown becomes lower as head level h *[x,t]* decreases; the flow rate depending on the tunneling depth is similar for all the studied cases (from green line to orange line) : the flow rate becomes about twice decreasing depth from 6 to 14 meters. This relationship is similar to Dupuit one for vertical wells where flow rate and drawdown are directly proportional.

Figure 5 shows that with same distance from the river, the flow/ depth from bedrock ratio is constant. This can underline that flow rate tends to increase linearly with depth.

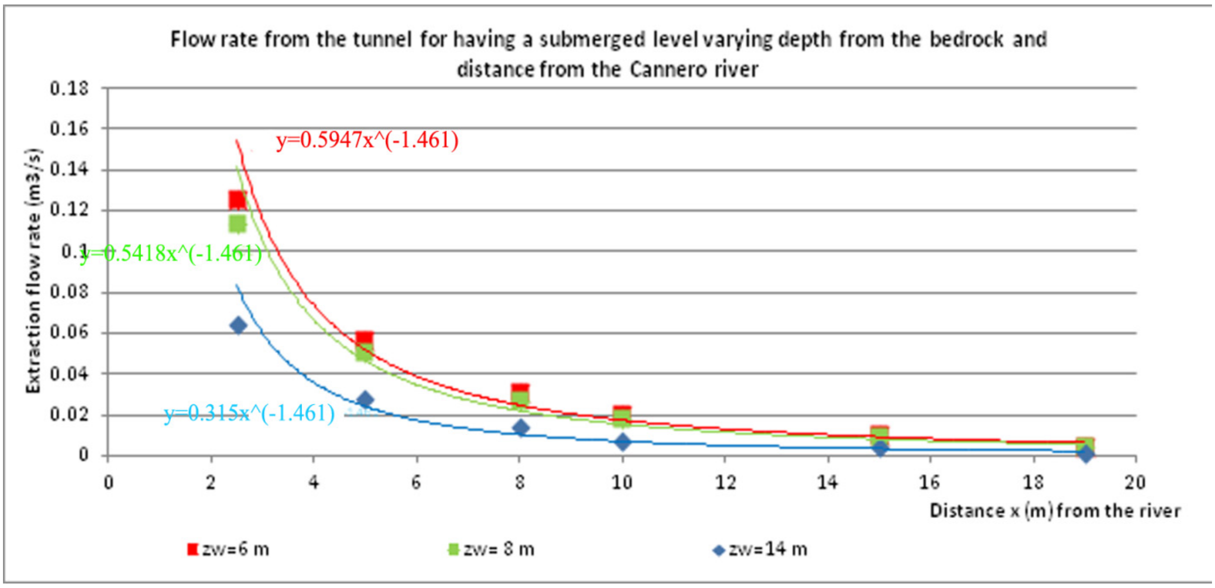

Figure 7. Flow rate behavior for a tunnel located in a different distances from the river and at different depth (Figure 2, case 2)

Figure 7 shows that the trend of the flow rate is at a constant depth changing the distance on the river is similar to an exponential type function. This function tends to zero value as the tunnel is designed far away from the stream since the changing water volume induced by the flood has not a huge influence on the flow regime. The closer the tunnel is to the river, the more the function diverges to infinity: in the center of the river, the flow rate tends to infinity for the logarithmic nature of (1.8). The similarity between the different behavior can be observed by exponential equations whose index is similar and the coefficient A is increasing with increasing $z_{w.}$

The drained flow from the tunnel is a function of the fluctuations recorded on the hydrograph of the Cannero river, it follows the trend with virtually no delay: an oscillation of the flood spread in the aquifer determines simultaneously a variation of the flow in the tunnel. It propagates with the same trend as the flood the more the tunnel is located in proximity of the river and at relatively low depths. Instead, at higher distances the flow rate necessary to the safety level is lower even if the value is always dependent on the tunneling depth because computed drawdown is even some meters (Figure 5).

5.2 Comparison between Empirical Analytical Results and Analytical Haitjema Model

Results obtained by (1.8) have been compared to the results of an analytic model, GFlow (by Haitjema, 1995). The software has developed to describe water flux on a horizontal plan, using Dupuit – Forchemeier's approximation. However, as it is shown by Haitjema (2005), GFlow can be used also to a model vertical sections of one or more aquifers.

A model of an aquifer with unitary depth, limited both on the upper and the lower side, by "no-flow" conditions, has been developed. Boundary conditions have been introduced to describe the hydraulic head, using the function of "line – sink" considering as "far – field"

The Cannero River has been inserted as "line – sink, far – field", with a variable hydraulic head to simulate the flood, previously modeled by the method proposed by Pinder and revised by Colombo et al. (2011), and the same values for parameters in Table 1.

The tunnel has been introduced as "line – sink" tunnel, setting the initial head value equal to the one measured in the River (20 m a.s.l.), and the final head set in order to maintain a dry condition in the tunnel (the measured head is calculated adding to the z position of the basis of the tunnel a 0.5 meters safety – level). The value of safety level is variable in function of the position of the same tunnel respect to the substratum (the depth of the tunnel varies from 13.5 m to1.5 m, where are located the head – target measurement points).

Comparing the results obtained in the first case, when the axis of the tunnel (tangent to the river) is 4.7 m far from the river bank (Figure 2), for increasing depths, the obtained results are shown in the Figure 8.

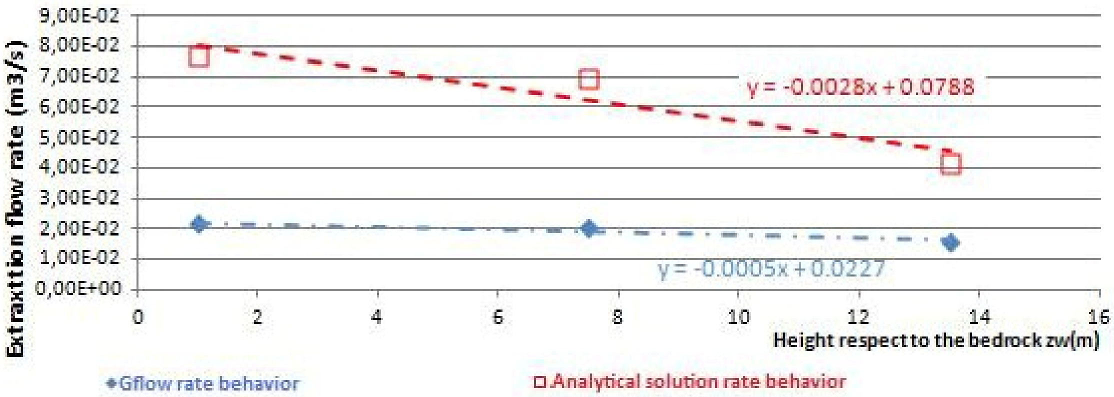

Figure 8. Flow drained behavior by the tunnel in order to respect the safety level of 0.5 m, for decreasing depths from the substratum (0 value indicates the bedrock)

The value obtained with the analytical model fits better the values computed nearby the bedrock (the angular coefficient of the regression curve is approximate 0, to indicate the sensibility of the model to the presence of the bedrock). The analytical formula (1.8) only considers the height of the bedrock instead of hydraulic conductivity: this approximation is shown in the previous figure, when the flow values are the more different, the closer the tunnel is to the bedrock.

The second case (Figure 9), instead, shows the flow, measured by a target set 0.5 m from the basis of the tunnel (positioned 17.5 far from the substratum), function of the distance by the river. The results are similar to the analytical model (1.8), so the two models are comparable. There is a small divergence approximating to the river, due to the logarithmic nature of the (1.8), and this kind of functions diverges to 0 (which represents the position of the river).

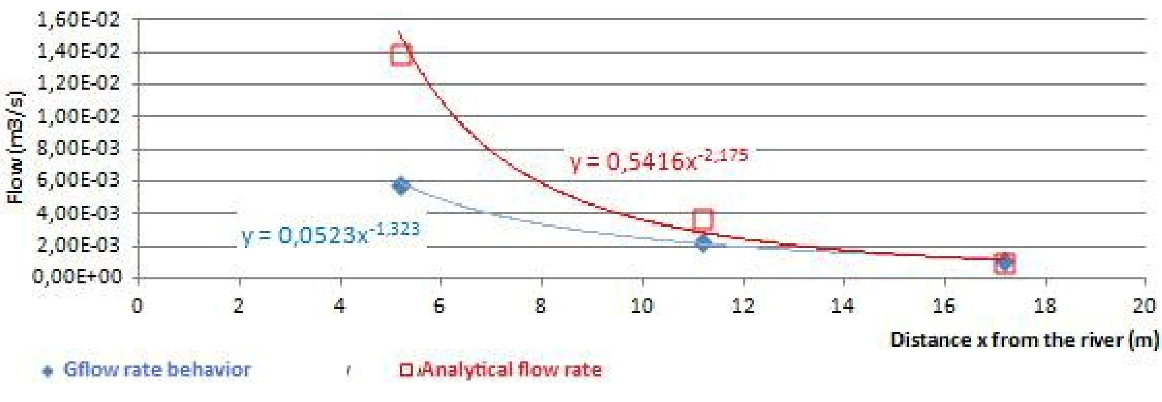

Figure 9. Flow drained by the tunnel in order to respect the safety level of 0.5 m, in function of different distances from the river (from 5.17 to 17.17 m) and comparison to the analytical solutions of the problem, obtained by (1.5)

5.3 Estimation of Tunnel Inflow nearby Cannero River

The second purpose of the study is to evaluate the discharge from the river to the tunnel. This objective has been reached using the Haitjema's analytical model (2005). The results obtained by the (1.8) show that the flow increases with the depth of the tunnel and with the proximity to the river, but the same (1.8) does not allow to compute the flow subtract to the river. The Haitjema model has been used to help to find the optimal position of the tunnel near the Cannero River. The tunnel has been positioned in two different places: nearby the river, with increasing depths (test set in Figure 10), or far from the river, with increasing depths (test set in Figure 11).

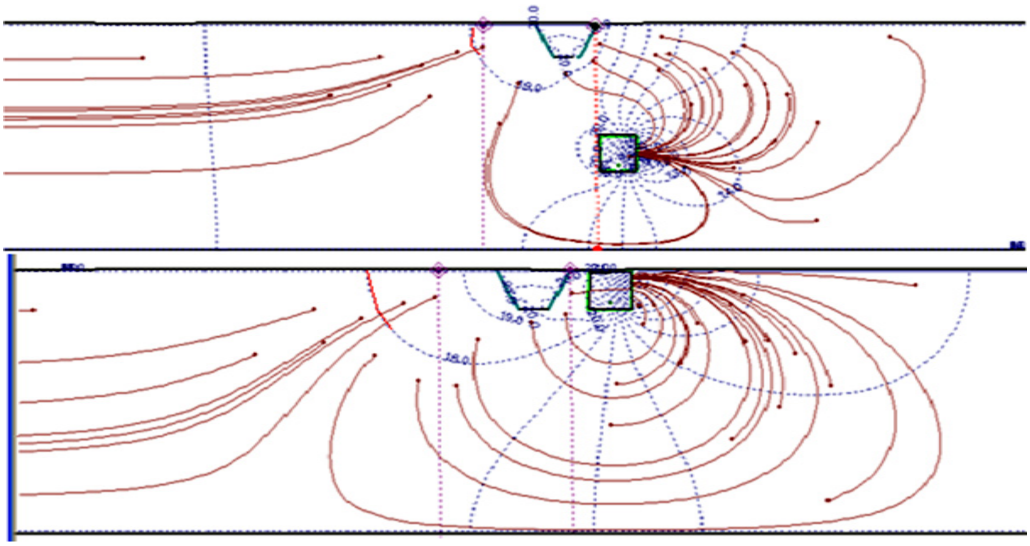

Figure 10. Representation of the river flux not drained by the tunnel (dot red line). In the case of profound tunnel near the river, the flux not drained is bigger than the case of the tunnel near the surface (as shown by the violet particle lines)

The model allows to calculate the flux towards the tunnel using the function "Inspector flux". In the case of profound tunnel, the estimated flux is 3.61 m^3/day, whereas for the superficial tunnel it is equal to 4.08 m^3/day. So, the flow coming by the river is quite important to control the drainage of the tunnel.

In fact, when the tunnel is far from the river, its position respect to the substratum is indifferent, because the effect of the river on the drained flow (about 3.53 m^3/d for the superficial tunnel and about 3 m^3/d for the depth one).

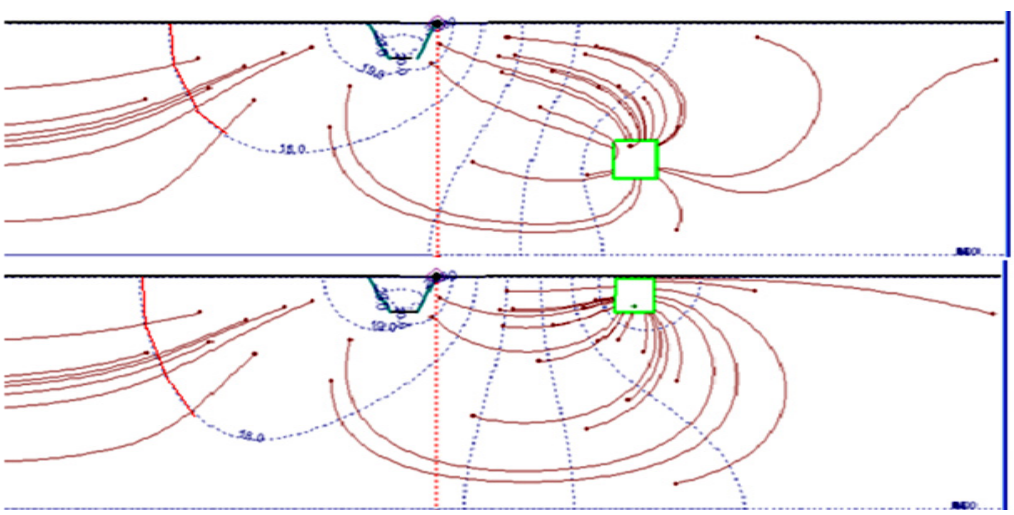

Figure 11. Flow rate from the river not drained by the tunnel (dot red line). In the case of tunnel far from the river, the drained flux is similar both if the tunnel is near the surface and near the substratum (as shown by the violet particle tracking lines)

6. Conclusion

Dunning et al., 2004 have shown as a river presence can influence the tunneling inflow behavior depending on its nearby aquifer location, such as river distance and impermeable bedrock depth. Consequently, the proper model for computation must take in account both river oscillations (Pinder et al., 1969) and bedrock depth respect to the tunnel base (Goode et al., 1987), in addition to the tunnel and aquifer properties.

The study presented a possible solution with a good approximation for the computation of the tunneling extraction flow located nearby a water river using (1.8). This relationship is similar to the Dupuit's vertical well formulation for hydrogeological parameters and for its behavior. Instead, from (1.8) the s/Q ratio at the same distance from the river is $\frac{s(x,t)}{Q(t)} = \frac{1}{2\pi K}\left(\frac{1}{b} + \Lambda\right) ln\left(\frac{x'}{x}\right)$ and inflow rate.

If the second part of this relationship as it is composed by constant parameters, is indicated with A, it can be concluded that for the same distance from the river the s/Q ratio is linear with different depths respect to the bedrock. For the (1.9. the s/Q ratio is also valid for the ratio between tunneling depth and inflow rate.

If A' is s'/Q' ratio at a different distance from the river, it can be seen that A/A' is strictly dependent on logarithmic tunnel distance function. In this way, the Figure 6 shows that for different tunnel location from the river, the flow rate/depth ratio becomes lower with increasing the distance from river. This coefficient depends on x-tunneling location at a constant depth. The flow necessary to have a fixed piezometric head in centerline becomes lower and lower with increasing distances from the river. This is due to the fact that, referring to Figure 5, drawdown is lower when distance from the river icnreases.

The computed flow takes in account of tunneling centerline drawdown due to the (1.9) (depending on both groundwater fluctuations in space and in time and safety level constraint). Sato (1983) demonstrates with some experimental studies that in the initial flow the piezometric depression is highest in the tunneling vertical centerline (Figure 12 and Figure 13). The obtained flow rate (Figure 6 and Figure 7) trough the (1.8) show that higher values depend on closed distance to the river (Figure 6) and on impermeable bedrock depth (Figure 7).

Results obtained applying the (1.8) have been compared with an analytical model Gflow (Haitjema H. M., 2005) obtaining similar results. For the result obtained in the first case (Different tunnel locations near river with the same distance but different depths) for both model a linear behavior can be observed which is more remarkable near the river. Also in the second case the exponential asymptotic behavior due to the increasing distances from the river can be observed for both models.

Figure 14 and Figure 15 show, in fact, that is possible to identify the non – drained zones, and it is possible find out that the optimal location for the tunnel is nearby the surface, but far enough from the river in order to have a limited inflow in the tunnel. If the tunnel is nearby a water river, it is important to underline that, if the location of tunnel is designed in depth, the river causes a limited influence.

The (1.8) is a good approximation for a first estimation of water drained volume magnitude by a linked tunnel to the river. The solution has been obtained considering a total lowering of the hydraulic head of the aquifer in tunneling centerline; it is limited by the fact that nearby the river the estimated flow rate tends to infinite value for the logarithmic behavior.

The analytical model allows to find out the tunnel inflow and to observe quickly if the river feeds tunnel during flood or ordinary flow. Considering the fact that the analytical model can give a fast evaluation about volume of water drained from the river, the analytical model, useful also for heterogeneous aquifer, is a good tool for designing analysis of planning solutions.

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