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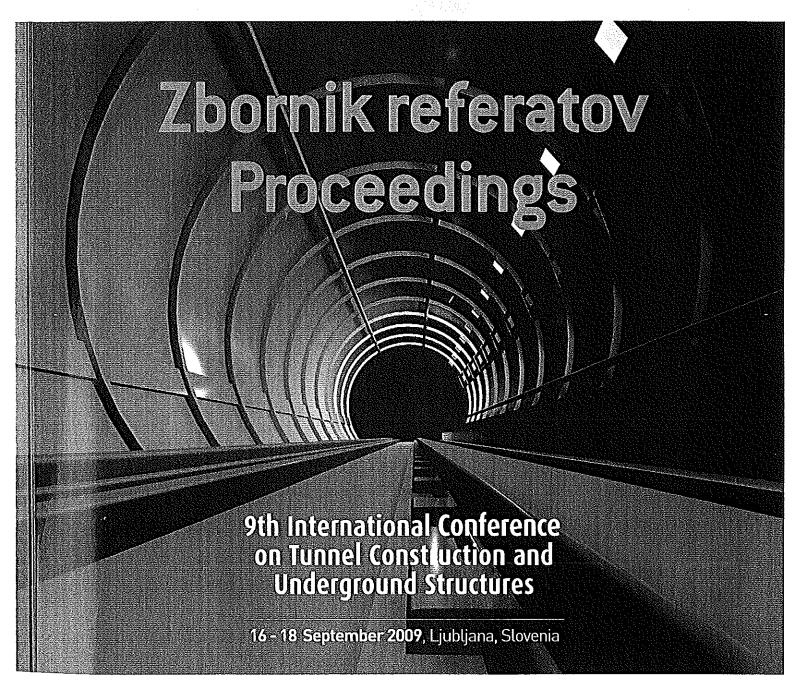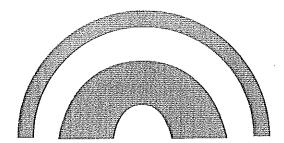
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Tema 3 / Topic 3:

Načrtovanje, raziskave, monitoring in regulative v predorogradnji
Design, Research, Monitoring and Regulatives in Tunelling

A Methodology for Analysing the Drainage System in Excavations between Sheet Pile Walls

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ABSTRACT

The development of an underground station below the water table requires rigorous and careful planning. As such an in depth knowledge of the station construction designs and the related hydraulic behaviour are required in order to design the most appropriate drainage system. Moreover, it is important to ensure the correct drainage of the excavation site to avoid liquefaction and to minimize water seepage.

As an example of an underground construction we consider the tunnel for the new subway line in the Barcelona metropolitan area (Línia 9) through the town of Prat de Llobregat in Llobregat River's delta, which is currently under development. Most of the tunnel sections are constructed with Tunnel Boring Machines (TBM) and the stations and the ventilation shafts are constructed using "Cut & Cover" technology between sheet pile walls. A typical station on the L9 line lies partially between the shallow and the main

aquifer. Specifically, it is located in the middle layer which is made up of silts and clays. In order to design the most appropriate drainage system we present a methodology whereby we generalise the hydraulic problem. This method enables rapid and efficient estimation of fluxes as well as a general understanding of the problem in a simplified manner.

KEYWORDS
Underground station, drainage, sheet pile walls, safety factor, empirical method.

1. INTRODUCTION
The construction of subway line infrastructures in densely populated areas has increased over recent years and these types of constructions are inherently hazardous. The hazards pose risks to workers who enter to the excavation as well as potential damage to surrounding buildings. Typical hazards include cave-ins, which pose great risks to workers, as well as falls, falling loads, hazardous atmospheres, and incidents involving mobile equipment. However, one of the most dangerous scenarios would be the collapse of the excavation due to the heterogeneity of the soil. Moreover, we must consider that some underground structures could be built below the water table.

In order to avoid any risk during the development of an underground station below the water table it is necessary to ensure the stability of bottom of the excavation site and avoid siphoning. Therefore, an in depth knowledge of the station construction designs and related hydraulic behaviour are required so as to design the most appropriate drainage system and calculate the required factor of safety. It is important to establish a rapid manner with which to evaluate the factor of safety and to date there are no simple and effective analytical solutions to do so. The development of a method to minimize time, effort and errors in calculating the stability of an underground excavation will contribute to more effective development of future works.

The goal here is to develop a methodology to calculate the drainage flow for the construction of the station, which is excavated below the water table, between sheet pile walls in a fast and efficient manner. We propose a simplified model problem and develop an empirical dimensionless formula to evaluate the drainage flow accounting for the hydraulic parameters of the soil and the station construction design.

2. METODOLOGHY

2.1 problem statement
To simplify the problem, we assume a vertical semi-confined aquifer of constant thickness (b) and of infinite extent (r∞), as is illustrated in the figure 1.The aquifer is anisotropic (kr ≠ kz) and a fully penetrating pumping well is located at the centre of this cylindrical model. The well is of finite radius (rw) and there is a cylindrical screen wall at a given distance (rexc D). The screen can penetrate to any depth within the aquifer, measured by the aperture AD and they have a very low permeability (kp).
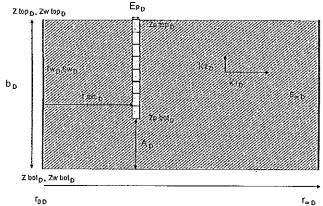

Figure 1.Schematic diagram of a fully penetrating well in a vertically anisotropic aquifer.

2.2 Dimensionless formulation
We study this problem in dimensionless form. Based on Darcy's law, the radial flow equation to describe the distribution of the drawdown in a confined anisotropic aquifer is given by:

$$0 = \frac{1}{r}\frac{\partial}{\partial r}\left(r\, k_r\, \frac{\partial s}{\partial r} \right) + \frac{\partial}{\partial z} k_z \frac{\partial s}{\partial z}$$

where r represents the radial distance from the pumping well, z represents the vertical distance, kr and kz represent the horizontal and vertical permeability of the aquifer, respectively, and s represents the drawdown.

In order to non-dimensionalize the problem we use the characteristic variables shown in the table 1.

Characteristic variable		Dimension
Well drawdown		[L]
Aquifer thickness		[L]
Excavation radius		[L]
Horizontal Permeability		[LT⁻¹]

Table 1.The characteristic variables of the dimensionless problem.

Once the characteristic variables are defined we can write the dimensionless variables as:

$$S_D = \frac{s}{s_c}, r_D = \frac{r}{r_c}, k_D = \frac{k}{k_c}, z_D = \frac{z}{r_c}$$

The resulting steady state radial flow equation in dimensionless form is given by:

$$0 = \frac{1}{r_D}\frac{\partial}{\partial r_D}\left(r_D \frac{\partial s_D}{\partial r_D}\right) + R \frac{\partial^2 s_D}{\partial z^2_D} \quad \text{where } R = \frac{a r_{exc}^2}{b^2}$$

2.3 Boundary conditions

The boundary conditions of the problem are illustrated in the Figure 2. The top and the base of the aquifer are non flow boundaries.

$$\left.\begin{array}{c} z_D = z_{top} \\ z_D = z_{bot} \end{array}\right| \rightarrow K_{zD}\frac{\partial s_D}{\partial z_D} = 0$$

The well drawdown is given as a prescribed head boundary as is the case at the infinite radial distance, where the drawdown is considered to be zero because the pumping well no longer influences at this distance.

$$z_D \in \left(z_{wbot}, z_{w\,top}\right) \rightarrow S_{(r=rw)} = 1$$

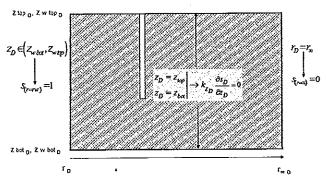$$r_D = r_\infty \rightarrow S_{(r=\infty)} = 0$$

Figure 2. Boundary conditions of the problem.

2.4 Numerical simulations

A series of numerical simulations, spanning a wide range of the dimensionless phase space, were conducted with our in house TRANSIN IV code [1]. This was done so as to obtain data for an empirical solution that allows us to calculate the drainage flow rate given the prescribed geometry (Figure 1). Some of the geometric and hydraulic parameters are taken as constant for all the simulations such as E_p, K_p, K_r and r_∞ (look at Appendix I). The non constant parameters and its values are defined in the table 2.

Parameter	Value
A	1,0.9,0.8,0.7,0.6,0.5,0.4,0.3,0.2,0.1, 0.08,0.06,0.04,0.02,0.01.
R	0.005,0.02, 0.05, 0.2, 1
r_w	0.01,0.02,0.025

Table 2. The dimensionless values of the non constant parameters.

For this specific problem, 225 simulations were conducted and for each of them we obtain a Q_0. Once we have our values of Q_0 we use the Furnival & Wilson algorithm [2] in order to obtain a best fit trend line accounting for all parameters.

In figure 3 the trend line is shown that relates various different variables considered (axis X) to the flow rate (axis Y), Q_0. The regression coefficient of the trend line is $R^2=0.9944$, suggesting a high goodness of fit

$$X = -0.0803\ln A - 0.3451\ln(1-A)\ln(\frac{1}{A\sqrt{R}}) - 0.0873\ln\left(\frac{r_\infty}{r_w}\right) + 0.0152\left[\left(\ln(1-A)\ln(\frac{1}{A\sqrt{R}})\right)\cdot\ln\left(\frac{r_\infty}{r_w}\right)\right] + 0.0099\left(\ln(1-A)\ln\frac{1}{A\sqrt{R}}\right)^2$$
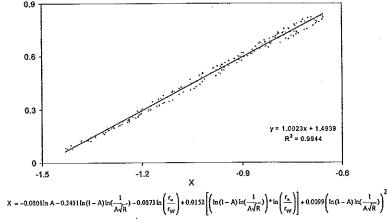
Figura 3. Trend line obtained grouping the parameters with Furnival and Wilson algorithm (1974).

This empirical solution for the dimensionless flow rate is given by:

$$Q_D = \alpha \ln A + \beta\left[\ln(1-A)\ln(\frac{1}{A\sqrt{R}})\right] + \gamma \ln\left(\frac{r_\infty}{r_w}\right) + \lambda\left[\left(\ln(1-A)\ln(\frac{1}{A\sqrt{R}})\right)\cdot\ln\left(\frac{r_\infty}{r_w}\right)\right] + \mu\left(\ln(1-A)\ln\frac{1}{A\sqrt{R}}\right)^2$$

where $\alpha = -0.0803$, $\beta = -0.3451$, $\gamma = -0.0873$, $\lambda = 0.0152$, $\mu = 0.0099$

If the empirical solution is compared with the results of the numerical model, the error in the simulations is less than 10% for more than 95% of the cases considered.

3. SYNTHETIC CASE

This study is motivated by the hydrogeologic monitoring of the L9 underground construction works through the town of Prat de Llobregat (Spain) and more specifically, in the drainage for the construction of underground metro stations and ventilation shafts. The constructions designs consist of perimetral sheet pile walls that are impervious to flow relative to the aquifer's inherent permeability. Thus, it is assumed to act as an impermeable barrier but the circumferential fluxes must be considered due to the aperture.

As an example we present a synthetic case. The goal is to calculate the drainage flow of an underground station with the following design constraints: (1) a pumping well radius is 0.2 meters, (2) an excavation radius of 20 meters, (3) an excavation that is located at 28 meters above sea level, (4) the bottom of the sheet pile walls is 12 meters above sea level (5) and a sheet-pile thickness of 1.2 meters. The hydraulic parameters are K_h= 10 m/day, a= 5, K_p= 0.0001 m/day and the aquifer thickness is 40 meters. To simplify the problem the initial piezometric head is 40 masl.

The first step in order to estimate drainage flow rate is to calculate the dimensionless parameters that drive the problem (see appendix I). Then we can estimate the drainage flow rate with our empirical solution and compare it a numerical one.

Q_D		Error (%)
Numerical Model	0.45634	3.49
Empirical solution	0.47286	

Table 3. Comparison of the flow rate for the synthetic example.

As indicated in table 3 the agreement between the empirical fit and numerical simulation is pretty good with an error of 3.5%. Thus it appears that the empirical solution presented provides a good approximation to evaluate the drainage flow rate in an underground station constructed between sheet pile walls.

Next we aim to calculate the factor of safety (FS) of the excavation. The FS is defined as the ratio between overburden pressure (PoD) and the hydrostatic pressure (PwD).

$$FS_D = \frac{Po_D}{Pw_D} = \frac{(h_{exc_D} - zi_D)\rho_{b_D}}{(h_D - zi_D)\rho_{w_D}}$$

If the (Pw_D) is equal or larger than Po_D, it is possible that siphoning or bottom uplift will occur. In order to work in safe conditions we consider that a FS greater than 1.2 is required. As an illustrative example, we evaluate the safety factor at

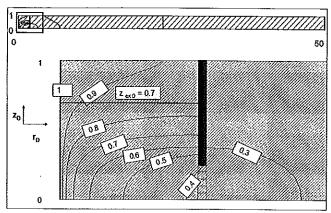the bottom level of the sheet pile wall of the aforementioned example.

Figure 4. Detail of the dimensionless drawdown of the synthetic example.

It is important to know the exact drawdown at the bottom of the sheet pile walls in order to accurately evaluate the FS. An easy manner to calculate the drawdown with the numerical model is by putting an observation point at the considered level (figure 4). The result is that s_D is equal to 0.438 (dimensionless units) and it is equivalent to a hydraulic head, h_D, equal to 0.847 (dimensionless units). Finally, the safety factor is evaluated as follows:

$$FS_D \frac{Po_D}{Pw_D} = \frac{(z_{exc_D} - zi_D)\rho_{b_D}}{(h_D - zi_D)\rho_{w_D}} = \frac{(0.7-0.3)2}{(0.847-0.3)1} = 1.46$$
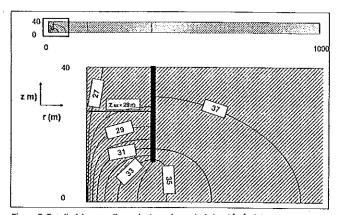

Figure 5. Detail of the non-dimensionless piezomètric level (m) of the synthetic example.

Finally, to compare the safety factor of the dimensionless and non-dimensionless synthetic models, a numerical simulation was realized with the non-dimensionless model and the hydraulic level is shown in the figure 5. Then, the safety factor is evaluated as follows:

$$FS = \frac{(z_{exc} - zi)\rho_b}{(h - zi)\rho_w} = \frac{(28-12)2}{(33.74-12)1} = 1.47$$

If the results of the safety factor are compared, the error between them is less than 1%.The error could be attributed to an error due to the numerical simulations and indicates that this is a reasonable approach.

5. CONCLUSIONS

The empirical equation presented herein provides a good first order approximation to evaluate in a quick and efficient manner the flow rate in an underground construction site. Moreover, when the empirical solution is compared with the results of the numerical model, the error in the simulations is less than 10% for more than 95% of the cases considered suggesting that the empirical solution is a good one. However, this empirical fit is entirely data driven with little physical motivation. As such we are currently seeking a more physically driven empirical solution.

Due to the simplifications in the problem statement, future work may consider an extension to include more layers with different hydraulic properties in the aquifer in order to generalise the problem to more realistic situations.

APPENDIX I. Notation of the used variables.

A	aperture, zp bot D- z bot D	[L]
Ep	sheet pile walls thickness	[L]
FS	safety factor	
Q	flow rate	[L3 T-1]
Qc	characteristic flow rate, Khbsc	[L3 T-1]
QD	dimensionless flow rate	
Po	Overburden pressure,(zexc-zi)*dt	[M L-2]
Pw	Hydrostatic pressure, (h-z)*dw	[M L-2]
a	anisotropy factor	
b	aquifer thickness, zsup- zinf	[L]
ϱb	bulk density, 2	[M L-3]
ϱw	water density, 1	[M L-3]
h	water table	[L]
hw	well head	[L]
kc	characteristic hydraulic conductivity, kh	[L]
kiD	dimensionless hydraulic conductivity	
kr	horizontal hydraulic conductivity	[LT-1]
kp	sheet pile walls hydraulic conductivity	[LT-1]
kz	vertical hydraulic conductivity , a Kh	[LT-1]
r	radial distance	[L]
rc	characteristic radius, rexc	[L]
rDi	dimensionless radial distance	
rexc	excavation radius	[L]
rw	well radius	[L]
r∞	infinite radius	[L]
s	drawdown	[L]
sc	characteristic drawdown, hn-hw	[L]
sD	dimensionless drawdown	
z	level	[L]
zexc	excavation level	[L]
zDi	dimensionless level	
z bot	aquifer bottom level	[L]
z top	aquifer top level	[L]
zp bot	sheet pile wall bottom level [L]	
zp top	sheet pile wall top level [L]	
zw bot	well bottom level [L]	
zw top	well top level [L]	

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