Design of water isolation grouting for reducing high water inflows in urban shallow tunnels

N. Koronakis, P. Kontothanassis & D. Katsaris
Omikron Kappa Consulting Ltd, Athens, Greece

J. Bournazos
Technical Manager AKTOR S.A., Athens, Greece

ABSTRACT: The paper presents a decision making process for the determination of the numerous grouting parameters during design and the necessary corrective and evaluation actions during field trials or systematic grouting. Special application of design and execution of grouting for shallow tunnels in urban areas is given. The evaluation of field trials is also discussed, thus fine-tuning the design and optimizing the water sealing efficiency.

1 GENERAL PLANNING – DECISION MAKING PROCESS

1.1 *Design considerations*

Successful treatment of a soil or rock-grouting problem depends on the basic understanding of the general engineering fundamentals, soil or rock mechanics, hydrogeology and environmental geochemistry. A grouting engineer must make a proper use of the geotechnical engineer's "toolkit" in order to determine the soil's or rock's flow properties, nature and structure - these factors are the cornerstones of any grouting program design.

1.2 *Decision making process- parameters*

The grouting concept must always be based on the idea to start performing the work by using the easiest and simplest available technique.

The decision making process in a grouting programme consists of several parameters, some of which are mentioned below.

1.2.1 *Injection & drilling sequencing*

The simplest injection hole layout is the parallel pipe array, in which all the injection pipes are placed parallel to each other. Construction constraints may require a fan array, in which the pipes diverge.

Injection staging involves primary, secondary, tertiary, and quaternary grouting stages, the last two being optional depending on the project requirements. Primary grouting refers to initial grout penetration in ungrouted zones where adjacent grout bulb boundaries are either in partial contact or not. Secondary grouting refers to grouting the gaps between primary grout boundaries. Tertiary and quaternary grouting targets spots that the first two stages missed. The result is a grouted soil or rock mass with overlapping grout bulbs.

Ultimate injection hole spacing for most grouting projects range from 0.8 to 2m. Typically, the greater the distance between injection holes, the less definitive the grout front becomes. When grout holes are spaced too close, the result is excessive drilling costs.

All holes of an intermediate set in any section of the grout curtain are grouted before the next set of intermediates is drilled. If grout frequently spreads from one primary hole to another, an increase in the primary spacing is necessary. As the split spacing technique reduces the intervals between grout holes, the average grout consumption per linear meter of hole should also become smaller.

1.2.2 *Permeability*

In-situ field permeability tests give more accurate results than lab permeability tests performed on representative field samples. Laboratory test results often have systematic errors. Furthermore, undisturbed cohensionless soil samples are difficult to obtain since they fall apart easily. Permeability is measured in cm/sec expressed by the variable k:

- $k = 10^{-6}$ or less: ungroutable
- $k = 10^{-5}$ to 10^{-6}: groutable with difficulty by grouts under 5cP viscosity and ungroutable for higher viscosities
- $k = 10^{-3}$ to 10^{-5}: groutable by low-viscosity grouts but with difficulty when k is more than 10 cP

– k = 10^{-1} to 10^{-3}: groutable with all commonly used chemical grouts
– k = 10^{-1} or more: use suspended solids grout or chemical grout with a solids filler

Field permeability test methods are the double packer test and the well pumping test.

1.2.3 *Injectability*

At present, there are no truly reliable small scale or laboratory methods which will accurately determine the injectability limits of soils or rocks. Tests are typically conducted with cylinders, which are filled with soil and grouted from the top down. These methods do not take into account the impact of boundary conditions on the ultimate injectability.

Thinner soil layers require higher injection pressures at a given injection rate and by decreasing the injection rate the effect of the layer thickness can be somewhat compensated.

A very general rule of thumb used in conjunction with the permeability coefficient of soil has been that:

– k > 1 x 10^{-1} cm/s are injectable with regular cement based (suspension grouts)
– k > 5 x 10^{-3} cm/s are injectable with microfine cement based (suspension grouts)
– k > 1 x 10^{-4} cm/s are injectable with solution grouts
– Reducing the cohesion of the grout mix, while maintaining a stable grout mix, can enhance its penetration.

It is important to establish the upper limit of silt content in a given soil that will still enable acceptable penetration. A soil with a "suitable" d_{10} d_{50} or d_{85} which would lead one to believe that it is perfectly injectable may be found in the field to not be injectable with microfine or suspension grouts due to the silt content.

1.2.4 *Grout mix design*

The choice of grout type is a function primarily of the aperture of the rock joints and cost for grouting projects that involve filling fissures. The use of stable grouts formulated with locally available ordinary Portland cement is recommended. Based on experience, the aperture of joints that can be grouted with ordinary Portland cement is:

a) 500 microns without special care.
b) 400 microns with extra care using high quality grout.

The groutability of fine cracks is related to the width of the crack and the grain size of the grout material, expressed as a groutability ratio for rock in the following formula:

$$Groutability\ Ratio = \frac{Width\ of\ \ Fissure}{D_{95}\ of\ \ Grout} \tag{5}$$

For groutability ratios greater than 5, grouting is considered consistently possible. For groutability ratios less than 2, grouting is not considered possible.

The D95 and practical grouting range for various grouts are provided in the following Table 1.

Table 1 Joint aperture range for various cement grouts

Cement grout	D95 of grout	Practical joint aperture
Ordinary Portland	80-100 microns	>400 microns
High Early Strength	40-60 microns	>200 microns
Microfine cement	10-20 microns	>50 microns

For each application of grout, it is necessary to select a starter mix, and then as grouting of the hole proceeds, decide whether to thicken the mix during grouting. Choosing a good starter mix will come from experience at a particular site after several holes have been grouted. At most sites a good starting point is a 2:1 (water ÷ cement by weight) mix.

Table 2 Suitable grout vs. permeability

Lugeon value	Permeability	Grouting material
4	5.2 x 10^{-7} m/s	Microfine cement grout
8	1.0 x 10^{-6} m/s	Microfine cement grout
15	2.0 x 10^{-6} m/s	Microfine cement grout
30	3.9 x 10^{-6} m/s	Cement rout with stabilizing additives
60	7.8 x 10^{-6} m/s	Cement grout with stabilizing additives
120	1.6 x 10^{-5} m/s	Cement grout with thixotropic additives

1.2.5 *Injection pressure*

A long standing rule of thumb for calculating the maximum allowable grout injection pressure has been 1 lb/in^2 per foot of packer depth in rock (22.59 kN/m^2 per meter of packer depth in rock) plus half the depth of overburden. This rule is based on the weight of rock and overburden directly above the grout hole. While this approach to determining maximum injection pressures may be applicable when grouting in poor or unknown geological conditions at shallow depth, 10m or less, it is felt to be too conservative for most other rock grouting. To reinforce the philosophy of using higher injection pressures, the European rule of thumb for safe grouting pressures is 1 kg/cm^2 per meter of packer depth. This is about four times the US rule of thumb.

1.2.6 *Completion & Refusal criteria*

Grouting may be continued to absolute refusal at the maximum grouting pressure, although this is not usually done. There are two methods that are most frequently used to determine when grouting is complete. One specifies that grouting shall continue until the hole takes no grout at three fourths of the maximum grouting pressure. The other requires that grouting continues until the hole takes grout at the rate of 30–40lt or less in 10 min measured over at least a 5-min period. This is often modified according to the mix and/or pressure applied.

If there is doubt about the completeness of treatment in any zone or area, a check hole or holes should be drilled. However, a quicker and less expensive check can be made by drilling and pressure testing of another grout hole. If tight when pressure- tested with water, the rock is satisfactorily grouted; if the hole takes water, additional grouting is indicated.

1.3 *Corrections & evaluation actions during field trials or systematic grouting*

It is well known, that there is not any grouting program worldwide established in the form of certain preliminary investigations and technical guidelines that was not finally subjected to significant modifications during the actual evaluation process and the exploitation of the results of the learning curves achieved on site. There are always significant control or even reset actions that are required for optimizing the grouting process efficiency and productivity as a whole, mainly taken by the on site engineers responsible in close cooperation with the designer, by taking into account the changeable prevailing conditions and the heterogeneity of the ground as well as the equipment and mix design possibilities. The contractor must establish a reliable monitoring and evaluation engineering team officially responsible for the decision making required at a daily basis on an immediate response on site, in conjunction with the systematic on site involvement of the designer in all the critical steps or cases actually anticipated.

Each grouting program has unique characteristics because of its immediate connection and interaction with the geological and geotechnical profile of the region in interest.

For instance, an ascending (up-stage) grouting method should change if an unstable to drilling ground layer causes significant problems to the drilling crew and the productivity. Possible actions are to reduce the borehole’s diameter if possible or to apply a descending (down-stage) grouting method.

Cataclastic rocks, breccia or conglomerates usually reduce the lifetime span of packers. So, it’s common for packers to be destroyed because some sharp rock cuts its protective rubber cover. Reducing the borehole’s diameter usually helps because there is not much movement and friction of the packer’s surface on the borehole wall during its swelling.

The most significant part of the grouting design is the determination of the completion and refusal criteria that the grouting engineer would apply on site. A successful choice of these parameters can judge the success of the grouting program as a whole. Usually, the completion and refusal criteria consist from a volume and pressure-time control guideline. If the completion criterion is achieved then grouting has to stop or the grout mix to change. So, a volume control guideline has to take into consideration the agitator’s capacity. This is critical because in this case the wastes are eliminated and this is of major importance especially in an urban environment. A pressure-time control guideline has to be calibrated according to the response of the rockmass in grouting as well as to the productivity targets of the project. So, if the time to reach the refusal pressure is significant and the pressure rises asymptotically in relation to time then the criteria should be calibrated accordingly.

1.4 *Usual problems during the application and evaluation of a grouting program on site*

Besides to the equipment capacities, the expertise of the crew and the grouting engineer is the most significant factor for the success of the grouting project. The readiness, the ability to judge and evaluate the design on site as well as the ability to translate and sense the grouting process are necessary characteristics that a grouting crew should have developed. A new learning curve is needed and for this reason the designer and the grouting engineer on site must be in close contact. The designer knows the marginal values and acceptance limits of each critical parameter. The grouting engineer on site is usually stressed by the need for organizing the optimum production system in relation with quality control aspects.

Usually, parameters such as, the completion and refusal criteria, the extent and sequence of drilling and grouting as well as the management of grout mixes (quantities, additives, time of preparation) and applied pressures are major subjects for discussion and calibration during the advance of the grouting project. However, the above-mentioned parameters are also the tricky subjects where misunderstandings occur and are not always applied strictly according to the initial design provisions.

2 SPECIAL APPLICATION OF GROUTING DESIGN IN ATHENS METRO

2.1 *General description*

The present paragraph refers to the execution of grouting works for water control purposes of a ventilation shaft, the access gallery and the chamber in the area of a running shallow track tunnel, for the construction of the western extension of Athens Metro Line 3, with conventional (NATM) method (Fig. 1). The basic demand of the design is to use construction methods to achieve all the demands of settlement requirements and to reduce or eliminate water inflows to acceptable limits, which will permit the safe and feasible construction of the access gallery and chamber. The overburden thickness is approximately 22m.

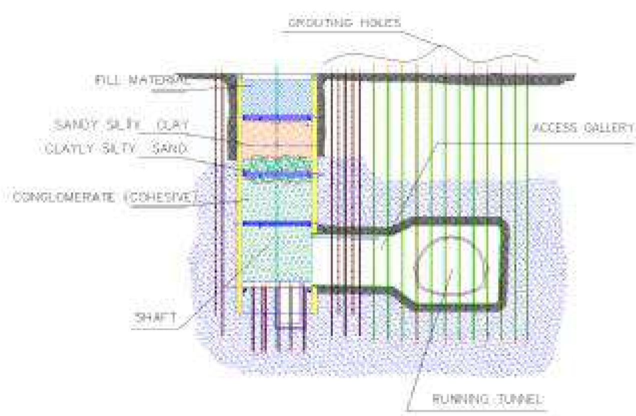

Figure 1. Longitudinal section of the shaft, access gallery and chamber with the running tunnel.

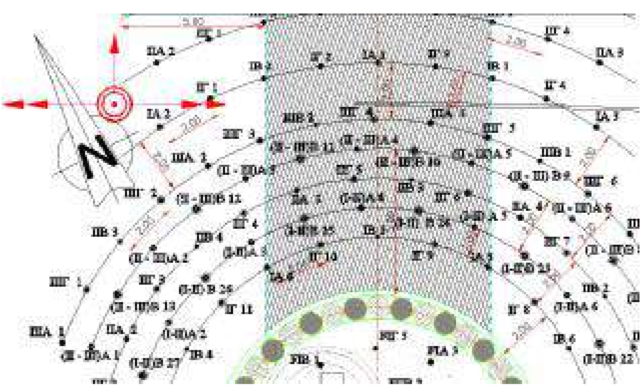

Figure 2. Plan view of a typical borehole grouting pattern around the shaft and the access gallery.

The main geological formations encountered in the area of interest, consist of (from ground surface to depth) alluvial deposits (mainly clayey gravels and sandy clay gravels), conglomerates and breccias ranging from loose to cemented and cohesive, sandy and clayey marls with sandstone intercalations and Athens schist as the main underlying bedrock.

The whole geological background is characterized by considerably high heterogeneity due to the frequent alterations and erratic lithological, petrographical and stratigraphical structures and characteristics of the main horizons by taking into account almost all the existing types of water permeability origin, ranging from the alluvial nature of the soil and soil like formations, which is mainly governed by the grain size distribution and the cohesion, up to the fissured rock properties of the highly cemented conglomerates, which are mainly determined by the joint properties and the composition of the matrix material. It is obvious that several modifications and differential arrangements have to be applied, especially in the "grey" (boundary) areas between soil like and weak rock nature of the conglomerate formation, which actually governs the response of the underground excavation and temporary support of the underground opening, with regard to the underground water inflows, seepage and piping phenomena.

Unexpectedly high ground water inflows were observed during the construction (January – February 2003) of the shaft (approx. 220 m^3/h, with very little water table drawdown).

The conglomerate is usually relatively well cemented and its permeability is high, mainly due to the existence of open cracks, fissures and other discontinuities providing selective flow paths. The large thickness of the conglomerates results in a high "average" permeability of the aquifer.

The hydraulic parameters assessed for this region gave the following values:

Permeability $k = 1\times10^{-3}$m/sec, Transmissivity $T = 8\times10^{-3}m^2/sec$ to $3.5\times10^{-2}m^2/sec$, Storativity $S = 0.04$.

The above values show that the water inflow in the area of the tunnel face is too high and well beyond the inflows (Figs 4-5), which can be reliably controlled by drilling drainage boreholes ahead of the tunnel's face. In addition to that, the pumping tests have shown that the drawdown cone is very sharp, which indicates that only few metres (3-5 metres) away from the tunnel face the water pressures will be practically hydrostatic, thus creating a large hydraulic head which can cause face blow-outs and collapse. Such events can occur in areas where the conglomerate is locally less cemented resulting in lower shear strength. A typical case can be a block of less cemented conglomerate having a width of 2-5 metres ahead of the tunnel face, with better-cemented material beyond that distance. The high water pressures in the more competent material can cause blowout of the less competent conglomerate at the tunnel face resulting in an extended face collapse.

2.2 *Grouting programme*

Since the shaft has already been driven up to its final bottom level, all the grouting work was performed from surface. Several preparatory works took place i.e. reestablishment of a surface collar plug in the ground surrounding the top shaft level by placement of 20cm of lean concrete (B10), use of thrown bulk sand in the shaft, for acting due to its own weight as a natural plug on all the existing discontinuities, cracks of the shotcrete and the existing open holes. A certain hole pattern was designed, consisting of three main (I, II, III) and two auxiliary [(I-II), (II-III)] rows of holes divided in primary (A), secondary (B) and tertiary (C) holes (Fig. 2).

The final spacing of the main rows of holes was 2m×2m and by taking into account the installation of the auxiliary holes (if needed), it was 2m×1m. The borehole sequencing was based on circumferential advance of the drilling and grouting works (inside each row) as the initial priority and after the completion of each row the work was expanded to the next row etc. For avoiding undesired leakages of pressured air and grout due to existing interconnections between adjacent boreholes, these were placed in distances ranging from 6 up to 12m (in general 8m).

The final depth of all the main rows of holes exceeds the depth of the shaft for 6 up to 7m. Certain bottom shaft holes were executed in order to provide the necessary water isolation in this critical area of the shaft. The whole grouting concept was based on the stage up method in stages of 3m of grouting in conjunction with the use of conventional single pneumatic packers for each stage.

The “roof” (upper) grouting stage does not exceed approx. 13m of depth. A variety of grout mixes (w/c) was available (per weight): 2:1, 1,5:1, 1:1, 0.8:1, 0.6:1, with 0, 1 and 2% (per weight of cement) bentonite. For special purposes, a mix of 0.5:1 (per weight) with special additives for antishrinkage effects and easier pumping, was also available, been distinguished by almost similar thixotropic characteristics like the 1:1 mix.

The achieved verticality of the holes was of major importance due to the close arrangement of the holes and any effort was made for avoiding more than 0.5m deviation in the bottom of the holes. Casing of the holes was necessary.

The borehole design has been based on the assumption that the minimum crack sizes of the conglomerate exceed 0,5mm (500μm). Due to the petrographic features of the conglomerate formation, the majority of the existing discontinuities to be grouted was distinguished by horizontal or subhorizontal dips (since most of the existing fissures are erosional, oxidization or of weathering nature), thus making in-shaft grouting a non preferable alternative, in regard to its efficiency. Different mixes were used in different rows of holes or even in different stages in one hole, due to the existing heterogeneity of the fissured conglomerate structure based on the fast and reliable evaluation of the existing grouting results, by taking into account the resultant combinations of grout intake, pressure, time and composition of mix.

Due to the fact that the basic direction of water inflow in the area of the shaft comes mainly from North to South, an asymmetrical hole arrangement was applied in order to balance the applied hydrostatic pressures in the area of the shaft and to contribute to the most efficient water isolation of the structure through grouting. The applied pressures in any case did not exceed 15 bars being necessarily differentiated according to the depth level of each stage and to the mechanical properties of the formations for avoiding uplift phenomena and hydrofracturing.

2.3 *Evaluation of field trial tests and conclusions*

Grouting in this ground profile was successive and therefore necessary for the decrease of the expected water inflows of the underground openings. Moreover, it is emphasized that grouting was applied with the simplest possible method. No special grout mixes were used as well as any sophisticated procedures or equipment. The method applied was an ascending one, with one pneumatic packer and simple grout mixes.

The actual consumption of grout is the only reliable tool for the evaluation of the grout process (Fig. 3). There was a significant consumption of grout while the Lugeon tests indicated a coefficient of permeability of 10^{-6} m/sec and even less. The initial pump-out tests indicated a coefficient of permeability of 10^{-4} to 10^{-3}m/sec, which correlates well with the actual grout consumption.

In order to correlate the grout consumption (which depends from flow Q and time T) with permeability, the following Table 3 is presented.

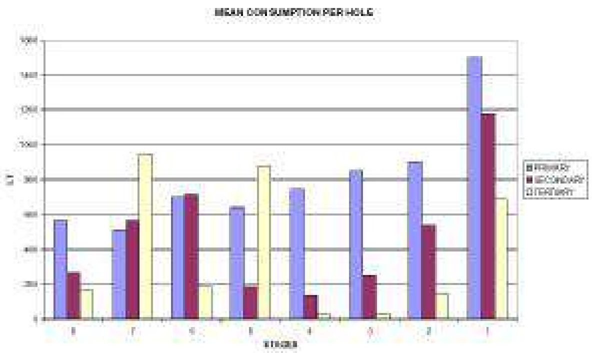

Figure 3. Mean consumption of grout per hole for each stage for all the executed primary, secondary and 8 tertiary boreholes. Average values, Primary = 802lt, Secondary = 479lt, Tertiary = 384lt.

Table 3. Injectability formula $Peff = \frac{Q \cdot Vi \cdot di}{2 \cdot k \cdot v \cdot e} \ln\left(\frac{R}{r_o}\right)$

	Injectability formula			
Grout injection rate	Q(lt/sec)	35	35	35
Kinematic viscosity of grout	Vi(sec)	47.25	47.25	47.25
Viscosity of water	v(sec)	35	35	35
Specific gravity of grout	δi(kN/m^3)	14.02	14.02	14.02
Hydraylic conductivity of soil with water	k(m/sec	10^{-5}	10^{-4}	10^{-6}
Thickness of soil horizon	e(m)	3	3	3
Constant depending on soil type (1-10)	R	5	5	5
Radius of borehole	r_o(m)	0.05	0.05	0.05
Grout pressure	Peff(kN/m^2/m)	848	85	8477
	Peff(bar/m)	8.48	0.85	84.77

Table 3 shows that in order to achieve a mean value of flow 35lt/min and pressure 8.5bars (the mean values used for grouting in situ), then the permeability coefficient should be 10^{-5}m/sec and higher. The above value of effective grout pressure is directly proportional to the grout spread in the soil and therefore the effective groutable radius. The effective pressure presented to Table 3 corresponds to 1m radius which is the spacing interaction between primary, secondary and tertiary boreholes.

Furthermore the specific grout consumption (volume of grout/volume of grouted region) taking into consideration only the primary and secondary holes reached the value of 3.7%. The specific grout consumption for the primary holes, was 2.5% and for the secondary holes 1.2%. The significant decrease of the grout consumption between the primary and secondary holes reaching the 50% reduction is a robust factor for the success of the grouting programme.

Figure 4. Shaft during pump-out test.

Figure 5. Water inflow from drainage holes in the shaft.

REFERENCES

Omikron Kapa Consulting Ltd, July 2003, Technical Report No 3AW3CW395R905A, Prof. Daniel – New detailed final design of grouting works for shaft, access gallery and associated works.

Omikron Kapa Consulting Ltd, March 2004, Technical Report No 3HW3CW401R901A, Evaluation report of investigation grouting results and testings in Prof Daniel shaft.