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Numerical analysis of the effects of grouting on mitigating the risk of hydraulically induced failure during deep shaft excavation

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ABSTRACT: Hydraulic failure presents a significant stability and safety issue for deep shaft excavation projects that deal with a high groundwater table in cohesionless soil. In urban environments, the lowering of the groundwater table for deep shaft excavations is usually not an acceptable solution to mitigate these risks due to the surface settlements it may cause, thus jeopardizing the safety of the surrounding structures. In this paper, a case study of an inlet shaft of a TBM-bored wastewater tunnel in Belgrade, Serbia was analyzed in order to show that grouting techniques can be an appropriate measure for reducing hydraulically induced instability. Since the location of the shaft is near the riverbank, the soil stratigraphy consists of granular soils with a high groundwater table. Several different variations of grout curtains around the shaft were modelled by conducting steady state flow analysis using commercial software based on the finite element method, and the obtained results were used to determine the risk of hydraulically induced failure at the bottom of the excavation. Based on the obtained results it is concluded that the application of grout curtains is a suitable solution for increasing the factor of safety for hydraulic heave problems as well as for soil failure problems. However, when encountering an aquitard layer during the earlier phases of excavation, grout curtains have no effect on lowering the pore pressures that are inducing the uplift at the bottom of the aquitard layer.

1 INTRODUCTION

As one of the typical failure mechanisms during deep shaft excavation in cohesionless soils, hydraulic failure can represent the predominant stability and safety issue in cases where a high groundwater table is present outside the bounds of the excavation pit. These issues are even more accentuated in cases where initial lowering of the groundwater table is either unfeasible due to surface settlements it may cause, thus jeopardizing the safety of neighboring structures, or the required level to which the groundwater table should be lowered to in order to meet the requirements of hydraulic safety deems it impractical in terms of cost and duration of works.

Hydraulic failure presents a risk to deep excavations when there is significant difference in groundwater levels inside and outside of the excavation, which results in the seepage of the groundwater around the toe of the non-porous barrier lining the excavation (diaphragm walls, pile walls, sheet-piles, grout curtains, etc.). Hydraulic gradients, pore pressures and seepage forces are the predominant actions to be considered in design against hydraulic failures, and therefore special attention must be given to those parameters which have the greatest influence on these actions – soil composition, the variation of soil permeability, and the variations in water levels and pore pressure (Frank et al. 2004).

Based on the soil composition, hydraulic failure at the base of deep excavations can occur via different failure mechanisms (Figure 1). Hydraulic uplift (Figure 1a) occurs when a soil layer of low permeability, like clay for example, is situated below the level of excavation, through which there will be practically no groundwater flow, causing an uplifting pressure at

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the bottom of the soil layer. Hydraulic heave (Figure 1b) occurs when the ground below the excavation is permeable, like sand or gravel, causing groundwater flow upwards into the excavation. Additionally, if the soil composition is unfavorable in regards to particle and/or void size and distribution, the upward groundwater seepage can cause internal erosion that leads to material transport, and subsequently to failure by piping, where a pipe-shaped discharge tunnel is formed around the toe of the excavation wall eventually leading to complete failure of the structure as soon as the discharge tunnel in the soil reaches the water level outside the excavation (Vukotic & Pusic 1986).

When unaccounted for, hydraulic failure of deep excavations can cause catastrophic consequences that risk the safety of workers on the construction site, as well as cause stagnation in the construction process leading to potentially great economic loss. Although failure of deep excavations is not caused exclusively by hydraulic basal failure, it has presented a major component in several infamous cases of deep excavation failure. In January 2000, there was an extensive collapse of an excavation for the Taegu Metro in South Korea, attributed to the presence of unidentified strata of sands and gravels which were subjected to a rapid increase of the groundwater level not included in the design (Figure 2a; Endicott 2015). In Singapore in 2004, a 30 m deep excavation alongside the Nicoll Highway failed partially due to insufficient toe embedment for hydraulic cut off, resulting in four fatalities and several injuries (Figure 2b; Endicott 2015).

To evaluate the proper solution when dealing with hydraulic stability problems, a case study of an inlet shaft of a TBM-bored wastewater tunnel in Belgrade, Serbia was analyzed. As part of a broader wastewater treatment project, a tunnel under the Sava River is necessary, and to construct this tunnel using a TBM, deep inlet and outlet shafts are required for TBM assembly, launch and extraction. A schematic cross section of the project at hand is shown in Figure 3.

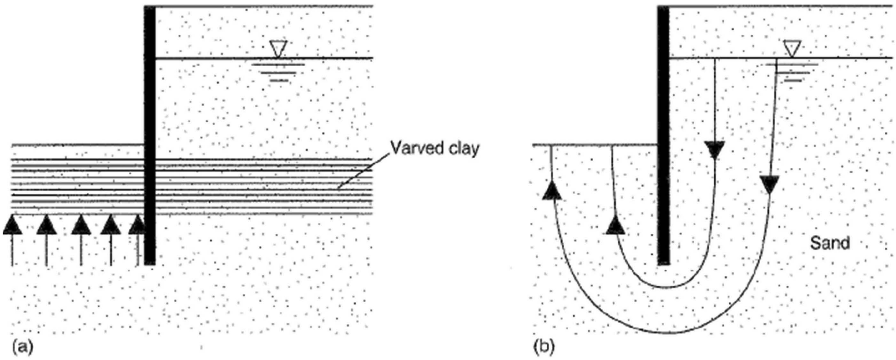


Figure 1. Examples of hydraulic failure mechanisms due to different soil conditions: (a) failure by uplift and (b) failure by heave (Frank et al. 2004).

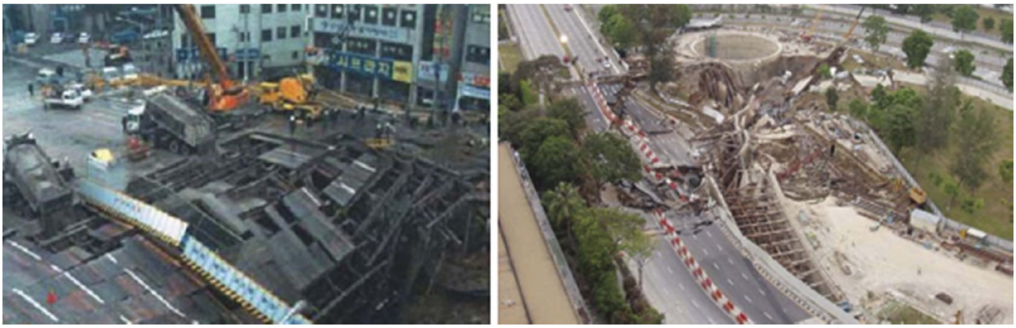


Figure 2. Failure cases of deep excavations partially due to hydraulic instability: (a) Taegu Metro in South Korea and (b) Nicoll Highway collapse in Singapore (Endicott 2015).

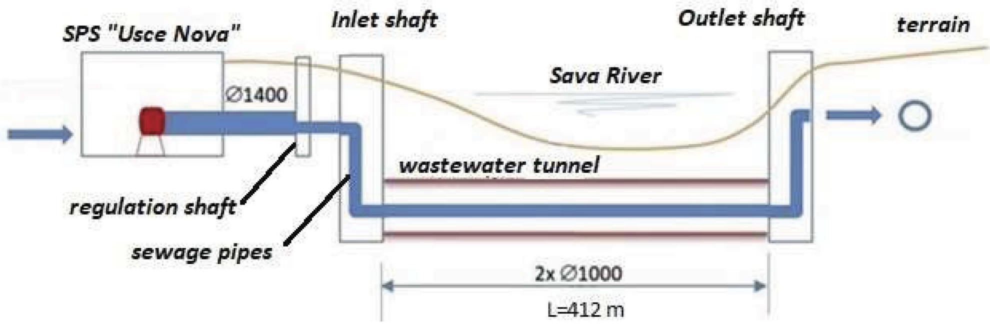


Figure 3. Schematic cross section of the case study project.

2 CASE STUDY PROJECT DESCRIPTION

The shaft is a permanent structure, and it consists of concentrically placed reinforced concrete piles with the length of 40 m, and a diameter of 80 cm, while the excavation is 33.20 m deep in total (42.90 masl). The excavation inside the piles has a diameter of 18 m, and the lateral resistance of the shaft is ensured with seven reinforced concrete rings along the intrados of the shaft. The excavation process is executed in seven steps, with each step ending with the construction of a reinforced concrete support ring.

Since the location of the shaft is near the riverbank, the soil stratigraphy generally consists of granular soils with a high groundwater table. The ground surface is at 76.10 masl, while the groundwater level is in direct hydraulic dependance with the level of the Sava River and is set to 70.00 masl for the purpose of this study. Based on the available geotechnical report, the soil in the zone of the inlet shaft is approximated with six horizontally layered quasi-homogeneous zones. The material parameters of importance for stress-strain and flow analysis of the shaft are shown in Tables 1-2 for each soil layer.

It can be immediately realized, due to the position of the shaft and the significant difference in the water pressure head outside and inside the shaft, that hydraulic failure is of major concern for the overall stability of the construction process. Based on the permeability coefficients given in Tables 1-2, three distinct zones can be observed that differ in behavior when subjected to upward seepage forces as a consequence of groundwater flow around the toe of the pile wall:

- Zone A – Soil subjected to uplift during excavation (for excavations above the silty layer SM/SC i.e., above 57.10 masl)
- Zone B – Soil subjected to hydraulic heave during excavation (for excavations between the silty layer SM/SC and the bedrock i.e., between 57.10 masl and 47.10 masl)
- Zone C – Porous bedrock through which groundwater infiltration is expected but without risks of hydraulic failure

The conventional methods when dealing with the hydraulic failure of the excavation base of deep shafts, whether designing against uplift or hydraulic heave, entail lengthening the flow path by lowering the toe of the flow barrier or lowering of the initial groundwater level (Puller 2003). For this project, additional lowering of the toe of the pile wall is to be avoided since the piles are already of significant length and any additional lengthening of the piles would further risk the inaccuracy of their verticality, which is already an area of concern for the construction phase. On the other hand, lowering of the initial groundwater level has shown unsatisfying numerical results when it comes to the settlement of the ground surface. This would be especially problematic for the functionality and even safety of the neighboring sewage pumping station "Usce Nova" (Figure 3), which is an essential element of the entire wastewater treatment project.

Due to these reasons, the implementation of grout curtains for ensuring the hydraulic stability of the structure during excavation in considered. In the following sections, the applied methodology for analysis is shown, and the obtained results are assessed and discussed.

Table 1. Parameters of soil layers modelled with the Hardening Soil material model.

Layer	Description	Z_{top} [masl]	Z_{bot} [masl]	γ [kN/m^3]	c [kPa]	ϕ [°]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	k [m/s]
N	Fill	76.10	71.10	17	0	33	17.50	52.50	$5.5 \cdot 10^{-5}$
SP/SW	Poor to well graded sands	71.10	61.10	19	0	31	13.50	40.50	$5.5 \cdot 10^{-5}$
SM/SC	Silty to sandy sediments	61.10	57.10	18.5	17.5	18.5	7.25	21.75	$5.5 \cdot 10^{-7}$
SP/SW	Poor to well graded sands	57.10	51.10	19	0	31	13.50	40.50	$5.5 \cdot 10^{-5}$
GW	Well graded gravel	51.10	47.10	19.5	0	35	27.50	82.50	$5.5 \cdot 10^{-4}$

Table 2. Parameters of soil layers modelled with the Hoek-Brown material model.

Layer	Description	Z_{top} [masl]	γ [kN/m^3]	$\lvert\sigma_{ci}\rvert$ [MPa]	m_b [-]	s [-]	a [-]	E_{rm} [MPa]	k [m/s]
KP-PS	Sandy porous limestone	47.10	21.6	7.80	1.262	$2.2 \cdot 10^{-3}$	0.508	380	$1 \cdot 10^{-4}$

3 METHODOLOGY OF ANALYSIS

For the purpose of numerical analysis, a two-dimensional axisymmetric steady-state groundwater flow model of the inlet shaft is created using Plaxis 2D software (Figure 4a). The cohesionless upper layers of soil are modelled with the Hardening Soil material model (Table 1), while the bedrock of sandy porous limestone is modelled with the Hoek-Brown material model (Table 2). The pile wall is modelled with an equivalent linear-elastic plate element, to which interface elements for modelling the soil-structure interaction and the impermeability of the pile wall are added. The reinforced concrete bracing rings are modelled with fixed-end anchor elements with the adequate stiffness, and each of them is placed 1 m above the corresponding level of the stage excavation base (Table 3).

The construction procedure that is modelled consists of the Initial phase, followed by the phase in which the pile wall is modelled and then seven excavation stages (ES 1-7), whose details are shown in Table 3.

In variations of the model where the application of a grout curtain was analyzed, an additional soil volume with a thickness of 1 m was created from the base of the pile to the analyzed

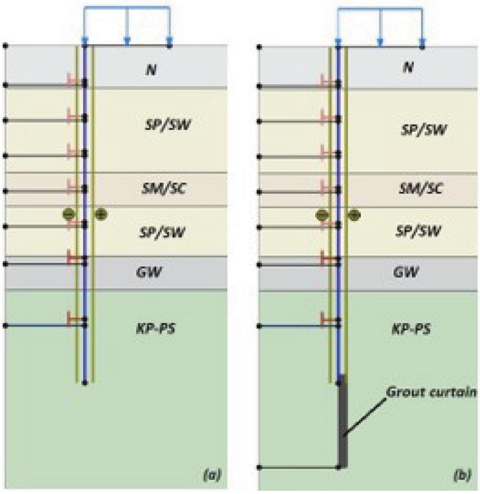


Figure 4. Numerical models in Plaxis 2D: (a) model without grout curtain and (b) model with grout curtain.

depth of the grout curtain (Figure 4b). The material model of the grout curtain is identical to that of the bedrock layer KP-PS, with the only difference being the permeability coefficient k.

Table 3. Overview of construction stages and the required ultimate limit state verification for each.

Stage	Stage excavation depth [m]	Level of excavation base [masl]	Excavation zone	Required check
Initial	±0.00	76.10	-	-
Pile wall construction	±0.00	76.10	-	-
ES 1	-4.65	71.45	A	-
ES 2	-4.20	67.25	A	UPL
ES 3	-4.20	63.05	A	UPL
ES 4	-4.20	58.85	A	UPL
ES 5	-4.20	54.65	B	HYD
ES 6	-4.45	50.20	B	HYD
ES 7	-7.30	42.90	C	GEO

The ultimate limit state necessary for verification differs between different excavation stages. For ES 1, the level of excavation is above the natural groundwater level, so no verification is needed.

For ES 2-4, because the silty layer SM/SC acts as an aquitard, it is necessary to verify the uplift ultimate limit state (UPL). UPL is verified in accordance with the Eurocode EN 1997-1 standard, where the design values of the destabilizing water pressure at the bottom of the aquitard $Q_{dst,d}$ must be less than the design values of the stabilizing overburden weight $G_{stb,d}$ for each excavation stage. The destabilizing water pressure at the bottom of the aquitard is determined directly from the results output. For each of the ES 2-4 the following equation must be satisfied:

$$Q_{dst,d} = \gamma_{G,dst} \cdot u_{dst,k} < G_{stb,d} = \gamma_{G,stb} \cdot \sum\nolimits_j \gamma_j \cdot d_j \Rightarrow F_s = \frac{Q_{dst,d}}{G_{stb,d}} > 1.0 \tag{1}$$

where $\gamma_{G,dst}$ = partial safety factor for loads; $u_{dst,k}$ = pore pressure at the bottom of the aquitard layer; $\gamma_{G,stb}$ = partial safety factor for resistances; γ_j = respective unit weight of the overburdening soil layer j above the aquitard and d_j = respective thickness of soil layer j above the aquitard.

For ES 5-6, because the sand layer SP/SW acts as an aquifer, for these stages it is necessary to verify the hydraulic heave ultimate limit state (HYD). HYD is verified in accordance with the Eurocode EN 1997-1 standard precisely, using the submerged weight approach, where the design value of the destabilizing seepage force $S_{dst,d}$ acting within the relevant soil column must be less than the design value of the stabilizing submerged weight $G'_{stb,d}$ of the relevant soil column. The destabilizing seepage force is determined by integrating the product of the hydraulic gradient (a result output) and the length on which the gradient acts, from the excavation base level to the toe of the groundwater barrier. For each of the ES 5-6 the following equation must be satisfied:

$$S_{dst,d} = \gamma_{G,dst} \cdot \gamma_w \cdot \int i \cdot dz < G'_{stb,d} = \gamma_{G,stb} \cdot \sum\nolimits_j \gamma'_j \cdot d_j \Rightarrow F_s = \frac{S_{dst,d}}{G'_{stb,d}} > 1.0 \tag{2}$$

where $\gamma_{G,dst}$ = partial safety factor for loads; i = hydraulic gradient acting on part of the relevant soil column with the length of z; $\gamma_{G,stb}$ = partial safety factor for resistances; γ'_j = respective submerged unit weight of the soil layer j in the relevant soil column and dj = respective thickness of soil layer j in the relevant soil column.

It should be noted that the partial safety factors in Eurocode EN 1997-1 with which the characteristic values of the actions and resistances are multiplied, differ between UPL and HYD. While the partial safety factor for the stabilizing resistances $\gamma_{G,stb}$ is equal to 0.90 in both cases of design, the partial safety factor for the destabilizing forces $\gamma_{G,dst}$ is equal to 1.00 when designing for UPL, as opposed to it being equal 1.35 when designing for HYD.

For ES 7, the excavation has reached the layer of bedrock in which hydraulic failure is not expected even though groundwater infiltration through the excavation base is expected. For this stage, an analysis of the ground failure ultimate limit state (GEO) is executed by determining the factor of safety using the Safety analysis option in Plaxis, which is based on the strength reduction method described in the Plaxis manual. In accordance with the Serbian National Annex of Eurocode EN 1990, GEO is conducted using the Design Approach 3, in which the material strength properties are divided by the partial safety factor γ_M = 1.25, while the service load applied on the ground surface is multiplied by the partial factor of safety γ_Q = 1.30. This must result in a factor of safety greater than 1.0.

4 CALCULATION RESULTS

The original set-up of the structure does not provide satisfactory results when conducting the necessary verifications for hydraulic safety during excavation. A grout curtain previously described in Section 3 was added with the length of 10 m and with varying values of its permeability coefficient *k*. The factor of safety for ES 6 was determined for each value of *k* and the results are shown in Figure 5. It is concluded that the necessary value of *k* that adequately diverts the flow of groundwater around the toe of the grout curtain is 10^{-6} m/s, but also that there is no significant difference in behavior of the groundwater or the value of the factor of safety if the value of *k* is further increased. This means that the minimal necessary watertightness for the proper functionality of the constructed grout curtain in this project is represented by the value of $k = 10^{-6}$ m/s.

With the established value of the permeability coefficient *k*, different lengths of the grout curtain were modelled to assess the influence of the grout curtain length on the corresponding factors of safety for each ES 2-6. The results are shown in Figure 6, keeping in mind that different limit states are calculated for different stages, as explained in Sections 2 and 3.

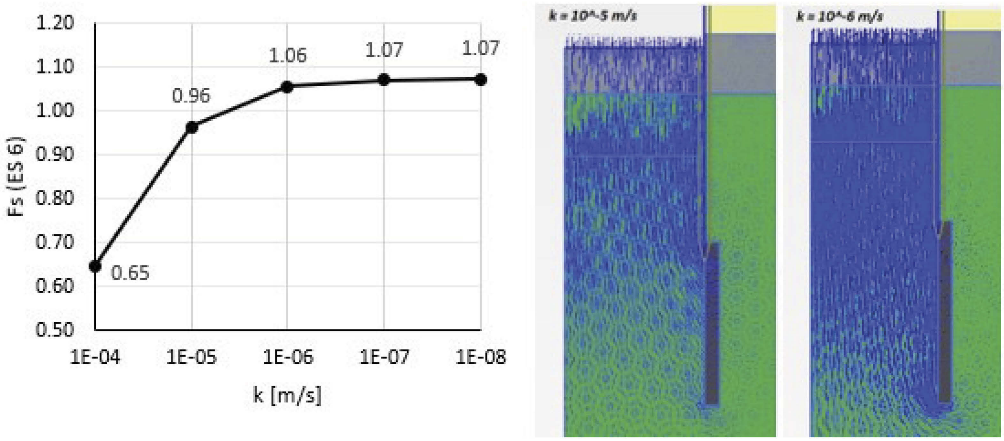
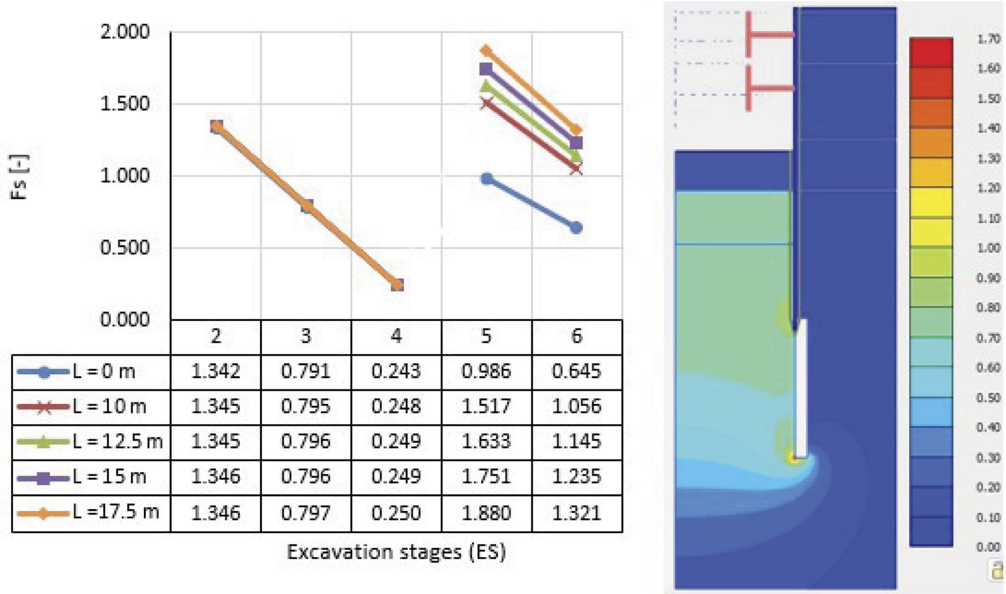


Figure 5. Values of the factor of safety for ES 6 depending on the *k* value of the grout curtain (left) and the behavior of the groundwater flow depending on the *k* value of the grout curtain (right).

What can be concluded based on the results shown in Figure 6 is that the presence of a grout curtain has a significant impact on safety against hydraulic heave (HYD) for ES 5-6 but has no effect on ensuring safety against uplift (UPL) for the earlier stages ES 2-4. For this reason, additional measures are needed for ES 3-4 in order to reduce the destabilizing pore pressure acting on the bottom of the aquitard. One of the measures that can be applied is drainage from the sandy layer underneath the aquitard, which is applied to the existing Plaxis models by adding a well element with a 30 cm diameter inside the shaft at a distance of 2 m from the piles and its activation during ES 3-4 (the excavation stages that have a factor of



	2	3	4	5	6
L = 0 m	1.342	0.791	0.243	0.986	0.645
L = 10 m	1.345	0.795	0.248	1.517	1.056
L = 12.5 m	1.345	0.796	0.249	1.633	1.145
L = 15 m	1.346	0.796	0.249	1.751	1.235
L =17.5 m	1.346	0.797	0.250	1.880	1.321

Figure 6. Comparisons between safety factors depending on the grout curtain length *L* and the excavation stage (left); and the distribution of the hydraulic gradient *i* for ES 6 and *L* = 10 m.

safety less than 1.0). The results of the model with the drain element and the previously explained grout curtain of 10 m in length is shown in Figure 7.

For ES 7 a comparison of the factor of safety in function of the grout curtain length is shown in Figure 8. Although it is unclear whether the implementation of the Strength Reduction Method in accordance with the Eurocode standard EN 1997-1 should be applied to characteristic soil strength parameters or to the soil strength parameters that are reduced in accordance with the selected Design Approach, for both cases the presence of the grout curtain has significant impact in increasing the factor of safety and therefore reducing the risk of soil failure at the base of the excavation.

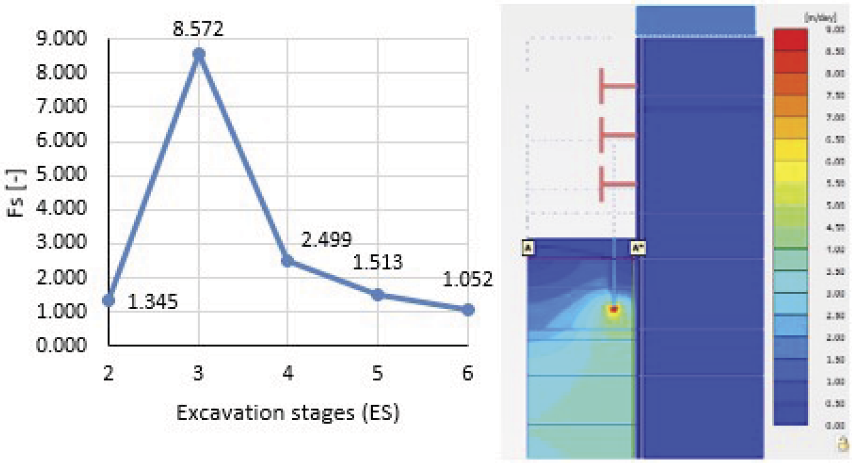


Figure 7. Factors of safety through all ES in the model with a drain element and a grout curtain of *L* = 10 m (left); groundwater flow through drain element for ES 4 (right).

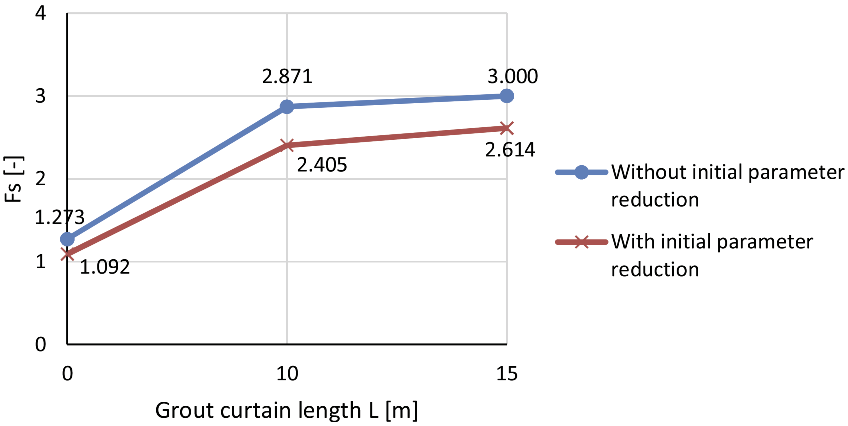


Figure 8. Factor of safety obtained with plaxis safety analysis for ES 7 depending on the grout curtain length *L*.

5 CONCLUSION

As one of the typical failure mechanisms during deep shaft excavation in cohesionless soils, hydraulic failure can represent the predominant stability and safety issue in cases where a high groundwater table is present outside the bounds of the excavation pit. When the option of lowering the groundwater table is not applicable, grouting can present a rational solution to mitigate the risk of hydraulically induced failure during deep shaft excavations.

Several different variations of grout curtains around the case study shaft were modelled using the finite element method. Steady state flow analysis was conducted in Plaxis 2D software and the obtained results were used to evaluate and cross-compare the risk of hydraulically induced failure at the bottom of the excavation. Due to the distinct soil stratigraphy, different limit state verifications were needed for different excavation stages.

The minimal permeability coefficient necessary for the proper behavior of the grout curtain is $k = 10^{-6}$ m/s and any further reduction of the water permeability coefficient does not have an influence on the behavior of the grout curtain or the groundwater flow. Based on the obtained results it is concluded that the application of grout curtains is a suitable solution for increasing the factor of safety for hydraulic heave problems and for soil failure problems. However, when there exists a risk of uplift issues in the upper layers of excavation, grout curtains have no influence on lowering the pore pressures inducing the uplift at the bottom of the aquitard layer.

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