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Metrolink Permeabilities Targeted with Grout Bottom Plug to Avoid Upwelling in Station Boxes

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Contents

1. Technical Note on Metrolink Permeabilities Targeted to Avoid Upwelling in Station Boxes.

1.1 Overview

The station boxes are designed to ensure drawdown of the water-table will not take place outside of the station box. Modelling (EIAR Appendix A19.8 2D Plaxis report) has confirmed that grouting procedure and permeability ((permeability of 10^{-8} m/sec) will insure there is no measurable impact on the surrounding water table and as such no settlement in surrounding soils.

This note outlines the grouting methodology and also justifies the permeability (10^{-8} m/sec) targeted for avoiding any upwelling through the placement of a grouted bottom plug, if necessary, before the placement of the station slab.

1.2 Grouting Design Summary.

The EIAR has outlined the process of grouting in section A5.12 (Diaphragm wall (D Wall) and Toe grouting). The baseline assessment (ground conditions) of hydrogeology and modelling of seepage rates is presented in Chapter 19. This chapter also includes the Plaxis model methodology and results in Appendix A19.8). Seepage rates (with and without planned grouting) are presented based on a detailed understanding of the geology and hydrogeology at each station based on extensive site investigation including pump testing.

Chapter 19, Figure 19.20 presents a schematic of the grouting design. The design foresees the D-Wall and Toe grouting being augmented by placement of a grouted bottom plug as required.

The D-wall will be installed to a depth of 2 metres below bedrock interface followed by installation of grouting (toe grouting) to a further 5 metres below ground. The D-Wall and Toe grouting will be installed prior to any excavation and will not involve any dewatering and therefore no lowering of the existing water table will take place.

The grouting essentially cuts off any groundwater pathway within the Glacial Tills, Base of Drifts, Weathered bedrock and fresh rock fissures. Thus, the settlement assessment (Building Damage Report A.17) is consistent with a groundwater level which is not drawn down and is constant with the natural fluctuations.

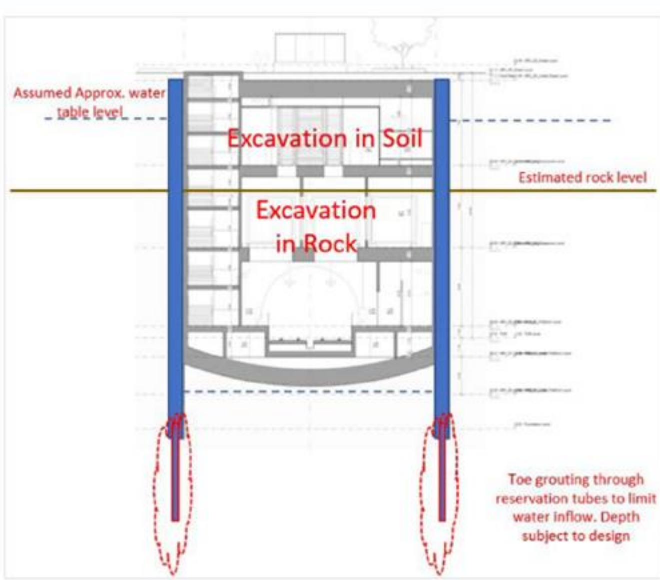

Diagram 19.20: Toe Grouting to Limit Groundwater Inflow into the Excavation through the Base

Detailed design for grouting is included in Appendix A5.5 including section 5.4. “Grouting below Diaphragm Walls”, where the permeation grouting is anticipated to consist of the drilling of a number of boreholes through reservation tubes cast into the diaphragm walls as illustrated in Figure 5.12, followed by the staged injection of cementitious grout covering the required depth, injecting to a pre-set target pressure.

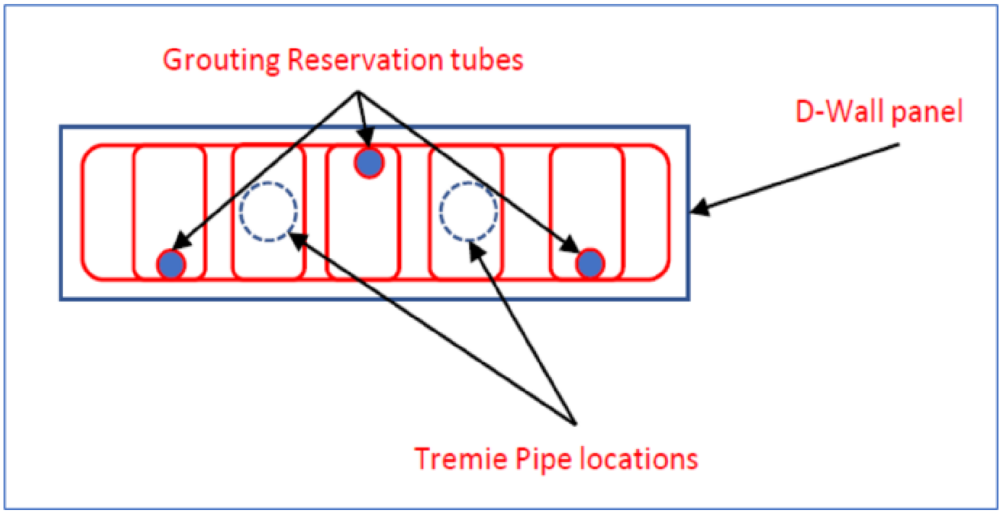

Figure 5.12: Grouting Reservation Tube layout in a Diaphragm Wall Panel

Mitigation design includes installation of monitoring boreholes around below ground stations and monitoring of the local water table conditions for a period of one year prior to commencement of excavation. Boreholes will be installed within each station excavation (after the installation of the D-walls and Toe grouting) to facilitate pump testing which will confirm seepage rates post and confirm the requirement for a grouted bottom plug at each station box.

1.3 Summary of Geology and Hydrogeology

EIAR Appendix A19.8 2D Plaxis model is calibrated with site specific geotechnical data collected for each station. This includes thickness of each layer, the location of the groundwater level and the geotechnical parameters where, permeability has been properly defined.

- Made ground (QX): for this layer a typical mean representative value would be around 2 m, with a permeability of $7.65x10^{-7}$ m/s.
- Brown and Black Boulder Clay (QBR & QBL) is the most common superficial strata present along the alignment. The thickness is quite variable. In addition, two geotechnical units exhibiting granular behaviour have been established (QBL/QBRs<10m and QBL/QBRs>10m)
  - Brown & Black Boulder Clay (QBR & QBL<10m) with permeability of $7.62x10^{-}7$ & $7.21x10^{-7}$m/s respectively.
  - Brown & Black Boulder Clay (QBR & QBL>10m) with permeability of $64.x10^{-6}$ & $7.15x10^{-7}$m/s respectively.
- Base of drift deposits with top weathered rock (BoD). It is mainly composed of sand and gravel layers with erratic boulders of diameter more than 250 mm. Glacial sediments were deposited over the topography which was developed thousands of years before the glacial age; for this reason, the carboniferous rocks are weathered in the first 2-5 m. This layer includes material which has a very high porosity and permeability, forming one of the principal aquifers beneath Dublin, with values of 2.90x10-4 m/s.

- Lower Carboniferous Bedrock Formations, consist of:
    - Lucan Formation (CLU). Dark grey-black, fine grained, graded limestone with interbedded calcareous shale with permeabilities of $4.7x10^{-6}$ m/s.
    - Lower part of Malahide formation (CMLO). Argillaceous limestones, nodular wackestones and shales with permeabilities of $1.38x10^{-6}$ m/s.
    - Upper part of the Malahide formation (CMUP). Calcareous shales, siltstones, and sandstones with thin limestones. with permeabilities of $5.79x10^{-6}$ m/s.
    - Waulsortian Formation (CWA) described within the investigations as Micritic Limestone (CWA)" massive and compact, with high RQD, and high geotechnical quality, with permeability of $5{,}69x10^{-7}$ m/s.

In conclusion, all ground properties assessed have been properly assigned to the station models where the following aspects have been considered.

1. The Dublin Boulder Clay (QBL/QBR) is the primary superficial deposit overlying bedrock in Dublin, characterised by a relatively simple microstructure with low water content, void ratio, permeability, and high density. This shallow aquitard is feed by the rainfall or streams water infiltration and determine the location of the water table within the boreholes.
2. The main aquifer where the flow paths run through is considered the Base of Drifts (BoD) and the Top of the Weathered Rock.
3. Lower carboniferous fresh rock formations have been properly assessed with Geotechnical investigations where RMR and RQD indexes has been assigned at each coring.
4. Through the monitoring wells, the ground water table has determined h along the MetroLink alignment and is well located in the Dublin Boulder Clay (QBL or QBR) for each station.

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1.4 Grout Bottom Plug - Basis for Design.

The study conducted using 2D Plaxis models to assess upwelling ratios before and after the construction of the grout bottom plug. This note summarises the information provided, which outlines the permeability target required to ensure the proper functioning of the grout bottom plug and maintain a dry excavation.

During the grouting process at the bottom plug decisions are made according to observations of the site-specific bedrock conditions. The main work is preparing grout mix material, drilling boreholes and injecting grout material into fissures and fractures of the rock mass.

As included in the Design Approaches for Grouting Rock Fractures, Theory & Practice (Jalaleddin Yaghoobi Rafi, 2013), Stille (2012) stablished that the Required Conductivity as well as Required Sealing Efficiency determine Difficulty of Grouting Work.

Required Sealing Efficiency Required Conductivity	<90%	90-99%	>99%
>10^{-7} m/s	Uncomplicated Grouting	Fair Grouting	Difficult Grouting
10^{-7} to 10^{-8} m/s	Fair Grouting	Difficult Grouting	Very Difficult Grouting
< 10^{-8} m/s	Difficult Grouting	Very Difficult Grouting	Very Difficult Grouting

With the process of grouting described in section A5.12 (D-Wall and Toe Grouting) the risk of seepage at the bottom is even further reduced. It is reasonable, therefore, to set the Required Sealing Efficiency on the 90% for achieving a permeability of 10^{-8}m/s.

The Grouting Intensity Number (GIN) concept was first introduced more than 30 years ago by Eng. Lombardi and Eng. Don Deere, for rock grouting in a wide range of rock conditions, from karstic limestone, through finely fissured chalk, to heavily fractured sedimentary and volcanic formations, and have come to value the technique for its simplicity and efficiency, to the extent that it is now a prime consideration when reviewing any rock grouting solution for either block consolidation/impermeabilization, or as a grouted cut-off.

The Grouting Intensity Number (GIN) technique is considered not so much as a method of grouting, but simply as a tool, one of many essential tools used by the grouting engineer to achieve a successful outcome.

The relationship between permeability and Grouting Intensity Number (GIN) can vary based on several factors including the type of rock or soil being grouted, the properties of the grout used, and the specific conditions of the grouting operation. However, in general, a higher GIN indicates a lower permeability.

Intensity	GIN [bar. litre/m]	RMR		RQD	
Very high	> 2'500	81-100	very good	91-100	excellent
High	1'500 - 2'500	71-80	good	76-90	good
Moderate	1'000 - 1'500	41-70	fair - good	51-75	fair
Very low - low	< 500 - 1'000	<40	very poor - poor	<50	very poor - poor

It is important to note that the relationship between GIN and permeability is not a direct conversion, as GIN is a measure of grouting intensity rather than permeability. The effectiveness of grouting in reducing permeability depends on various factors including the volume and pressure of the grout injected, the distribution of injection points, and the characteristics of the formation being grouted. Therefore, it's not straightforward to provide a precise conversion between GIN and permeability in terms of meters per second (m/s).

The following parameters must be determined.

- Injection Pressure. Grout mixtures in each fracture will move faster with increasing pressure but too high pressure will give heave. Depth to fracture (distance from the surface), fracture orientation, rock quality and depth of grouting are important factors in deciding the grouting pressure.

- Stop Criteria. Stop criterion can be set in the way that penetration of the smallest grout-able fractures shall reach at least up to halfway between the boreholes (Gustafson & Stile, 2005).

- Borehole spacing. Boreholes spacing can be used as a tool besides grouting pressure and material properties to reach desired spread.

- Grouting Time. The grouting time is correlated to the rock mass joint system situation. In this respect, in blocky rock mass with wide joints, grouting for long time led to over spread of grout while in crushed rock mass with narrow joints there is low risk of over spread and increasing grouting time improve the sealing efficiency of performed job.

- Fracture aperture. One of the important characteristics of the joint that affect grouting design is the aperture size. According to Fansson, et al. (2012) fracture aperture describes behaviour of grouting as wall as determining penetrability and penetration length.

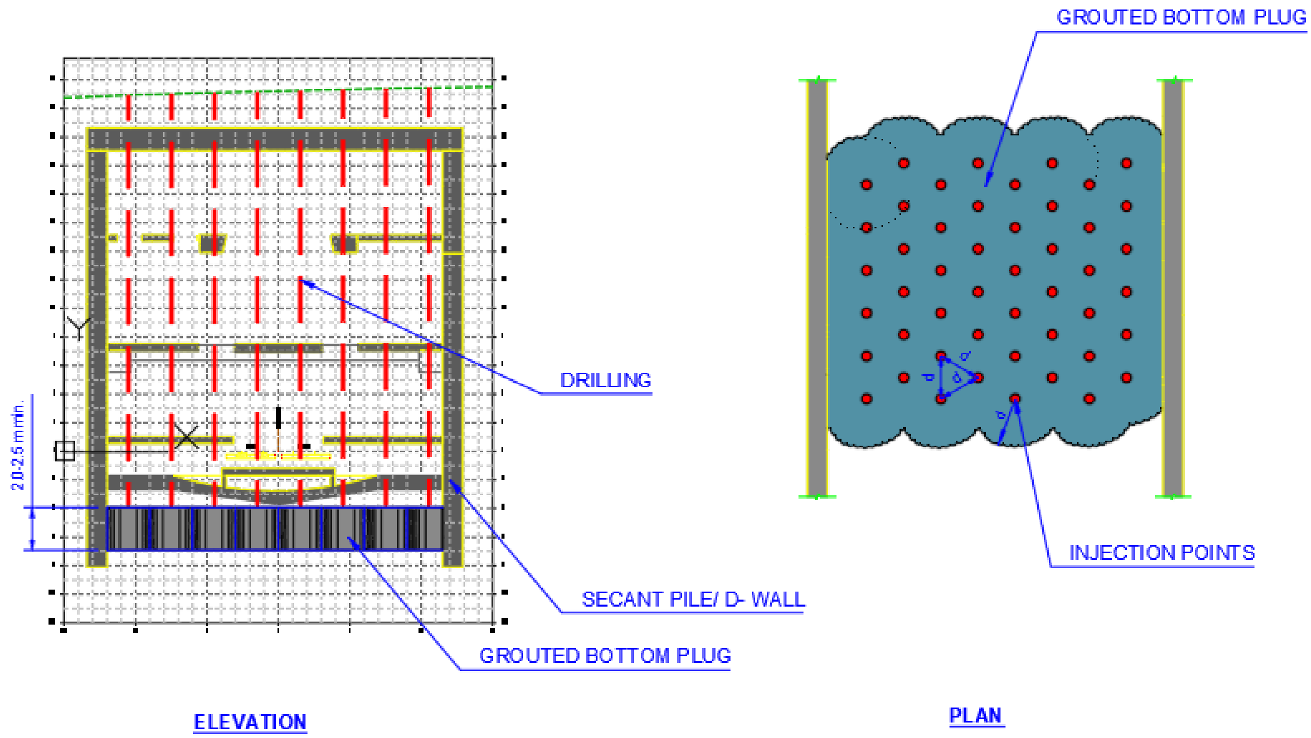
Both Jet-Grouting and Rock Sealing Injection Technique are normally adopted in a variety of geometrical configurations to ensure provisional or final earth retaining and waterproofing functions at the bottom and walls of excavations (e.g., Balossi Restelli et al., 1986; Santoro & Bianco, 1995; Sondermann & Toth, 2001; Miyasaka et al., 1992).

Since permeability of jet grouted elements or of jet grouted structures is an important concern, the design shall state parameters to be measured in specified acceptance tests.

1.5 Grout Bottom Plug Design for Permeability Target of $1x10^{-8}$ m/s

Regarding MetroLink Project, in the case of the Jet Grouting bottom plug technique, the basic solution consists in a continuous impervious barrier formed with assemblies of overlapped columns. On the other hand, grout injections have been widely used in sealing underground excavations to prevent water ingress. For example, cement and sodium silicate grout (C–S grout), a typical quick-setting slurry, is becoming increasingly more popular in fracture-sealing grouting operations (W. Zhang, *Shandong University of Science and Technology*. 2018).

Once the dimensions of the plug are fixed, the spacing between columns might be optimised considering the role of geometric defects on the water flow rate passing through the plug.

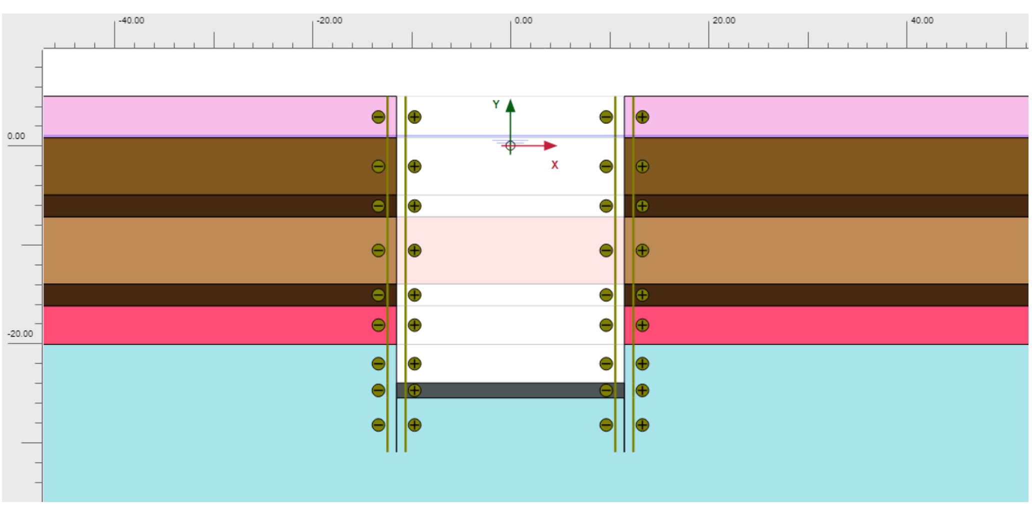


1.6 Grout Bottom Plug applied within 2D Plaxis Modelling

For modelling the bottom plug, within the 2D Plaxis models a new material called "Grout" was created with a permeability of K = 10^{-8} m/s.

Furthermore, within 2D Plaxis Models, a conservative model was adopted with a bottom plug thickness equal to 1.50 m, meanwhile normally the thickness of these plugs is usually somewhat greater (> 2-2.5 m).

The standard geometry for the bottom plugs that has been included in the Plaxis models is shown in the following figure.



As can be observed in the table below, the grout bottom plug reduces the upwelling flow ratios at all stations, involving an effective and appropriate design system. This data is derived based on site specific data derived from site investigation at each station box. The table below provides seepage rates which are conservative as they do not consider the use of toe grouting.

STATION	BEFORE PLACING THE GROUT BOTTOM PLUG	AFTER PLACEMENT OF THE GROUT BOTTOM PLUG
	Qt Plaxis	Qt Plaxis
	(l/s)	(l/s)
SEATOWN	1,14	0,10
SWORDS CENTRAL	0,69	0,09
FOSTERSTOWN	0,26	0,04
DUBLIN AIRPORT	0,38	0,22
DARDISTOWN	0,40	0,01
NORTHWOOD	2,65	0,38
BALLYMUN	3,48	0,34
COLLINS AVENUE	2,32	0,32
GRIFFITH PARK	3,04	0,38
GLASNEVIN	6,95	0,33
MATER	3,47	0,42
O´CONNELL STREET	2,46	0,37
TARA	2,91	0,42
STEPHENS GREEN	3,00	0,38
CHARLEMONT	2,34	0,32

1.7 Conclusion

As described, the D-Wall and Toe grouting will be installed prior to any excavation and will not involve any dewatering and therefore no lowering of the existing water table will take place. The grouting essentially cuts off any groundwater pathway within the Glacial Tills, Base of Drifts, Weathered bedrock and fresh rock fissures. Bottom plugs made of partially overlapping jet grouted columns can be conveniently used to waterproof station boxes below the water table.

With the process of grouting (D-Wall and Toe Grouting) as described in section A5.12 of the EIAR, the risk of upward seepage at the bottom is even further reduced. Pump testing at the base and monitoring of surrounding water levels will be undertaken to confirm expected modelled seepage rates. If necessary, further grouting via a grouted bottom plug will be undertaken as required until a permeability of 10-8 m/s is achieved.

In accordance with Jalaleddin Yaghoobi Rafi, 2013, and according to the Practical Application of GIN, it is moderately reasonable to set the Required Sealing Efficiency on the 90% for achieving a permeability of 10^{-8}m/s, where the quality of the rock frequently in a fair-good class involves Moderate Intensity of Grouting. This is consistent with the bedrock formations encountered during site specific investigation in the MetroLink Project.

The procedure for installation of the grouting by D-wall and toe grouting will ensure that local groundwater levels are not changed during or post construction. The settlement assessment (Building Damage Report A.17) is consistent with a groundwater level which is not drawn down and is constant with the natural fluctuations.

1.8 Bibliography used

Clif Kettle & Maren Katterbac. 2015. PRACTICAL APPLICATION OF THE GIN (GROUTING INTENSITY NUMBER) CONCEPT.

Jalaleddin Yaghoobi Rafi, 2013. DESIGN APPROACHES FOR GROUTING ROCK FRACTURES, THEORY AND PRACTICE.

EN 12716 July 2019. EUROPEAN STANDARD FOR EXECUTION OF SPECIAL GEOTECHNICAL WORK JET GROUTING

Appendix A. Practical Application of the GIN (Grouting Intensity Number) Concept

Practical application of the GIN concept (Part 1)

Clif Kettle & Maren Katterbach

Designer's overview

The GIN concept is a self-regulating approach of controlling simultaneously both the injection pressure and rate of injection, to avoid a combination of high volumes and high-pressure, whilst at the same time setting defined limits on maximum volume and maximum pressure. In general terms the GIN concept aims to optimize the grouting process. In particular, it aims 1) to grout only where absolutely necessary, in this way avoiding any waste of grout and 2) to use highest practicable grouting pressures without causing any damage, in order to enhance the efficiency and success of the grouting operation.

This concept was first introduced more than 30 years ago by Eng. Lombardi and Eng. Don Deere, with the intention of avoiding damage to the fissured rock formation, whilst greatly improving the efficiency and effectiveness of grouting operations. One of the intentions of the process is to equalise the radius of flow in fissures of varying widths.

Remarkably, with all the advancements in grouting over the last decades, the GIN concept has remained largely intact and has proved to be a reliable tool to manage efficiently the grouting process under varied conditions in numerous projects worldwide. With its well-founded physical basis, its generality, and finally its simplicity, the GIN concept clearly and consistently illustrates that grouting does not, and should not, represent an obscure art.

Contractor's overview

Bachy-Soletanche personnel have been using the GIN concept for rock grouting for more than 30 years in a wide range of rock conditions, from karstic limestone, through finely fissured chalk, to heavily fractured sedimentary and volcanic formations, and have come to value the technique for its simplicity and efficiency, to the extent that it is now a prime consideration when reviewing any rock grouting solution for either block consolidation/impermeabilisation, or as a grouted cut-off.

The GIN technique is considered not so much as a method of grouting, but simply as a tool, one of many essential tools used by the grouting engineer to achieve a successful outcome. As with any tool used in any type of work, it requires understanding, skill, and experience to be able to employ it effectively in the workplace. Furthermore, GIN grouting involves experienced observation and interpretation throughout the grouting programme. Based upon the initial observed results, the GIN value, and the various injection parameters, should be adjusted where necessary during the course of the grouting programme, but thereafter, the objective should be to change as little as possible to maintain a consistent strategy.

The technique has proven itself on worksites where other techniques have failed, and has delivered a high quality of ground treatment in challenging rock conditions, whilst at the same time providing significant economic benefit for both client and contractor alike.

For success and maximum efficiency it is essential that the technique, as with all techniques, is configured to suit the local ground conditions. This may seem obvious, but there have been many cases of specifications and grouting strategies being too rigidly applied, sometimes simply copied from elsewhere, in the expectation that these can be imposed on the ground, and that the ground will comply. Clearly, it will not, and thus this approach is predestined for failure.

Within the Bachy Soletanche group, the GIN concept of fissure grouting in rock is seen as a major advance in the practical application of rock grouting technology. This view is also widely held amongst practising contractors due to the simplification of the core injection process, the self-regulating control of excessive hydro-fracture pressures, and the improved facility for comparison and interpretation of the grout injection data across numerous phases of injection.

On the following pages, some general technical aspects related to GIN grouting will be discussed. In the next Groutline issue (Match 2016), several case histories of projects in which Bachy-Soletanche has been involved are presented.

Technical aspects related to GIN

Basic rules for GIN injection

When it was introduced some 30 years ago, the grouting intensity number was just a numerical value, defined as the product of injected grout volume and applied pressure, GIN = P.V. However, over time, with technological advances and improved field experience of the approach, further aspects related to grouting of fissured rock masses have been developed and incorporated within GIN injection.

Despite various developments, the basic GIN concept itself has remained unchanged across the industry, so that today there is a broad consensus as to what constitutes the essential features of this technique, which can be summarized as follows:

- application of a **single GIN value**, the product Pressure x Volume, which is constant for all stages and boreholes, or (at least) all stages within a given phase of injection, and preferably for the entire grout programme. The GIN boundary curve defines the limits within which injection should be executed.
- application of **a rheologically stable grout mix** whose design and constituents is appropriate for the rock conditions and desired residual permeability.
- use of a **single**, rheologically stable, **grout of low water-cement ratio**. Without this, it is impossible to compare grout absorptions between different phases and injection on a similar basis.
- establishment of a **maximum injection pressure**.
- application of a **minimum effective flow rate**, the equivalent of a refusal criteria, to terminate injections if injection flow rates become too low to be practicable.
- establishment of **consistent injection parameters** for maximum pressure, maximum volume, and uniform injection rate up to the point at which the GIN curve intersects the GIN envelope boundary curve.
- once the injection has reached the boundary curve, a **progressive reduction in the maximum pressure**, following the GIN boundary curve as the volume increases, continuing up to the point at which either maximum target volume, or minimum flow rate, are recorded.
- estimation of the **target volume**, based upon knowledge of the rock formation and the required ground treatment geometry
- plotting of results in the format of an **Equivalent Lugeon**, provides an indirect measurement which allows an approximation of the rock mass transmissivity with water. This can provide a very useful means of observing in real time the progressive reduction in permeability achieved by successive phases of grouting, and even during an individual injection.
- execution of **test grouting** as **direct unambiguous way** to confirm the appropriateness of the mix design and grouting parameters.

With the appropriate planning, equipment, and control systems, GIN grouting is very simple to apply in practice.

The function 'Equivalent Lugeon' has been recognised by many practitioners. This function, calculated on the basis of the ratio between the viscosity of the grout and the viscosity of water, is useful for tracking the evolution of the injection, and the progressive reduction in permeability and transmissivity. It is noted that Equivalent Lugeon is actually a rather inappropriate and controversial name for this parameter, and its use gives rise to misunderstanding and resistance amongst the grouting fraternity. However, since this phrase is already widely used, it is difficult to change its name without generating confusion.

Establishing the GIN value

In general terms the GIN concept helps to obtain the best grouting result with minimum effort. The three underlying parameters to achieve this are the grouting intensity number itself, the maximum pressure and the maximum (target) volume. The GIN value is the product of P, the injection pressure, and V the cumulative volume. It is a constant for any given injection, so that the pressure decreases as the injection progresses. The plot of this function forms a limiting boundary curve, (See Figure 11), which helps to avoid a combination of high pressure and high volume, which could have the potential of damaging the rock formation and risking surface heave. The curve, plotted with P on the y axis, and V on the x axis would at infinity by asymptotic. The extent of the curve is therefore limited by a cut-off at P_{max} (maximum allowable pressure), and a cut-off at V_{max} (target injection volume for the injection stage).

The definition, purpose, and the selection of appropriate values for the GIN, P_{max} and V_{max} are discussed below.

GIN value

The choice of the proper grouting intensity number (GIN) itself is based on both, geological conditions as well as on the project design and requirements.

Before addressing the determinant geological factors, it needs to be noted that the GIN concept has been specifically developed for, and is therefore intended only for, fissure grouting. Like for any other grouting method, special attention must be paid to larger voids, which should be filled with a low mobility grout (LMG) or another appropriate low cost material. This confutes the sometimes still existing misconception that GIN grouting is generally not applicable in limestone. In fact, numerous foundations composed of fissured limestone have been already successfully grouted using the GIN technique. If local conditions, such as the presence of large dissolution features often associated with this type of rock, called for it, a corresponding special treatment to fill these voids was simply adopted.

As with the choice of the proper grouting method, be it fissure grouting or void filling, the selection of the adequate GIN value depends on the local site conditions and the expected final result. Whether the purpose of grouting is to reduce the permeability of the rock mass or to strengthen the foundation, the GIN value on a site can be generally correlated to certain geotechnical zones. Where a site is characterized by highly variable rock mass conditions distinguishing several geotechnical zones, this might indicate a need to apply different GIN values. Generally, for rock masses of good quality, a higher GIN value can be used, whilst in weaker zones of lower strength, grouting should be performed more cautiously, by applying a lower grouting intensity. Table 1, as a rough indication, shows the relationship between some common GIN values, the grouting intensity scale, and in accordance with the above, gives a direct correlation with the geomechanical rock mass quality.

Thus, *Grouting intensity number, GIN ~ Rock mass quality*

changes are to be avoided in order to keep the control and analysis of the grouting as simple as possible. Occasional modifications might be necessary, but should be always based on a rational basis to avoid the grouting becoming confusing and obscure. It is noted that test grouting sections on the site into the actual rock mass allow

Table 1 GIN values with typically correlated geomechanical rock mass quality ranges. Note: the indicated GIN values should be consistent with the project requirements, and borehole location.

Intensity	GIN [bar. litre/m]	RMR		RQD	
Very high	> 2'500	81-100	very good	91-100	excellent
High	1'500 - 2'500	71-80	good	76-90	good
Moderate	1'000 - 1'500	41-70	fair - good	51-75	fair
Very low - low	< 500 - 1'000	<40	very poor - poor	<50	very poor - poor

It is worthwhile noting that, in contrast to many other fields of engineering, the design of a grouting job strongly depends on the rock mass - a natural medium which is not designed by ourselves. As consequence, there is always an unavoidable uncertainty in the definition of the generic mechanical or hydraulic parameters, and the engineer must be aware of this variability when using those parameters as basis for the grouting design.

It frequently occurs that the actual rock mass conditions do not correspond to the ones anticipated and assumed in the initial design phase. If this discrepancy becomes significant, it might indicate the need to change the grouting intensity according to the new findings. Optimally, the GIN value for any given rock formation should be chosen at the beginning of the design procedure, and kept constant for each phase, or for the whole, grouting programme. For some sites the GIN value might require to be adjusted after the initial results are analysed, and possibly even reviewed further as the grouting works progress. However, any abrupt and frequent

to significantly reduce any possible changes of the grouting design to a minimum.

Apart from geological aspects, the general project requirements and grouting objectives should be carefully considered when establishing the GIN value. For many applications, it is possible to assign priorities to certain zones, which are then treated using higher grouting intensities.

Thus, *Grouting intensity number, GIN ~ Project requirements*

Considering a grout curtain, for example, after impounding of the reservoir, a lower water pressure is to be expected in the higher abutments than in the central part of the dam. Consequently, a lower grouting intensity might be acceptable at higher locations. A similar allocation can be made for the constraints related to the hydraulic gradient imposed by the project. The hydraulic gradient in the rock zone to be treated will highest at a shallow depth and diminishes quite fast while depth reaching its minimum in the lowest point of the curtain. Accounting for the fact that in this lowest part the real efficiency of the curtain is by definition zero, the requirements for the grouting intensity might actually also be defined less stringent in this lower zone.

In this way unnecessary grouting in zones of minor importance can be avoided, while the main effort can be focused on the most relevant zones. This helps to significantly optimize the whole grouting process.

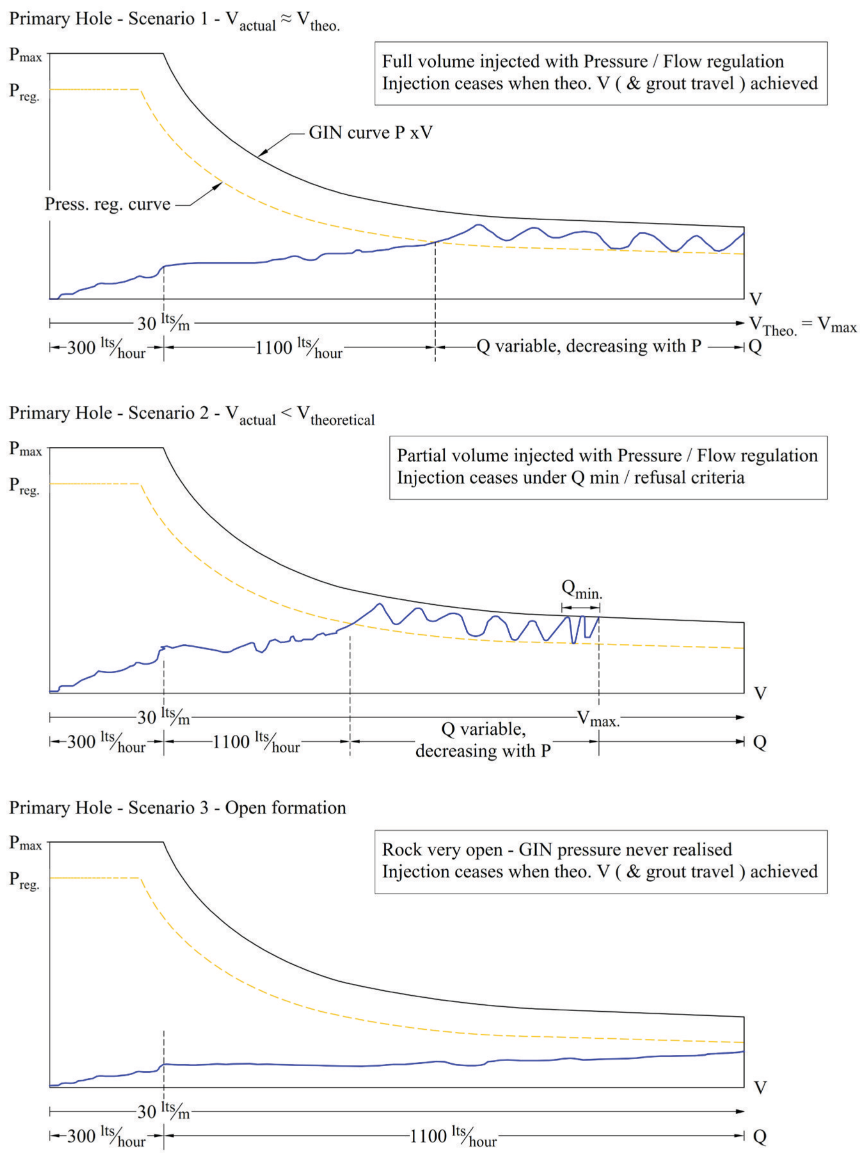
Accordingly, the GIN number itself incorporates both geological and project design aspects. The intensity is therefore directly related to the rock mass quality as well as the relevance of the grouting result for the project.

Once selected, the GIN value controls the injection parameters within a safe working envelope. However, the GIN value needs to also reflect the constraints of the practicable values for the minimum flow rate and minimum controllable pressure of the grout pump (typically 200-300 l/ hr, and approximately 2 bars).

For any given grout type, and injection rate, the evolution of the GIN value over the duration of the injection will depend upon the rock conditions, the grout rheology, and the injection rate. Once the plot of P x V reaches the boundary curve, the injection flow rate, controlled by computer piloted grout pumps, is progressively reduced or increased automatically to maintain the product P x V at or just below the GIN curve until either the maximum target volume is injected, or until the flow rate reduces to a minimum practicable level, at which point the injection is complete.

When establishing a GIN value it is therefore also necessary to consider particularly the likely flow rate during the latter stages of the injection, (approaching the target volume) to ensure that this is compatible with the minimum practicable flow rate for the grout pump, and grout gelling properties, to avoid line blockage.

Application of a single GIN value allows direct comparison of the



P_{max} = maximum safe net pressure for the formation (generally 2-3 times $P_{hydrostatic}$)
P_{reg} = P_{max} < 1-2 bars, the presssure at which continuous flow regulation commences
V_{max} = Theoretical target volume based upon the Σ fissure width/m, hole spacing, cutoff width
Q_{min} = minimum practicable and/or cost effective flow rate for pump
Q_{max} = maximum practicable flow rate for pump 1'500 lt/h, (generally 1'100 lt/h)

Figure 1. Typical examples of the evolution of the GIN value.

graphical and numeric data for individual borehole stages, and for the various phases of injection. It also allows the grouting engineer to rapidly assess and gain a feel for the progress of a single injection and / or the progress of the grouting programme, either by observation of the real-time plot of the GIN curve and the evolving GIN value during the injection, or by visual inspection of the graphical plots on completion of the daily injection programme. Figure 1 gives typical examples of the evolution of the GIN value, within the GIN boundary curve.

Maximum injection pressure

The maximum pressure limit P_{max} serves mainly to select the proper grouting equipment, such as pump, tubes and valves. Like the GIN itself, it should be defined so that it complies both with the rock mass properties and project requirements.

If the purpose of grouting is, for example, the impermeabilization of a dam foundation, the maximum pressure should be chosen according to the expected future water losses and uplift pressures after impounding. It has to be sufficiently high in order to avoid a fissure opening when the reservoir is impounded. A common value for the maximum pressure at the borehole mouth is around 2 - 3 times the future water pressure at that location. Another important aspect to be considered when selecting the proper maximum pressure is the allowable hydraulic gradient of the rock mass. In this: the higher is the hydraulic gradient the higher shall be the maximum injection pressure.

In practice, the maximum pressure can be set in a number of ways. The most reliable method remains certainly the execution of grout test sections on site in the same conditions using the proposed mix design. Another indirect method is to conduct hydro-fracturing tests in the pre-injection investigation boreholes, and to apply a factor of safety to the measured hydro-fracture pressure. In contrast to grouting test sections, for hydro-fracturing tests there is no volume constraint for the water, which is first of all risky. Secondly, acknowledging the difference in water and grout mix, a careful evaluation of the test results by an experienced person is required to be able to extract the desired information for the actual admissible grouting pressures. Alternatively, an estimation may be made with the confining overburden and surcharge pressure, or the limit may even be set on an empirical basis based upon previous experience in similar rock conditions and/or depths of injection.

It is important to recall that the GIN technique is actually self-regulating. Any possible adoption of the pressure with depth to avoid grout outflow or damage due to too higher pressures,

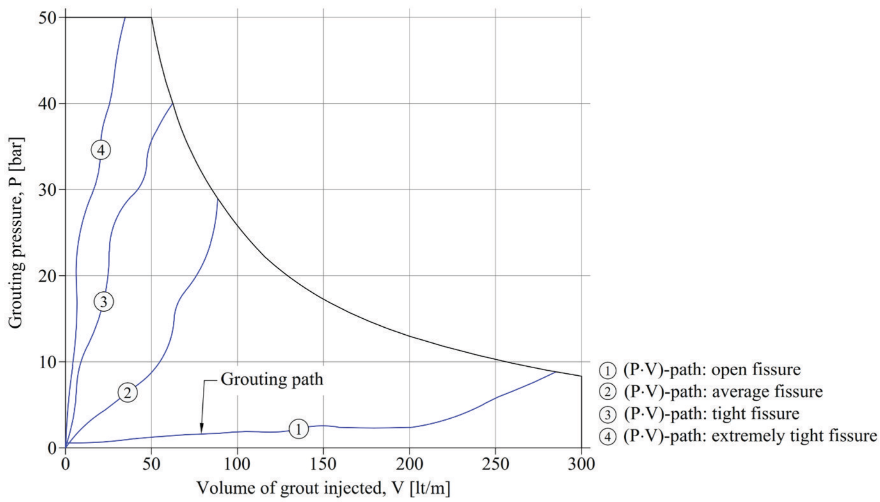


Figure 2. Grouting paths for different fissure openings, illustrating the self-adaptive nature of GIN grouting.

as is sometimes erroneously done, becomes therefore superfluous. Following the GIN concept, the grout takes near the surface or gallery, where the fissures generally tend to be rather open, automatically increase, while the pressure remains rather low. At depth, on the other hand, the openings are generally smaller so that less grout is absorbed. As shown in Figure 2, the grout path in this latter cases (grout paths 3 & 4) is steep reaching quickly higher pressures. Therefore, respecting this self-adaptive nature of GIN grouting, once a certain maximum pressure is defined, it should be kept constant. Changing systematically the maximum pressure in function of depth does not only unnecessarily complicate the whole grouting procedure, but it also carries the risk of stopping grouting before the natural equilibrium is actually reached, resulting in an incomplete execution of the works. The only zone where a certain pressure limitation might be acceptable is the upper 5 m, in order to avoid grout break-out to the surface, especially if grouting is not performed through a concrete slab or similar. To ensure an efficient grout result along the entire borehole length, it is common practice to increase in addition

In this respect, it is recalled that the adequacy of the selected maximum grouting pressure can be best confirmed by several representative grouting test sections.

Maximum grout take (target volume)

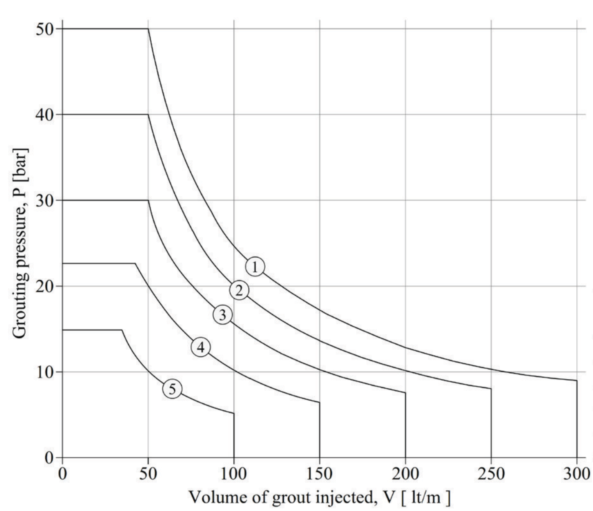
The maximum grout take does actually not present an absolute stop criterion. It rather defines a decision point on whether to

- Continue grouting
- Terminate grouting
- Pause grouting and restart later after setting of grout
- Abandon the hole & drill another one nearby
- Modify the grout mix

In contrast to the grouting intensity number and the maximum pressure, this parameter is mainly defined considering economical rather than physical aspects. A rough indication of commonly chosen maximum grout takes, V_{max}, for certain grouting intensities is given in Figure 3.

Mix design

One of the key aspects of the GIN concept is the use of a single stable grout mix. The mix should be formulated to achieve the specified performance criteria as efficiently as possible (i.e. the minimum number of boreholes, the minimum number of injection phases, and the optimum injection rate throughout each individual injection). Its selection and design is based upon a thorough understanding of the site rock conditions, including fissure widths. It stands to reason that one of the most important aspects actually limiting the groutability is the maximum cement grain size relative to the fissure width. As a general rule, for a fissure to be groutable, its aperture should be at least three times the maximum grain size of the cement. Finally, the mix is also of low water-cement ratio to ensure both long-term strength and durability, and the avoidance of bleed within the voids and fissures of the formation.



Nr.	Intensity	GIN [bar·lt/m]	P_{max} [bar]	V_{max} [lt/m]
1	very high	> 2'500	50	300
2	high	2'000	40	250
3	moderate	1'500	30	200
4	low	1'000	22.5	150
5	very low	< 500	15	100

Figure 3. Typical range of GIN values, as well as corresponding maximum pressures and volumes.

Stable mix

Generally a stable mix is a grout consisting of a cement-based slurry, with additives if necessary, to ensure that no water is expelled from the suspension when injected at pressure (i.e. no pressure-filtration). The stability of the grout ensures that

- the grout rheological properties remain constant throughout the injection to maintain the fluidity and penetration capability
- the progressively reducing absorption of grout can be clearly observed, understood, and measured, as the works progress
- no water filled zones are left

Consistent rheological properties ensure a realistic comparison of grout injection data between subsequent phases of injection, and during the course of a single injection.

This is why the mix should not be fluidified with excess water. Water should be mainly considered as transport medium for cement grains not as physical component of the mix.

Current practice is to employ a grout of low water cement ratio (typically 0.6 -1.1), so that once an individual injection is completed, the potential for bleed in-situ is minimised. It also ensures long-term strength and durability reducing the requirement for successive re-injections.

Single mix

For successful and efficient grouting, it is highly recommended to inject a single grout type with a consistent water/cement ratio for all injections and all phases of the works. Combined with the stability of the grout, a single mix enables the accurate verification and control of the increasing competence and water-tightness of the strata with the grouting works progress.

Recognizing the importance of using a single mix is one of the main aspects where the GIN approach differs from classical grouting practice of 30 years ago. Traditionally, the w/c ratio was lowered in steps (see Figure 4) to increase the cohesion, and in this way lower the normalized pressure, P/c. The introduction of the GIN concept can be said to present a turning point away from this traditional approach of thickening the mixes in steps.

For GIN, (as indicated by the blue line in Figure 4), it is recommended to

- Use 1 unique stable mix throughout the grouting works
- Limit the grouting pressure with increasing volume take
- Reduce the normalized pressure (P/c) by progressively decreasing the pressure.

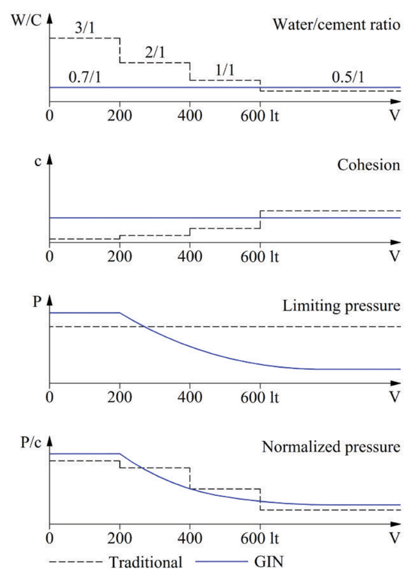


Figure 4. Mix and pressure evolution -Traditional versus GIN grouting.

The use of a single, stable, grout mix avoids many potential errors in mix formulation and in the interpretation of the most relevant injection data - the volume per linear meter injected. In the past, much effort has been expended in trying to accurately convert injected volumes into a dry weight of material per linear metre - a pointless exercise in terms of the specified objectives and technical management of the works, and only of interest for assessing payment.

Multiple mixes, changed during a single injection according to certain volumetric or pressure criteria, have resulted in a flawed understanding of the grout absorption due to the fact that insufficient consideration was taken of the distance over which the grout has been pumped, and/or the volume of grout in the system. There have been sites where mixes have been changed in a rigid succession, when one of the mixes in the sequence has been still wholly or partly within the delivery system, without ever reaching the point of injection. Consequently, the basis for changing the grout mix was flawed, and a calculation of the total dry weight of material injected into a grout stage at the time of refusal was incorrect, so that decisions on subsequent injections were based on a false premise and understanding.

The changing of mixes, in particular the thinning or thickening of the grout mix already in the system, is prone to errors of mix formulation and preparation, whether manually or automatically batched, and this has led to errors in calculating the effects of varying viscosity and head loss, the extent of pressure filtration and sedimentation, and hence in understanding the effective penetration of grout into the formation.

However, the real advantage of a single mix is that it is designed specifically for the rock conditions on site, and particularly for the finer fissures required to be injected to achieve the specified residual permeability,

Another real and valuable advantage is to enable a simple and direct comparison of injections from stage to stage, hole to hole, and between successive phases of grouting. This is invaluable in understanding and visualising in real-time the improving condition of the rock mass and reduction in mass permeability.

Further, providing care is taken with the mix design to control the evolution of the mix viscosity, the gel time, and the setting time, so that the mix remains rheologically consistent throughout the injection, the injection can be used as a surrogate hydraulic or packer test. Real-time plotting of the Equivalent Lugeon can indicate visually the increasing 'tightness' and reducing permeability of the formation as the injection proceeds. Field experience has shown this value - the misnamed Equivalent Lugeon - to be a remarkably good and consistent indicator of the true residual permeability, expressed in Equivalent Lugeons.

In summary, a carefully designed single mix greatly facilitates the work of the grouting engineer and the operatives in the field, has real technical advantages, and provides an accurate and reliable basis for comparison of grout absorptions between different injections stages and different boreholes, and between successive phases of grouting.

Use of multiple mixes, including accelerator and/ or gelling agent

When employing the GIN grouting, the flow rate is automatically controlled to ensure that the function P x V remains within the boundary curve. It follows that towards the end of a given injection, the injection rate may be approaching the limit of the pump, i.e. approximately 180 L per hour.

Considering for example a grout curtain. Due to its geometry and the need to keep a constant length for the grout injection line to ensure constant head loss at a given flow rate, the total volume of grout in the injection system might be as high as 450 L (150 L in the grout line, 250 L in the grout agitation tank, and 50 L in the grout Packer and stage). Clearly, if the new mix is introduced into the system, whether with or without an accelerator, it could take up to 2 hours for this mix to arrive at the point of injection, particularly as flow rates are progressively reduced.

This suggests that the use of an accelerated mix, where the accelerator is added at the mixing station, is not compatible with the GIN idea when following the standard GIN procedure, as this could lead to premature sealing of the borehole before the required volume is injected. Therefore, accelerated mixes might only be applicable when either:

- a pre-injection stage water test indicates an exceptionally high Lugeon value
- there is a high hydraulic gradient across the injection zone, with risk of grout dissipation
- grout is being freely absorbed with minimal pressure increase at the point where the target volume has been injected

at which point a decision could be made to introduce an accelerated mix for a single one-off, non-GIN injection to deal with a significant local feature such as a major fissure or preferred seepage path. Whether an accelerator is added for a single on-off injection, or used systematically in poor or voided ground, the accelerator should be added at the point of injection, via the packer, using a separate supply line for the additive, an in-line mixer, and with a variable flow or proportioning pump to adjust the flow according to the rate of injection to maintain the correct additive proportion in the mix.

The same considerations should be made to changing the grout mix at any point within a GIN injection, since as the injection progresses, and the flow rate gradually reduces, it is highly likely that the new grout mix could still be advancing within the injection lines at the time that the injection is nearing completion. We would strongly recommend therefore the use of a single grout mix throughout any GIN injection, and wherever possible, the use of a single grout mix throughout the whole injection program for a given phase of the works.

Grouting procedure

Tracking the GIN boundary curve

Injection of an individual stage proceeds on the basis of pre-set injection rates, until the value of P x V reaches the limit of the boundary envelope defined by the GIN value. Once the product of P x V reaches the boundary envelope, it is necessary to progressively reduce the flow rate as the cumulative volume increases, in such a manner that the product of P x V remains constant at or just below the limiting GIN value. This operation could be, and has been in the past, carried out manually - but this might be extremely difficult. Current best practice is to employ piloted grout pumps which have the facility to be controlled by computer at all stages of an injection, utilising continuous real-time feedback of data on the pressure, cumulative volume, and flow rate to the grouting computer, in such a manner that in real time the computer can respond to the incoming data and can automatically slow down the rate of pumping to allow P x V to track the GIN curve until one of several criteria are reached.

These are

- maximum pressure - no further injection is possible without exceeding the allowable pressure
- maximum volume - the cumulative volume of grout injected has reached the target limit for the borehole / injection
- minimum flow rate - this is a condition where in order to maintain the plot of P x V coincident with the boundary curve of the GIN envelope, the injection rate falls to a level which is impractical, un-desirable on economic grounds, or poses considerable risk of blockage of the grout pump and/or injection lines.

Consistent injection rate

There is no inherent advantage, technically or commercially, to either client or contractor in injecting grout slowly. Provided that the limiting grout pressures are not exceeded, the aim should be to pump as quickly as practicable. The GIN technique ensures that the limiting pressure is progressively reduced as the total injected volume increases, and this limit is defined and enforced by the GIN boundary curve.

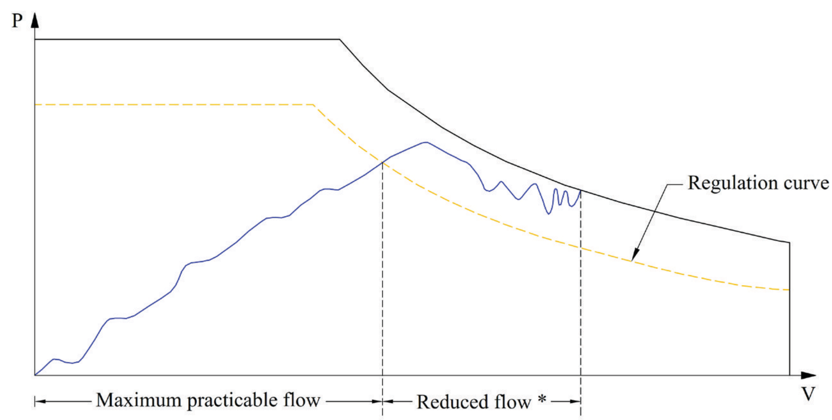


Figure 5. Flow Regulation during the grouting process.

It is prudent to limit the injection rate over the first 15-50 L to avoid immediately reaching the maximum limit pressure, and modern control measures allow for an injection rate of, for example, 300 L per hour until this volume has been placed. Thereafter, the pump can be programmed to seamlessly and automatically increase injection rate up to its practical maximum, typically in the range 1'000-1'200 L per hour. This injection rate should ideally be constant for all injections, and each injection will continue at this rate until the plot of the GIN value P x V approaches to within approximately 1 bar below the GIN boundary curve.

Practical experience has shown that it is convenient to define a certain regulation zone, when approaching the GIN curve, for which a reduced flow rate is imposed. As shown in Figure 5, this zone is bounded by the GIN curve itself and by a parallel regulation curve typically at around 1-2 bars below the GIN value. Within the regulation zone the pump flow rate varies automatically according to the cumulative grout volume and the rock conditions, to maintain the GIN plot within the regulation zone until the injection terminates on minimum flow or maximum volume. The path of the GIN plot and the point at which the GIN plot intersects the boundary curve will be dependent upon the mix, the pump injection rate, and the rock characteristics. Once the cumulative volume injected reaches the target volume for the stage, or the pump reaches its minimum practicable and/or economic pumping rate, the injection terminates automatically. The target volume and the minimum flow rate are all pre-set into the software and cannot be accidentally exceeded.

Once automatic regulation commences, limiting the injection rate, for low grout quantities, for too long a time in this regulation zone, would make the grouting works unnecessarily complicated and uneconomic. There are mainly two options for the termination criteria – either continue grouting at a reducing flow rate until the flow rate reduces to a pre-determined rate (somewhat equivalent to a classical 'refusal' criteria), or the GIN curve is followed until the previously defined maximum volume is reached.

Applying the same criteria to every single injection ensures that the graphical plot for each injection can be compared with that of every other injection, and can provide a great deal of information about progress and success of the individual injection and the progress of the works. It also, together with the constant GIN value and mix characteristics, adds greatly to the substance and accuracy of any numerical analyses.

A key element of this visual inspection is to see on completion of the injection whether the full target volume has been injected, or whether the injection is terminated too early. The grouting engineer can see at a glance what percentage of the target volume has not been placed, and, can make a judgement as to whether this is due to improving rock conditions and reduced transmissivity, or whether the grout mix is inappropriate for the formation, and it allows him to see whether the GIN value is appropriate or not. If he has any concerns on these issues then, of course, he must be prepared to modify the parameter accordingly. However, this should ideally be done for all remaining boreholes. Varying the injection parameters for each individual stage renders realistic and systematic analyses of the results extremely difficult, and prevents the application of some very valuable comparative analyses.

To avoid such an unnecessary complication of the grouting process, it is advisable, in the early stages of the project, to immediately drop back and carry out one or two secondary injections after the first 3-4 primary holes have been completed, to verify that the assumptions made in terms of target grout volume, GIN value, and the optimum injection parameters, are correct. The parameters should then, if required, be modified at this early stage and maintained unchanged wherever possible for the remainder of the works to keep the grouting works as clear and manageable as possible.

Minimum flow rate

The minimum flow rate set for the injection should be a pragmatic decision based upon the characteristics of the pump, technical and cost efficiency considerations, and understanding of the gel and set times of the selected grout, and especially upon examination of the GIN curve and the implied injection pressures at the point on

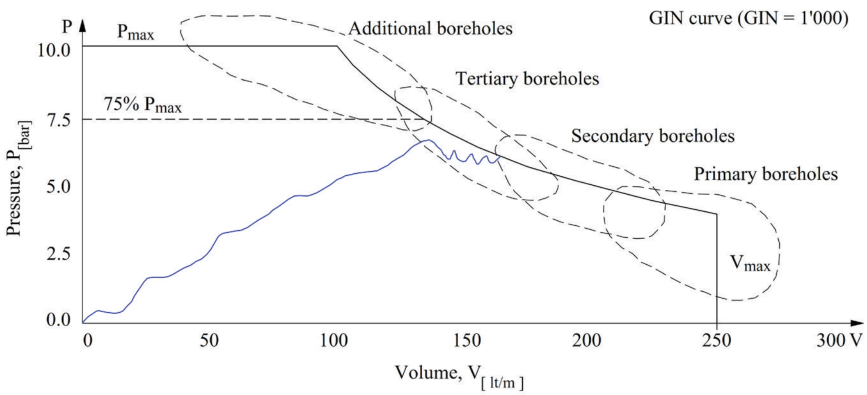


Figure 6. Grouting development from stage to stage and decision criterion for additional boreholes.

the curve where the maximum target volume has been placed. If, at the maximum target volume, either the minimum flow rate defined by the GIN curve is below the minimum desirable injection rate, or the injection pressure is too low for accurate regulation then the design GIN value may have to be increased accordingly.

These considerations need to take into account the experience of the grouting engineer in similar rock conditions and with the characteristics of the equipment being used. There is no technical or commercial advantage in continuing the injection to a point where any further minimal improvement in the rock condition is not justified by the cost of continuing injection, or beyond the point at which there is a risk of grout line blockage or inefficient injection due to a change in the rheology of the grout mix.

Successful completion of grouting

Decision for additional boreholes

In accordance with the rock mass conditions and project requirements, grouting might be systematically executed from primary or secondary boreholes, depending on the hole spacing. The decision for additional, i.e. tertiary or quaternary boreholes is then based on the final grouting pressure reached. According to the GIN concept, and as a result of the split-spacing borehole pattern, grouting is a self-adaptive procedure: first wide fissures are grouted at rather low pressures, before by the following higher order boreholes increasingly smaller openings are filled using higher pressures, as shown in Figure 6.

Consequently, when applying the GIN technique, it can be observed that in general the final grouting pressure does continuously increase from phase to phase, whilst the grout takes are generally decreasing. This development from the lower right to the upper left of the GIN curve, reflects in fact that for each phase the widest remaining joints, not injected during previous phases, are filled. Such grouting results are therefore considered much more meaningful in terms of the actual groutability than any water pressure tests.

Generally, the grouting works are said to be completed if the GIN curve is reached at 50 to 75% of the final pressure. If the grouting path intersects the GIN curve at lower pressures, for example as shown in Figure 16, this phase cannot yet be considered finished and additional boreholes or phases are to be executed. These additional boreholes do not necessarily need to be drilled to full depth. Instead, their optimum depth should be selected based on the grouting results of adjacent boreholes at certain depth intervals. This simple design consideration shows how, by proper integration of the observational method within the grouting procedure, the full benefit of the self-adaptive

Table 2. Guidelines for acceptable foundation permeabilities, according to Houlsby and ranges for typical allowable hydraulic gradients allocated to different dam types.

Dam Type	Curtain	Recommended Lugeon	Typical allowable hydr. gradient **Δ**
Concrete Dams	Single row	3 - 5 Lu	5 - 10
	Multiple row	5 - 7 Lu	1 - 5
Embankment dams with narrow core (earth / rockfill)	Single row	3 - 5 Lu	5 - 10
	Multiple row	5 - 10 Lu	1 - 5
Embankment dams with a wide core & membrane faced dams	Single row	5 - 10 Lu	1 - 5
	Multiple row	7 - 15 Lu	1 - 2
All dam types with foundation material prone to piping or wash-out by seepage in general	Single row	3 - 5 Lu	5 - 10
	multiple row	2 - 4 Lu	5
All dam types, if water loss by seepage becomes relevant for the project, and thereby warrants considerable expenditure to stop it	Single and multiple row	1 - 2 Lu	>25

nature of the GIN concept can be gained, thereby achieving a complete, efficient, cost-effective, and safe grouting job.

Acceptable final permeability

Before defining an acceptable final permeability for a grouting job, one should first think about what might actually be the consequence of the seepage and/or leakage caused by it. There should be a clear differentiation between seepage, which is defined as interstitial movement of water in the foundation, or the abutments, and leakage, which is flow of water through holes or cracks.

Taking a closer look, it quickly becomes clear that foundation permeability may directly affect the stability of the structures to varying degrees, mainly depending on the dam type and height. Whilst for rock fill dams, for example, a certain amount of leakage is common and is rather of little relevance, for concrete dams, in particular if they are large, the same leaks might already significantly impair their safety.

This distinction was already recognized by Lugeon in 1933, when he came up with first indications for allowable foundation permeabilities. He suggested a limiting Lugeon value of 3 for small dams and a Lu < 1 for large dams, respectively. Based on subsequent experience and critical expert reviews, this concept has been further refined over time, in particular focusing on the actual warranty for grouting. Today, engineers commonly refer to the guidelines proposed by Houlsby [3], which can be summarized as indicated in Table 2. In the same table also some typical ranges for allowable hydraulic gradient allocated to different dam types are given.

It is obvious that the highest hydraulic gradients in the rock mass occur in the contact zone at the dam foundation. In the treated zone they diminish with increasing distance from the dam rock mass contact surface at the foundation. Both, the recommended Lugeon and typical allowable hydraulic gradients as listed in Table 2 refer therefore to the zone close to the dam rock mass interface in the central foundation part. With depth corresponding less stringent values (i.e. higher Lugeon and lower gradients) might be acceptable.

These values are obviously intended for guidance only and their appropriateness must be reviewed and verified individually for each project in terms of the project-specific risks. To arrive at an appropriate value, It is important to identify the possibility of encountering particular features and peculiarities of the site by means of thorough geological and hydrogeological investigations, and to evaluate their influence on the permeability on a short and long term. If permeability and geological conditions on one site are highly variable, certain generalizations are necessary.

Relevance of additional testing - pre-injection and post-injection

The determination of permeabilities is essential both to justify the need for grouting, and to evaluate the success of the works executed. Thus, water pressure tests should be performed in exploratory primary holes before grouting and in check holes after completion of grouting in a certain section. These tests are required to compare the initial and the final permeabilities of the rock mass and to assess in this way the grout efficiency and success, respectively.

On the other hand, the execution of pre-injection water pressure tests in individual grout stages during the grouting programme, is not generally necessary, and might negatively affect the already treated rock mass. In addition, such tests during the injection works may not be representative, since there is no direct and/or consistent relation between the penetration of grout and that of water in a rock mass. As shown in Figure 7, a unique wide crack (A) may give the same Lugeon value as a high frequency of fine joints (B), while due to its binghamian rheology as well as the maximum cement grain size, the actual grout take might be much lower in the latter case.

This is why water pressure tests do actually not give any indication on the actual grout takes to be expected. The only reliable way to obtain information on the actual groutability is therefore by the grouting process itself, which should show a consequent pressure increase and volume reduction from stage to stage. The use of Equivalent Lugeon analyses can substitute for pre-grout tests in a given stage, and provide intermediate

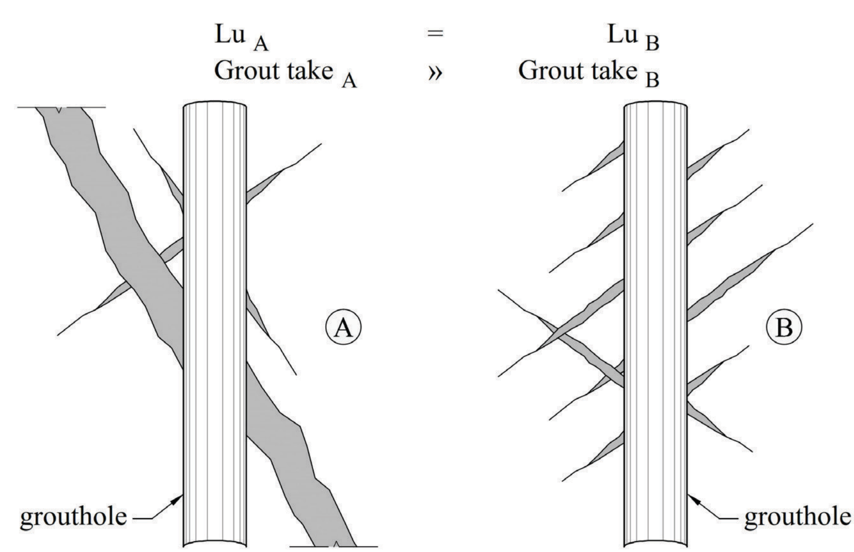


Figure 7. Difference in Lugeon values and grout takes for different fissures.

data on the progress and effectiveness of the grouting programme. For the determination of the actual fissure conditions, that is especially their aperture with reference to the situation shown in figure 7, a complementary inspection by a borehole camera provides important information and is therefore highly recommended.

With the GIN method it is not the Engineer that defines the final pressure, but it is the rock, with its localized (stage) fissured status that decides what will be the final pressure to be reached.

Therefore, water pressure tests before and again after grouting the grouting programme, are allowed and even recommended, in order to evaluate the success of the performed injection works, in terms of the final permeability conditions achieved. To give a true indication of the residual rock mass permeability, post and pre-injection water tests must be executed at significantly lower pressures (normally equal to the predicted groundwater pressures in service) than the grouting pressures. Failure to follow this procedure will mean that the water tests will effectively be testing fissures which have not been affected by the grouting, and at pressures exceeding the service groundwater head, rendering the results un-representative.

In the upcoming Groutline issue (March 2016), the successful implementation of GIN grouting and other above mentioned design concepts in several challenging cases will be presented.

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As promised, below, some of my comments, as a strong supporter of the GIN method.

Being Europeans the authors of this articles, I think they didn't, in my opinion and correctly from their point of view, emphasize a very important point about the GIN method that, again in my opinion, is essential. With the GIN it is possible to use higher grouting pressures than the grouting pressures normally used in North America. Also today, and for important projects, I am reading Grouting Specifications where the grouting pressures are still evaluated with the "infamous" (in my opinion) "Rule-of-thumb" of 1 psi/ft (23 KPa/meter). Parenthesis. [Talking one moment about the "Rule-of-Thumb" (expression still used in our industry), my question is; how Engineers, as we are, can use a "rule-of-thumb" criteria? Are we Engineers or magicians? With all the respect for the magician. Will you be comfortable going to the 54th floor of a high rise building built by a structural engineer with rule-of thumb criteria?]. Close parenthesis.

With the GIN method it is not the Engineer that defines the final pressure, but it is the rock, with its localized (stage) fissured status that decides what will be the final pressure to be reached.

The article gives, additionally, a good approach to use and values of what should be the "consistency" of grouting flow.

Another point that I would like to emphasize is that with the GIN method we can have a better characterization of the status of the rock mass keeping constant as many parameters as we can; specifically flow and grout mix. We avoid consequently, for instance, fictitious "termination criteria" due to change of grout mix, thicker.

Interesting to hear some comments also from you, if any!

As usual, I make the same request, asking you to send me your grouting comments or grouting stories or case histories. My coordinates remain: Paolo Gazzarrini, paolo@paologaz.com , paologaz@shaw.ca or paolo@groutline.com.

Ciao! Cheers!

Appendix B. Design of Jet Grouted Excavation Bottom Plugs

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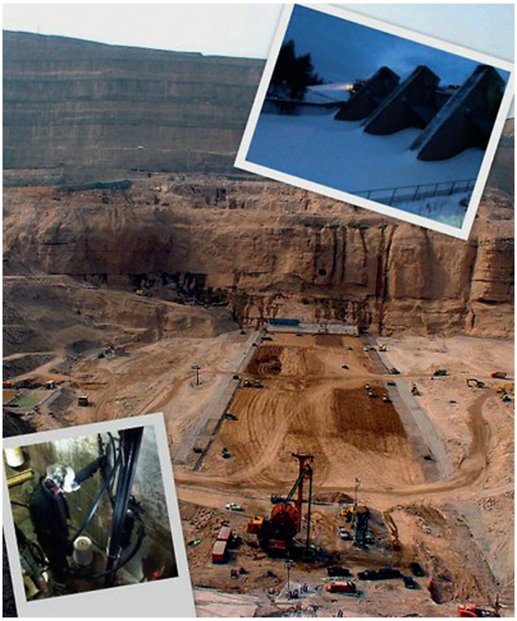

DESIGN APPROACHES FOR GROUTING OF ROCK FRACTURES; THEORY AND PRACTICE

Jalaleddin Yaghoobi Rafi

BeFo Report 124

Cover photo:
Background – Grouting at the curtain of Gotvand Dam, Iran.
Small photos – Grouting at the rock bedding of Laxede Dam, at Harads, Sweden.

STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

DESIGN APPROACHES FOR GROUTING OF ROCK FRACTURES; THEORY AND PRACTICE

Injektering av bergsprickor - Design metoder i teori och praktik

Jalaleddin Yaghoobi Rafi
KTH Royal Institute of Technology

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Preface

Sealing of tunnels and caverns is an important activity within rock engineering and construction, especially in an urban environment and other areas which are sensitive to an affected groundwater. Dam structures founded on rock also need to be sealed, and in both cases, this is widely done by rock mass grouting. Rock grouting is a cost and time consuming process and design improvements should give direct positive effects of these works.

This research aims at verifying the so-called ”Real Time Grouting Control Method” (RTGC method) and also the possibilities to identify hydraulic jacking based on this theory. This is done by case studies from tunnels and dams. The report describes different methods for empiric based grout design and its advantages and disadvantages are discussed. Also, an analytical solution based on theories for a new stop criterion on grout penetration length and to control deformation in fractures and hydraulic uplift is presented.

The present licentiate thesis was performed by Jalaleddin Yaghoobi Rafi at the Division of Soil and Rock Mechanics at the Royal Institute of Technology - KTH, Stockholm, Sweden with Håkan Stille and Stefan Larsson as supervisors. A reference group followed the project and contributed with valuable advice and discussions. This group consisted of Rolf Christiansson (Swedish Nuclear Fuel and Waste Management Co - SKB), Mats Holmberg (Tunnel Engineering), Thomas Dalmalm (The Swedish Transport Administration), Lars Hässler (Golder Associates), Tommy Ellison (Besab) and Per Tengborg (Rock Engineering Research Foundation - BeFo). The research project was financed as a result of a common call by BeFo, The Swedish Research Council for Environment, Agricultural Sciences and Spatial Planning - Formas and The Development Fund of the Swedish Construction Industry - SBUF.

Stockholm in December 2013

Per Tengborg

Förord

Tätning av bergtunnlar och bergrum är idag en betydelsefull aktivitet inom bergbyggande, speciellt i stadsmiljö och andra områden som är känsliga avseende påverkan av grundvatten. Dammkonstruktioner grundlagda på berg behöver också tätas och i båda fallen används injektering av bergmassan. Berginjektering är en kostnads- och tidskrävande process i byggskedet och designförbättringar bedöms kunna ge direkta positiva effekter vid dessa anläggningsarbeten.

Föreliggande forskningsarbete syftar till att bekräfta den så kallade ”Real Time Grouting Control Method” (RTGC-metoden) och vidare att verifiera möjligheten att upptäcka hydrauliskt upplyft (jacking - öppning av bergsprickor) med denna teori. Det utförs med hjälp av genomförda projekt från tunnlar och dammar. Rapporten beskriver olika metoder för empiriskt baserad design av injektering och dess för- och nackdelar diskuteras. Vidare redovisas en analytisk lösning baserat på teoretiska samband för nytt stoppkriterium utifrån inträngningslängd och för att kontrollera deformation i sprickor/hydrauliskt upplyft.

Detta licentiatarbete utfördes av Jalaleddin Yaghoobi Rafi vid avdelningen Jord och Bergmekanik vid Kungliga Tekniska Högskolan - KTH, Stockholm med Håkan Stille och Stefan Larsson som handledare. En referensgrupp har följt projektet och bidragit med råd och diskussioner. Referensgruppen bestod av Rolf Christiansson (SKB), Mats Holmberg (Tunnel Engineering), Thomas Dalmalm (Trafikverket), Lars Hässler (Golder Associates), Tommy Ellison (Besab) och Per Tengborg (BeFo). Projektet har finansierats av BeFo, Formas och SBUF som ett resultat av en gemensam utlysning.

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Sammanfattning

Cementbaserat bruk är vanligt förekommande i syfte att täta sprickor i berget och minska bergmassans permeabilitet. Dessutom utgör injektering av bergmassan vid tunneldrivning en viktig aktivitet i drivningscykeln. Stora mängder injektering används också vid tätning av berggrunden i samband med byggandet av dammar grundlagda på berg. Med hänsyn till den tid och kostnad som injekteringen utgör för dessa typer av projekt är det nödvändigt att kunna förbättra och optimera designmetoderna för injektering.

Vid en framgångsrik injektering är målet att uppnå den eftersträvade tätningen av sprickorna och samtidigt förhindra rörelser i berget på grund av injekteringstrycket. Empiriska metoder har utvecklats i syfte att bestämma tillåtna injekteringstryck, lämpliga egenskaper för injekteringsbruk och stopkriterier. Det finns emellertid otydligheter i hur de ska användas och deras förmåga har ifrågasatts. I dessa metoder har antaganden och kriterier baserats på tumregler och erfarenhet från tidigare projekt. De största osäkerheterna i anslutning till dessa metoder är emellertid kopplade till brukets spridning i sprickan under injekteringen och om sprickan vidgas.

I avhandlingen beskrivs ett teoretiskt underbyggt angreppsättet som är baserad på en analytisk lösning, vilket möjliggör en uppskattning av inträngningslängden för injekteringsbruket i sprickan i realtid, d.v.s. parallellt med att injekteringen utförs. Vidare kan teorin uppskatta trender för flödet av injekteringsbruket. I samband med utvecklingen av denna teori har gränser för elastisk vidgning av sprickorna och brottgränsen för lyftning av bergmassan tagits fram. Detta möjliggör både identifiering av när uppsprickning påbörjas och uppskattning av sprickans vidgning i realtid.

I detta arbete har det teoretiska tillvägagångssättet som kallas för “Real Time Grouting Control Method” validerats genom fallstudier. Egenskaper för använda material, injekteringstryck och flödesdata tillsammans med karaktäristiska geologiska egenskaper har samlats in från projekt utförda i sedimentära bergarter (Gotvand Dam i Iran och THX Dam i Laos) och hårt kristallint berg (Citybanan i Sverige). Teorin har gjort det möjligt att studera flödet av injekteringsbruk och vidgning av sprickor i sedimentärt berg. För projekt utförda i hårt kristallint berg med övervägande vertikala sprickor bekräftar teorin att användandet av högre injekteringstryck är möjligt, vilket kan minska den erforderliga injekteringstiden.

I arbetet har ingen hänsyn tagits till eventuella variationer i brukets egenskaper under injekteringen. Vidare har ingen hänsyn tagits tillvariationer av sprickans aperatur. Trots dessa antaganden har lovande resultat med att verifiera uppskattningen av brukets spridning och risken för lyftning av bergmassan under injekteringen uppnåtts med den använda metoden.

Summary

Currently, cement base grout is used widely for sealing of the rock fractures in order to decrease the permeability of rock mass. Grouting procedure is one of the main tasks in cycle of rock excavation. In addition, huge amount of grout should be used during dam construction in order to seal the bedding and embankment walls. Therefore, considering the effect of grouting in duration and cost of the project, improving the design methods seems essential.

In successful grouting the goal is to achieve the required sealing of fractures while avoiding ground movement due to applied pressure. Empirical methods have been developed to decide the pumping pressure, grout mix properties and stop criteria in order to fulfill requirements of successful grouting but there are ambiguities in using them and performance of them have been questioned. In these methods, assumptions and criteria are based on rules of thumbs and experiences from previous projects. The main uncertainties connected to these methods are identifying amount of grout spread and state of the fracture.

Theoretical approach is an analytical solution which provides the chance for estimation of penetration length of the grout in real time. Furthermore, void filling fracture aperture and trend of the grout flow are estimated. As the development of this theory, elastic and ultimate jacking limits have been established based on the estimated penetration length. Therefore, it is possible to identify jacking of the fracture and estimate the state of the fracture in real time.

In this research work, performance of this theoretical approach which is called “Real Time Grouting Control Method” has been validated through case studies. Properties of the used material, data for pressure and flow in addition to geological characteristics have been gathered from projects in sedimentary rock (Gotvand Dam in Iran and THX Dam in Laos) and hard rock (City Line Project in Sweden). This theory made it possible to observe overflow of grout and jacking of the fractures in sedimentary rock. In place of hard rock with mostly vertical fractures, this theoretical approach confirms usage of higher pressure which will shorten the grouting time.

In this research work, variation in properties of the grout mix during grouting has been neglected. Moreover, orientation of the fracture and its deformation due to injection pressure are not considered. Despite these assumptions, the results were promising and performance this approach in estimation of grout spread and identifying jacking of the fracture has been verified.

List of publications

Paper I: Rafi, J, Stille, H, Bagheri, M, 2012. Applying "Real Time Grouting Control Method" in Sedimentary Rock, in *4th International Conference on Grouting and Deep Mixing*. 16-18 February, New Orleans-USA.

Paper II: Rafi J, Stille H, 2013. Controlling jacking of rock considering spread of grout and grouting pressure, Accepted to be published in *Journal of Tunneling and Underground Space Technology*.

Paper III: Rafi J, Tsuji M, Stille H, 2013. Theoretical Approaches in Grouting Fractures of the Rock Mass: Theories and Applications. Accepted in *the 47th US Rock Mechanics / Geomechanics Symposium.* 23-26 June, San Francisco, CA, USA.

Paper IV: Rafi J, Tsuji M, Stille H, 2013. Theoretical approaches in grouting design: estimation of penetration length and fracture deformation in real time, Presented in *Bergmekanikdagen*, 11 March, Stockholm-Sweden.

1. Introduction

1.1 Background

The goal of grouting is to make it possible for the grout to penetrate enough to seal fractures and fissures and reduce the permeability up to a certain amount. Maximum boundary is to avoid over spread of grout material which guaranty the optimized performance of grouting work i.e. keeping the project in time and avoiding extra cost. A grouting method could in general consist of five main activities: (1) drilling; (2) grouting; (3) waiting; (4) probe holes/water loss measurements; and (5) re-grouting (Dalmalm, 2004).

According to Houlsby (1990), to perform grouting work, set of main steps should be followed.

Investigation: in this step, geology of the area and permeability situation are investigated. These data are the inputs for the grouting design. Extensive investigation reduces uncertainties and result in accurately tailored design to the condition which can save money in long term. In this step properties of joints are studied. Spacing, width and inclination of joints and soundness and strength of rock are among the important geological factors. For controlling seepage, according to Houlsby (1990) knowledge of permeability is essential and it should be described how much seepage can pass through, under standard pressure head. The most important outcome of this investigation is size of cracks. Lugeon test is the most common way to quantify the permeability of rock mass.

Design of grouting: in this stage, with an overview from the data obtained from investigation area, spacing and length of boreholes, properties of required material and injection pressure are decided. Also, decisions are made to fulfill criteria of successful grouting. According to Houlsby (1990) estimation of grout quantities and cost of grouting is another important step in design process.

Execution: after investigating the field and designing grouting work, the executive procedure can be started. Without any theoretical approach, execution is mostly based on rules of thumb and decisions are made according to observations during the process. The main work in this phase is preparing machineries and facilities, preparing grout mix material, drilling boreholes and injecting grout material into fissures and fractures of the rock mass.

Compilation: refusal point has been discussed and several methods and techniques have been suggested (for example Houlsby, 1990 and Lombardi, 1993). Achieving maximum certain pressure, maximum certain injected volume or production of pressure and volume (P.V) are among popular refusal points. Based on theories developed by Gustafson & Stille (2005), it is possible to estimate penetration of the grout mix analytically in real time, thus required penetration length can be set as the stop criterion. The main part of this research work is about evaluation and verification of this theory.

Assessment: the procedure should be assessed to identify if grout has spread enough around the borehole and not more than required. The other considerable issue is the stresses induced to the rock mass due to grouting which may change the flow regime due to induction of new fractures and increasing the size of existing fractures. Prolonging the procedure, the induced excess pressure may lead to larger deformation and in worse case cause damages to the on ground structures. Also, the increase in the size of fracture aperture will lead to lower penetration length than expected which will affect sealing efficiency of the grouting work.

This research study has focused on evaluation and verification of the theories for estimating the state of grout spread and fracture in real time. Refusal point has been synchronized with penetration length of the grout considering the limit of acceptable deformation. Performance of grouting work has been assessed by analyzing pressure and flow data which are indicator of physical grout interaction in the fracture. Furthermore, pumping pressure, borehole setting and material properties can be optimized by using this approach. This study has an impact on grouting works and may lead to reduce costs and shortening duration of grouting process.

1.2 Previous studies

In successful grouting the goal is to have enough spread of grout around the borehole with no undesirable ground movements. To achieve this goal, pumping pressure and refusal criterion are decided based on rules of thumbs and experiences from similar projects. The specification of the maximum pressure is set to avoid undesirable deformations. Also, to avoid over spread of grout, a certain maximum limit for the injected volume is considered. In order to avoid long and undesirable pumping, refusal criterion connected to minimum flow is normally set as well.

Grout intensity number (GIN), introduced by Lombardi (1993), set limitation on product of pressure and volume to avoid expansion of energy which can open up the fractures and may lead to hydraulic uplift. The product of pressure and volume should therefore not exceed a given GIN value in order to avoid such problems.

In mentioned empirical methods, the reason to use volume instead of the theoretically correct penetration was practical (Stille et al. 2012). Recent efforts by Gustafson and Stille (2005) have solved the equations of grout spread by simplifying the existing voids in rock mass to channels or discs which make it possible to estimate grout penetration length and predict grout flow in a fractured rock mass and in real time. This will facilitate the analysis and make a significant improvement for managing grouting operation. According to Gustafson and Stille (2005), for each simplified geological model (1D and 2D models), spread of grout with specific properties and specific pumping pressure in specific time is a unique number. It means that the relative grout penetration, which is defined as the ratio between the actual penetration and the maximum penetration, is the same in all fractures (Kobayashi, et al., 2008).

Kobayashi and Stille (2008) described the use of penetration length as stop criterion as follow: "grouting is completed when the grout penetration of the smallest fracture to be sealed is above a certain minimum value (target value) or before the grout penetration for the largest fracture aperture reaches a certain maximum value (limiting value)". Application of this method validated through case studies with data from tunnels in hard rock of Sweden and pre cambrium rock and depicted in several different publications (Stille (2010); Tsuji, et al. (2012); Fransson, et al. (2012); Rafi (2010)).

To go further, the estimated penetration length has been used to anticipate ground movements. In studies performed by Brantberger, et al. (2000), Gothäll & Stille (2009) and Gothäll & Stille (2010), grout pressure correlated with grout spread and stresses induced in rock mass have been studied and formulated. The analytical solution for estimation of grout spread provides an interesting opportunity for developing the earlier works by using the estimated penetration length to predict deformation of fractures in real time.

Ultimate and elastic limits have been defined based on spread of grout mixture and properties of the rock mass (Stille et al., 2012). This approach enables defining an acceptable deformation limit, thus it is possible to decide the refusal based on required penetration length and the defined limit for deformation.

1.3 Scope of research

In these studies, the objective was validation of Real Time Grouting Control Method as well as verifying possibility to identify jacking based on this theory with data from dams and tunneling projects. For this purpose, application of Real Time Grouting Control Method has been established and software based on this model with better interface and easier functionality has been developed. Performance of this method has been validated through different case studies.

In the studies related to Gotvand Dam project, grouting data and cement material were collected during site visit. Properties of the grout material were obtained by testing the grout mixture in laboratory in Stockholm. In case of THX Project and City Line Project, properties of grout material and recorded pressure and flow data were obtained from the project consultant. Grouting works at some of the boreholes were analyzed and performance of this theoretical approach was validated. Furthermore, to verify the performance of the theoretical approach in identifying jacking, ultimate and elastic jacking limits based on the estimated grout spread have been established and success of grouting work in different boreholes has been examined. Moreover, a methodology proposed to optimize pumping pressure (optimum pressure gives the required spread of grout in shortest time with controlled ground movement) by considering required penetration and acceptable deformation. This study shows robustness of this method in optimizing the pumping pressure, refusal point, borehole setting and material properties which can affect performance of grouting work and cost and duration of the project.

1.4 Outline of thesis

This thesis contains an overview introduction which addresses the procedure of grouting work, followed by the literature review of existing procedure and approaches in practicing grouting work in which, by highlighting obstacles of current practice, the requirement for an analytical approach has been discussed. This analytical solution has been described and evaluated through the case studies in next sections. The structure of this thesis is as below:

Chapter 2

The properties of the joints as well as the requirement of penetrability and its effect on grouting are discussed and design elements which should be decided before grouting and be monitored during grouting are described. Furthermore, empirical design approaches which are instruction of how to set up the design elements in connection with each other to achieve the desired result (enough sealing efficiency in shortest time with the least damage) are depicted and the ambiguities and difficulties of them are discussed. Finally it is shown that uncertainties connected to the empirical design approaches can be reduced or eliminated if the behavior of the grout is studied analytically.

Chapter 3

The theoretical approach which is an analytical solution to establish new stop criteria based on penetration length and to define new limits for controlling fracture deformation and hydraulic uplifts are discussed briefly.

Chapter 4

Geological properties of the rock in studied projects as well as the properties of the used material and stop criteria in each project are shown.

Chapter 5

This chapter contains the summary of the main results from different studied cases.

Chapter 6

In final chapter, the general outcome of the project, limitations with the theory and the future works are discussed.

Results of this research work have been Published and presented in form of scientific papers as below:

In paper I, performance of Real Time Grouting Control method in sedimentary rock has been evaluated. In Paper II, this analytical solution has been implemented to estimate fracture deformation and identify risk of jacking based on the calculated penetration length. In papers III and IV, by using this theoretical approach, performance of grouting work in respect to fracture deformation at City Line project has been examined.

2. Current knowledge and practice

2.1 Investigation

Geology

As the most commonly developed of all structures, *Joints* properties affect the spread of grout and stability of rock mass. Roughness of joint walls, coating and filling, size, continuity, presence and shear strength of the joint are among these factors. Due to uncertainties in geological condition, Palmström & Stille (2010) suggest to establish a site engineering model to consider all the possible geological characteristics of the site and improve the understanding of geology during investigation and construction.

The better understanding from the geology of the area leads to better decisions. According to Palmström & Stille (2010) engineering properties of rock mass depend far more on the system of geological discontinuities within the rock mass than on the strength of intact rock. However, they mentioned that this does not mean that the properties of the intact rock material should be disregarded in the characterization. In case of wildly spaced joints or weak intact rock, the properties of the intact rock may strongly influence the gross behavior of the rock mass.

To precise the assumptions about properties of joints, different facilities and methods have been developed. Mapping, core drilling and logging are the field observations still used in many projects. To get more information about the size and distribution of fractures, Ivanetich , et al. (2012) suggest using Televiewer which is utilizing camera to monitor the wall of the borehole and produce high resolution images. District fracture network which integrates fracture characterization data within a 3D computer simulation is another tool to simulate the flow (Carter, et al., 2012).

The other important characteristic of the joint is the aperture size. According to Fransson, et al. (2012) fracture aperture describes behavior of grouting as well as determining penetrability and penetration length. As a methodology for prediction of fracture geometry, Fransson (2001) used principal steps to characterize and verify the boreholes. It starts with geological mapping which leads to distinguishing the orientation of fractures. Probability of conductive fractures obtained from frequency of fractures, information of the probe hole and transmissivity data in fixed intervals. The model then is simplified by considering the mean and standard deviation of conductive fractures. Thus probable interval and values of transmissivity, specific capacity and hydraulic aperture can be obtained.

Permeability

Permeability is defined as the ability of rock to transmit fluids where a pressure gradient exists and experiments have shown that the permeability of rock masses is primarily dependent on discontinuities (for example see: Wei Jiang, et al. (2009)**)**. According to them, aperture, frequency, orientation and roughness of discontinuities are affecting transmissivity of rock. The transmissivity and the corresponding hydraulic aperture are important parameters since the aperture influences both the penetration length of grout and the volume injected, i.e. the grout take (Gustafsson & Stille, 1996).

Design of grouting work depends on the purpose of the grouting i.e. amount of required sealing efficiency. Houlsby (1990) has considered the value of water, requirement for avoiding piping and type of the dame in distinguishing conductivity requirement (Fig .1).

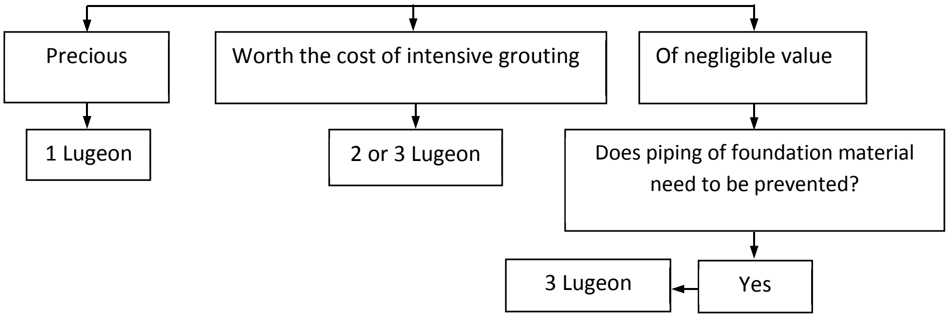

Fig. 1. Guid to the need for grouting at a dam (After Houlsby 1990)

According to Stille (2012) the required conductivity as well as required sealing efficiency determine difficulty of grouting work (table 1).

Table 1 Degree of difficulty as function of required sealing effect and conductivity of the grouted zone (After Stille, 2012)

Required Sealing Efficiency Required Conductivity	<90%	90-99%	>99%
$>10^{-7}$ m/s	Uncomplicated Grouting	Fair Grouting	Difficult Grouting
10^{-7} to 10^{-8} m/s	Fair Grouting	Difficult Grouting	Very Difficult Grouting
$< 10^{-8}$ m/s	Difficult Grouting	Very Difficult Grouting	Very Difficult Grouting

Moreover, pre investigations and observations during grouting work will have larger process in more difficult grouting. In uncomplicated case it is enough to only measure of water ingress to the tunnel while in difficult situation grouting work should be observed with real time application (RTGC). The proposal has been depicted in table 2.

Table 2. Proposal for pre-investigation and observations during grouting work depending on degree of difficulty (After Stille, 2012)

Degree of difficulty	*Pre investigation*	*Observations during grout work*
Uncomplicated grouting	• Establish major geological regimes	• Measurement of the ingress of water to the tunnel
Fair grouting	• Establish major hydro geological regimes and representative values of hydraulic conductivity	• Measurement of the ingress of water to the tunnel • Controlling stop criterion with theory of grout propagation
Difficult grouting	• Establish the hydro geological properties of the regimes like section transmissivity based on WPT • study of fracture transmissivity	• Measurement of the ingress of water to the tunnel • Pre-sounding and water pressure test (or inflow measurements) • Controlling stop criterion by applying RTGC
Very difficult grouting	• Establish the hydro geological properties of the regimes like section transmissivity based on WPT • Study of fracture transmissivity (detailed study of the regimes with core drilling, pressure build up tests and fracture transmissivity measurement can be an alternative)	• Measurement of the ingress of water to the tunnel • Pre-sounding and water pressure test (or inflow measurements) • Controlling both stop criterion and grouting process by applying RTGC • Control hole after grouting

2.2 Design elements

Designing the grouting work is to choosing pumping pressure, material properties and refusal point in the way to achieve the required sealing efficiency with the least damages. Different methodologies have been proposed to decide design variables. In this chapter, the concept of each variable and its effect in grouting process has been discussed. In the next chapter, the practical methodologies to decide these elements have been depicted.

Injection pressure

Deciding the injection pressure in grouting projects has been the point of discussion (Houlsby (1990) ; Lombardi (1997); Gothäll & Stille (2009); Stille, et al. (2012)). The grouting pressure should be designed in the way to not be higher than the minimum rock stress, in order to avoid jacking of joints (Gustafsson & Stille, 1996) as well as fulfilling penetration requirements. Grout mixture in a given fracture will move faster with increasing pressure but too high pressure will give jacking. Depth of fracture (distance from the surface), fracture orientation and amount of injected grout mix are important factors in deciding the pumping pressure.

The maximum applicable grout pressure depends on stiffness of the rock mass and the depth of the fracture. According to Houlsby (1990) the pressure should be adjusted to the maximum limit the foundation can take in order to get maximum penetration. Based on him, the pressure should be limited concerning the quality of rock mass and depth of the fracture. Based on this approach, different graphs have been developed to decide the applicable pressure (Fig.2) .

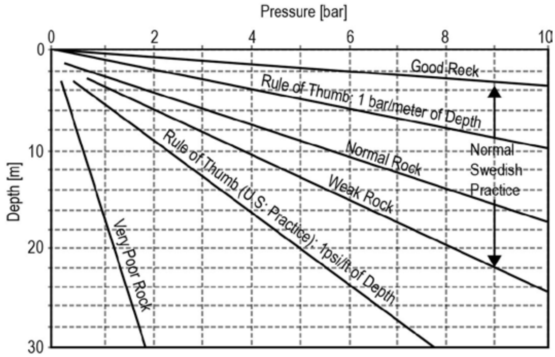

Fig. 2. Grouting pressure according to practice in Sweden and the USA (Weaver 1991). Both rock quality and depth of grouting are important factors in determining grouting pressure.

Stille, et al. (2012) explained that since the good rock mass has both smaller fractures and larger stiffness, in this case higher pressure can be used. In fractures close to the surface and where ground movement is limited, low pressure should be applied. In fractures in deeper zone, (Lombardi & Deere, 1993) suggests using higher pressure to open up the fractures and even induce new fractures in weak rock and fill them fully. This will be only valid in largest fractures and according to Stille, et al. (2012), since the elastic deformation may go back when the pressure is revealed, the smallest fractures can then be unsealed.

Orientation of the fracture is important since it affecting the stresses. In situ stresses in the rock mass are important factors affecting design of grouting. Hoek & Brown (1980) showed that unless at shallow depth, vertical stresses are in fair agreement with weight of the rock at a particular depth. Terzaghi & Richart (1952) suggested lateral stresses to be one third of vertical stresses in a typical rock with Poisson ratio of 0.25. Later, there were reports of measuring horizontal stresses of several times the vertical stresses in Scandinavia (Hast, 1958). Considering inability of rock to support large stress differences, it was suggested that the ratio of horizontal to vertical stress will be close to one over a period of geological time. From the measurements performed by Hoek & Brown (1980), this ratio (k) varies as below:

$$\frac{100}{z} + 0.3 < k < \frac{1500}{z} + 0.5 \tag{1}$$

Where *z* is the depth below the surface. Thus, at the depth less than 500 meters, horizontal stresses are significantly greater than vertical stresses while in deeper zone (over 1 kilometer) the ratio tends to 1. This result has been explained by Hoek & Brown (1980) that the highly existed horizontal stress lead to fracturing, plastic flow and time dependent deformation in the rock which tend to reduce the difference between horizontal and vertical stresses.

From discussion above, in horizontal fractures, lower pressure can lead to dilation of the fracture which may not affect the vertical ones.

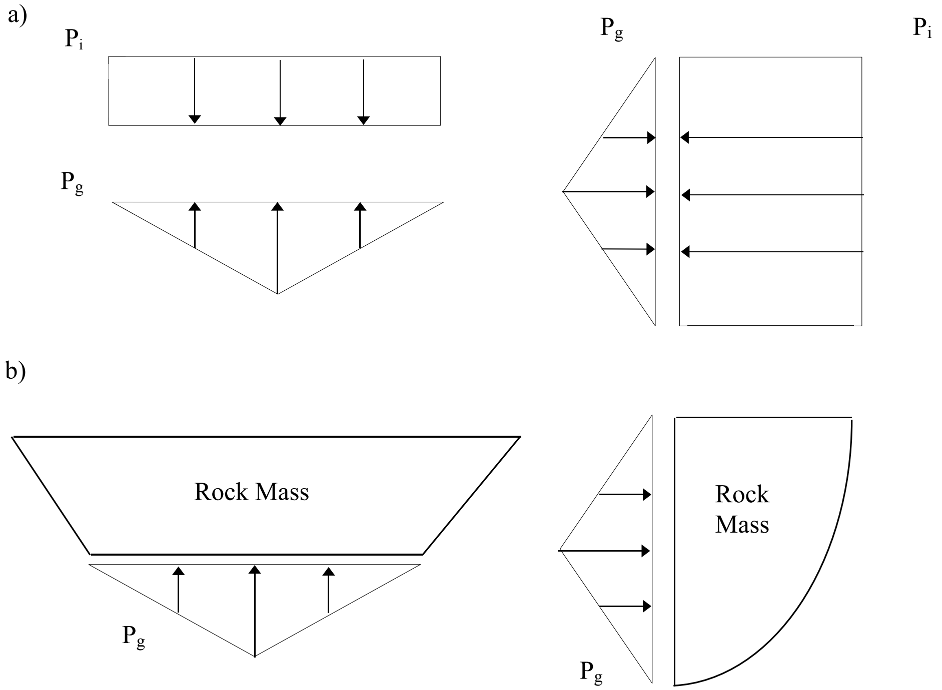

Fig. 3. Grouting in horizontal (left) and vertical (right) joints. a) is the stress situation and b) is the failure at ultimate limit.

At small penetration length and since pressure diminish rapidly as it spreads away from the borehole, the total uplift force may be in such cases much lower than the overburden even if the pumping pressure exceeds the overburden (Lombardi & Deere, 1993). Thus in order to achieve faster and better penetration in fine fractures, high pumping pressures can be used (Gothäll & Stille, 2010).

Stop criteria

Deciding the stop criteria to fulfill the requirements of grouting work is a challenging task. Experiences with grouting have shown that grout has to be injected to a full stop ("refusal") in order to obtaining a successful result. At the point of refusal, the injection pressure is balanced by the shear stress towards the fracture walls. Higher pressure should be applied to continue grouting spread. On the other hand, injection of larger volume of grout in larger fracture may not be necessary and would be waste of grout material. Thus the stop criteria should be set in the way to fill up the smallest fracture while not over spreading in largest ones. According to Fransson (2008), the penetration length for smallest fracture is sufficient to theoretically fill the fracture between boreholes including and overlap to increase the chance of sealing. Therefore limiting grouting volume (or the grout spread, as will be discussed later) is one of the stop criteria.

Stopping the procedure at a certain maximum pressure to avoid jacking of the fracture is the other limiting edge. In geology with tight rock, there would be no grout take in many of the boreholes therefore grouting time should be limited in boreholes with a certain minimum take as well. Deciding the stop criteria would be more efficient if enough information from grouting procedure and geology of the area is available. The theoretical approach which enables estimation of grout spread and jacking in real time (chapter 3), provide the possibility for defining a robust stop criteria.

Material properties

Both measurement of grout mix properties and choice of material are challenging issues in designing grouting work. According to Håkansson, et al. (1992) rheological properties of cement grout are sensitive to measurement techniques. Resulted values for yield stress and viscosity depend on the instrument and the rheological model.

Different approaches have been proposed for choosing the grout mixture properties. It is suggested by Lombardi & Deere (1993) to use only a single mix for the entire grouting work to provide a single Bingham fluid with known properties and also simplify the grouting procedure and reduce errors. They mentioned advantage of using stable thicker grout during grouting as less sedimentation, less bleeding, greater stability and finally less risk of hydro fracturing due to fast drop of pressure away from the grout hole as a result of grout cohesion. During service time thicker grout will give less shrinkage, higher mechanical strength, less porosity, lower permeability, greater chemical resistance and higher durability of grout curtain of dam. Thus the injected thin grout which will also fill larger fractures may not fulfill all grouting requirements.

The other approach is to start grouting with thinner material and thicken it to establish pressure and fill the fracture (see Houlsby (1990); Bonin, et al. (2012)). Difficulty in applying this method is about deciding how to change the grout to get the desired performance.

Applying changes in mixture properties in regards to the geology i.e. considering penetrability properties of the grout mix is another approach (Eriksson & Stille, 2003). Penetrability meter device developed at division of soil and rock mechanics at royal institute of technology (Eriksson & Stille, 2003) is used to find how the grout material can penetrate in the fractures. In this measurement method, minimum and critical apertures have been defined as ultimate boundaries indicating in which apertures the grout can penetrate and where the filter cake can be developed. To fulfill penetrability requirement, Funehag & Gustafson (2008) suggest usage of a Newtonian fluid such as silica beside the cement base grout to enter and seal small fracture apertures.

Penetrability properties depend on grain size, grain size distribution, super plasticizer, w/c ratio, chemical reaction, geometry of aperture and amount of mixture. Variation of penetrability properties directly affects spread of grout (Eklund & Stille, 2007). These properties can be adjusted based on requirements of penetration and deformation by alteration of material or grouting pressure or both together. In case of fractures with smaller apertures ($b<b_{min}$), there would be no grout take and increasing pressure may lead to hydro-fracturing. Thus changing penetrability properties in the way to decrease the minimum aperture is preferred. On the other hand the fracture with larger aperture ($b>b_{critical}$) may let infinite volume of grout to flow i.e. no filter cake may form. Thus other methods should be used to seal these fractures. Pressure in grout-able fractures (b_{min}<b<b_{max}) should be adjusted to not increase the aperture size over the critical limit which may not only reduce the sealing efficiency of the grouted fracture but it also may lead to remaining of unsealed smaller fractures after grouting.

Borehole spacing

Houlsby (1990) describes closure grouting as a method based on observations and filed data. Primary boreholes are fairly apart and secondary boreholes are casted in between. Lugeon test and grout take are criteria to decide about drilling tertiary and quaternary boreholes. This method is called split spacing and is useful in grouting curtain of dams. In successful grouting operation an overlap of grouting of the fractures is required and in result the first stop criterion can be set in the way that the penetration of the smallest grout-able fractures shall reach at least up to halfway between the boreholes (Gustafson & Stille, 2005). Thus, in this approach borehole spacing can be used as a tool besides grouting pressure and material properties to reach desired spread.

Grouting time

Dalmalm & Stille (2003) have suggested grouting time to be regarded during the design of grout work. In their research work, to optimize the grouting process, the grouting time is correlated to the rock mass joint system situation. In this respect, in blocky rock mass with wide joints, grouting for long time lead to over spread of grout while in crushed rock mass with narrow joints there is low risk of over spread and increasing grouting time improve the sealing efficiency of performed job. Interesting result could be achieved by correlating the injected grout volume and spread with time. for instance in the studied case by Dalmalm & Stille (2003) since most of the volume was taken in short time, decreasing grouting time and drilling more boreholes have been suggested as optimum design.

Later, by developing analytical solution (Gustafson & Stille, 2005) and (Stille, et al., 2012), grouting spread and state of the fracture could be estimated in real time, thus, as depicted in this research work, grouting time has been considered as the stop criterion where enough penetration length is achieved in boundary of desired fracture deformation.

Fracture aperture

One of the important characteristics of the joint that affect grouting design is the aperture size. According to Fransson, et al. (2012) fracture aperture describes behavior of grouting as well as determining penetrability and penetration length. Since transmissivity has correlation with cubic of the aperture size, thus, a small change in fracture aperture size will have significant effect on conductivity of the rock mass. This implies the importance of monitoring fracture dilation

The measurement method of the fracture aperture affects the result. Large difference of hydraulic aperture and void filling aperture at larger transmissivity at Portuguese dam has been depicted by (Carter, et al., 2012). It has been demonstrated by Kobayashi, et al. (2008) that estimated grout aperture based on theoretical method (considering the void filling by Bingham fluid, see section 3.1) is larger than corresponding hydraulic aperture. The same study performed by Tsuji, et al. (2012) with data from City Line project and similar results were obtained (Fig. 4).

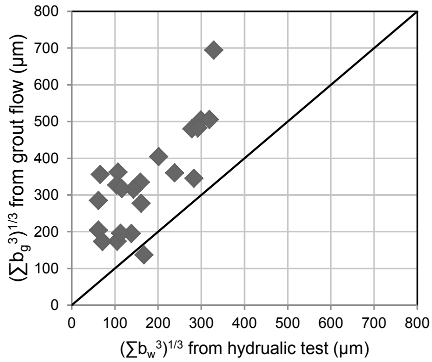

Fig. 4. Comparison of fracture aperture size obtained from the Lugeon test and the Real Time Grouting Control method which consider filling the voids with Bingham material using data from the City Line project. It is illustrated that the latter measurement method estimates a larger fracture aperture size (After Tsuji, et al., 2012).

2.3 Empirical Design Approaches

In previous section, design variables and their effect on grouting process were discussed. Different methodologies have been established to adjust these elements and fulfill successful grouting criteria. Below, some of these methods which are used worldwide in different projects are depicted and merits and disadvantages of them are discussed.

Rules of thumbs

Without any theoretical approach, execution of grouting is mostly based on rules of thumb and decisions that are made according to observations during the process. Houlsby (1990) suggests starting with thinner mix and lower pressure, and check for leakage, rock movement and loose standpipe. If none happened, pressure can be increased and the thicker mix can be used. It means that pressure and material properties are adjusted step by step. In this approach the aim is to getting as much cement in, as fast as possible. Pressure is increased up to maximum pressure with the thickest mix the hole will take. Grouting should be carried out until refusal is reached. As an empirical rule, grouting is completed when the grout flow is less than a certain value at maximum pressure or the grout take is above a certain value.

According to Houlsby (1990), pressure should be maximized as soon as possible before the grout become stiff and by decreasing the pressure, grout take should be decrease as well. Thus the grout take-time (V-t) graph will have decreasing manner. This graph can be used as an assessment tool in which constant or increasing trend of grout take during the time while the pressure is decreasing can be indicator of rock movement or leak. There are limitations in using this method which are discussed as below:

- Geological interpretation

To decide the maximum pressure, depth of the fracture as well as rock mass properties has been considered (see for example Fig.2). The problem is introducing the geological properties to the defined qualities in graph.

- Unknown spread at refusal point

  The other impotent shortcoming of this method is the ambiguity about the spread of grout. Since a certain maximum volume is considered, it is not known if the minimum required spread has been achieved. In case of larger fractures, there would be risk of overspread which is not detectable.

- No information about deformation of fracture

  The same limitation goes to the deformation of fracture which is not detectable. Despite empirical rules for choosing the maximum pressure to avoid jacking, the state of fracture at refusal point is not known.

Aperture Controlled Grouting (ACG)

In attempt to improve practical method, another approach, namely *Aperture Controlled Grouting*, has been introduced. Studying geological issues in more detail, applying changes in grout mixture and using monitoring tools during grouting procedures are focused in this technique. In this method, according to Carter, et al. (2012) after deciding volume based on void filling space and chosen hole spacing, grout rheology is adjusted to match fracture characteristics and achieve pressure build for refusal at the required take. This method can be considered as an update to Australian method suggested by Houlsby (1990) where grout mixes are thickened in sequence using a decision chart (Bonin, et al., 2012). Using modern and near-colloidal grouts rather than thin grout and improvement in deciding mixture thickening sequences are mentioned by Bonin, et al. (2012) as the updates.

The major advantage of ACG method according to Bonin, et al. (2012) is that the grouting criteria are based on fully capturing all of the geological and hydrological information of relevance to grouting the formations of concern by utilizing electronic monitoring system. According to Carter, et al. (2012) grouting to only the required volume necessary to be injected for that stage can optimize the closure and cost standpoints.

Based on the injection decision flowchart of ACG, grouting starts with thinner mix to penetrate in the smaller fractures and is thickened to close up large fractures. The difficulty in applying this method is changing the mixture during grouting which is not practical. Furthermore carrying out DFN (Discrete Fracture Network) studies are time taking and costly. Although using the DFN , regardless of the effort to build the model and rate of uncertainty in the resulted model, is an step forward to define characteristics of discontinuities, still the ambiguities about state of grout spread and fracture at refusal point are remaining.

Grout Intensity Number (GIN)

To control the energy induced in the fracture, production of pressure and volume is restricted. This limit which is called Grout Intensity Number has been proposed by Lombardi & Deere (1993) and trims the rectangle of maximum pressure-maximum volume which was proposed in empirical methods (Fig.5). Therefore, in tight rock mass with small fractures, the maximum pressure is build up with low amount of grout take (path 1). On the other hand, in largest fractures, large amount of grout is injected (Path 4). In other fractures, GIN limit controls the applied pressure by considering injected volume (Path 2 and 3). Lombardi & Deere (1993) propose gradually rising pressure and control it based on injected volume. Single steady grout material has been suggested to be used to confirm sealing of the grouted zone (see section 3.3). Quality of the performed grouting in split spacing system can be evaluated by analyzing these grout paths. According to him, as the grouting reduce from hole series to hole series, the inject volume should be reduced i.e. the path of grouting should shift to left side.

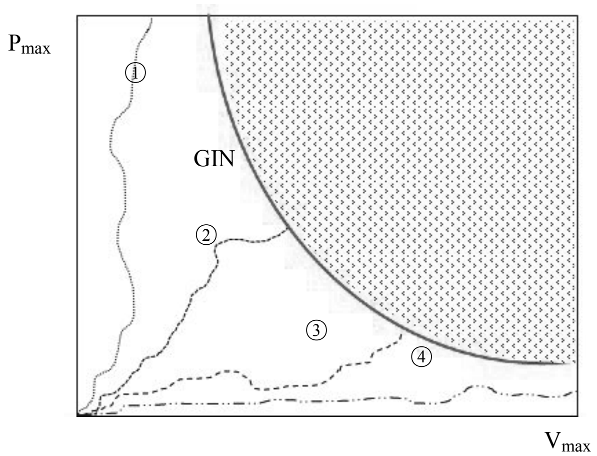

Fig. 5. GIN limit curvature trims the rectangle of P_{max}-V_{max}. Due to size of the fractures, grouting is stopped at different points.

Despite this method has been used in several different projects and still it is very common to be practiced, performance of this method has been questioned (see Ewert (1996); Rombough, et al.(2006); Shuttle , et al. (2007)). Some of the main difficulties in using this method are listed below:

- Nature of GIN Limit

  The nature of GIN limit is unknown. Since the grout is stopped at intersection of grouting path with this limit, the state of fracture is not known. Thus it is not obvious if it is safe to continue grouting up to this boundary and to what extend it can be exceeded. In other word, since GIN curve does not provide any information about the extent of fracture dilation, the safety margin in intersecting this limit is not known.

- Selection of GIN number

  Since the GIN limit not provides any information about the grout spread, selection of this number is not easy task. In practice, a GIN number selected by benchmarking similar projects and after assessing the results from the trial test, this number is adjusted.

2.4 Uncertainties with current approaches

In previous sections, geological and design related issues which are affecting the result of grouting work were discussed. It was depicted that the main uncertainty related to geology refers to estimation of fracture aperture size. Furthermore identifying the required sealing efficiency is important in designing and process of grouting work. Design elements were discussed as well and it was shown that due to lack of enough understanding about grout behavior, the criteria for deciding these elements may not fulfill the requirements of successful grouting. Empirical design approaches suggest deciding the grouting pressure, material properties and stop refusal point based on rules of thumbs. The main improvement in ACG method is modeling of the fractures with more investigation and details and using computerized monitoring. In GIN method, suggested limit is not physically understandable.

However it is not possible to investigate the ground to such extend to overcome geological uncertainties but there are uncertainties connected to the empirical design approaches, which can be eliminated by better understanding of grout behavior through an analytical solution. According to Kobayashi & Stille (2007) since empirically based stop criteria (as well as grouting pressure and grouting time) are determined without a theoretical basis and are not related to grout penetration, the grouting result may be inadequate or uneconomical. Thus it seems essential to have a tool for estimation of state of grout and fracture analytically.

Theoretical approach which is dealing with mechanical behavior of grout in the fracture makes it possible to estimate the penetration length and trend of grout flow as well as fracture dilation in real time based on the initial inputs from laboratory (material properties) and field data (geology of the area and injection pressure). Thus, by considering the criteria as reaching a certain grout spread with no ground movement, the initial estimated design elements can be decided. In next chapter this analytical solution has been explained.

3 Analytical approach for grouting design

In practicing grouting of the fractures, mathematical and numerical models can supplement empirical methods by simulating grout propagation which increase our understanding of grouting mechanics and developing new methods (Hässler, et al., 1992). To estimate the grout flow, an equation derived by Hässler (1991) considering inclination of the channel with several different Bingham fluids. In his model channel network concept has been chosen, since it leads to comparatively simple mathematics without taking away possibilities of great variability both in single joints and between joints. The mathematical model implemented in simulated program. Later the analytical solution introduced by Gustafson & Stille (2005), which gives a robust stop criteria based on required penetration length of mixture. It confirms the requested sealing efficiency while avoiding loss of material and extra process time. Through the efforts by Brantberger, et al. (2000); Gothäll & Stille (2009) and Stille, et al. (2012) monitoring of fracture dilation based on the estimated grout spread could be possible.

In this chapter, this analytical solution which is named as “Real Time Grouting Control” method is explained briefly. In next chapters, in addition to describing the geological situation of the projects and input data, the results have been discussed.

3.1 Penetration estimation

According to Gustafson & Stille (2005) and Kobayashi & Stille (2007) “Real Time Grouting Control method” is a theory to formulize grouting in order to estimate the spread of grout and predict flow trend. the input data are rheological properties of the cement base grout mix which are viscosity and shear stress, geological data (which are mainly the aperture size and depth of the fracture) and also the injection circumstances and adjustment such as pumping pressure, ground water pressure, borehole filling volume and injected grout volume.

By correlating the injected volume with grouting time, Dalmalm & Stille (2003) expressed the spread of grout in term of time and fracture geometry. Characteristic grouting time (t_0) is defined by considering rheological properties of grout mix as well as grouting pressure. Furthermore, since rock mass properties may vary several times in the project and as far as it is not possible to fully model the whole geology, to achieve a design method, Concept of dimensionality has been introduced (see Hässler (1991) and Gustafson & Stille (2005)). In one dimensional case, fractures are like channels and grout directly flows in parallel lines while in two dimensional case, grout flows in a disk around the borehole. The slope of the curve at logarithmic scale graph of injected volume versus grouting time is considered as the indicator for distinguishing dimensionality in every moment.

Relative time which is the ratio of grouting time to characteristic grouting time ($t_D=t/t_0$) has been correlated with relative penetration (I_D) for one and two dimensional flow in a unique graph (Fig.6). the relative penetration for constant pressure has the same time scale for all fractures with different apertures intersected by a borehole. This means that at a certain I_D, the grout has reached the same percentage of its maximum penetration length in all fractures at a certain time (Fransson, et al., 2007). In other word, relative penetration shows grout advancement regardless of the aperture size and demonstrates to what extent of its maximum penetration, grout can spread ($I_D=I/I_{max}$) .

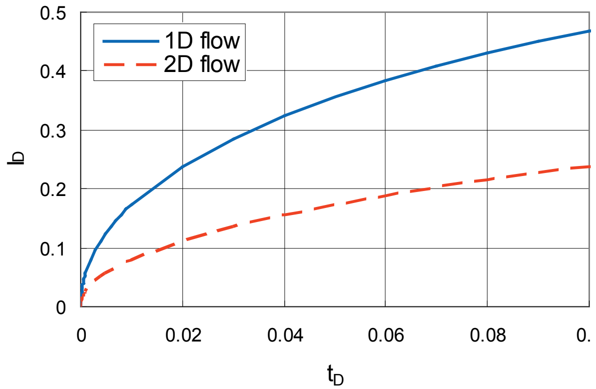

Fig. 6. Relative penetration as a function of relative grouting time on normal x-axis (Gustafson and Stille, 2005).

Since the maximum penetration length is directly correlated to the fracture aperture size, understanding about different measurement methods of the fracture aperture is important. According to Stille, et al. (2012), the fracture aperture can be determined by measuring volume and time and assessed dimensionality. The factors $\sum b_g^3$ for the 2D case and $\sum wb_g^2$ for the 1D case can be determined by curve fitting through the estimated scattered resulted from following equations (see Kobayashi, et al. (2008)):

$$\sum wb_g^2 = \frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \left(\frac{\Delta p}{2\tau_0} \right) / Q \tag{2}$$

$$\sum b_g^3 = 2\pi \cdot I_D \cdot \frac{dI_D}{dt_D} \cdot \frac{1}{t_0} \cdot \left(\frac{\Delta p}{2\tau_0} \right)^2 / Q \tag{3}$$

Considering Pareto distributed transmissivity of fractures (Fransson & Gustafson, 2005) the largest fracture dominates the water inflow and the penetration of grout (Stille, et al., 2009). In other way, flow is governed by aperture cubed ($Q \sim \sum b^3$) which means that few number of fractures with larger aperture stand for most of the flow. Thus the cubic root of the estimated factor in Eq.7 can be a good estimation for fracture aperture in 2D case. It should be noted that in 1D case, the width of the fracture must be known.

In this theory it is assumed that fractures are non-deformable and fluid is non-compressible. Also the pressure change in the groundwater ahead of the front of the grout can be neglected due to distance from the borehole and the viscosity of the grout being larger than that of water (see for example (Fransson, et al., 2007)).

In next section, estimation of fracture deformation due to injection pressure and based on grout spread has been explained. Since in estimation of penetration, the aperture size is assumed to be constant this means that the solution discussed in next section is valid as long as no deformation happen.

3.2 Jacking

According to Gothäll & Stille (2009) one of the limitations of the network model is to not considering mechanical properties of the fracture. According to them the grouting pressure would have some effects on the rock mass and fracture void space. Considering the induced stresses in the rock mass which were discussed as production of pressure and injection volume by Lombardi & Deere (1993), the effort has been made to correlate the GIN value with spread of grout. In this model, "Contact points of the fracture" (which later demonstrated by the concept of dimensionality) and the "geometry of the lifted rock" (which is described by the angle of the lifting cone) were considered to determine the risk of hydraulic uplift (Brantberger, et al., 2000). Based on this research work, the ultimate grouting pressure possible to be applied in a mainly horizontal fracture defined in terms of penetration length as below:

$$P_n \le P_{n,ultimate} = 1 + \frac{1}{I_n} + \frac{1}{3I_n^{\ 2}} \tag{4}$$

Where P_n (critical pressure/effective grout pressure) is normalized penetration and I_n is normalized pressure (Grout spread/depth of the fracture). According to Stille, et al. (2012), considering the elastic properties of the rock, fracture will be deformed at much smaller pressure. This deformation is elastic and is recoverable in case the pressure is revealed and the grout mix can be pumped out. If an acceptable opening of the fracture can be defined, by considering the properties of the rock mass, the maximum allowable pressure is defined as below:

$$P_n \le P_{n,acceptable} = \frac{k}{3I_n} + \frac{1}{3} \tag{5}$$

In which

$$k = \frac{3}{4} . \frac{E}{(1-\vartheta^2)} . \frac{\delta_{acc}}{\rho g h^2} . \frac{P_g}{P_e} \tag{6}$$

P_e is the grouting pressure reduced with the initial normal stress, E is the young's modulus of the rock mass and v is the Poisson ration. h is the depth of the fracture to the free surface. $P_{n,\ acceptable}$ determines the maximum allowable pressure to achieve the required penetration in boundary of acceptable deformation. The elastic and ultimate jacking limits are depicted in figure 7.

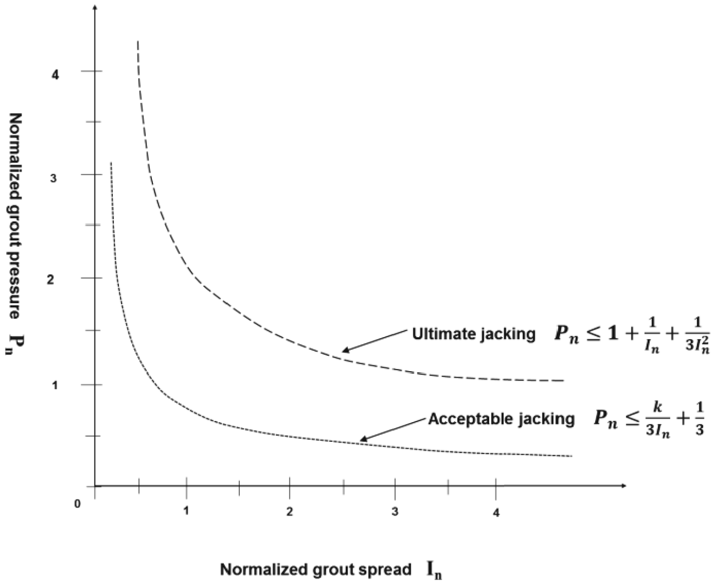

Fig. 7. Maximum normalized pressure as a function of normalized grout spread for both the ultimate limit state (Eq. 1) and the acceptable limit state (Eq. 2). The curves are calculated for $P_w = 0$ (After Stille et.al 2012).

3.3 Assessment

In this research work, results are evaluated theoretically by comparing the estimated flow trend from the theory with the recorded data which describes the process physically. LOGAC flow meter is used to register injection pressure and grout flow during the procedure. Well convergence of these flows confirms the efficiency of the solution. According to Lombardi & Deere (1993), contacts of the grout with fracture walls increase during spreading, thus the decrease of flow in applying constant pressure is expected. At holes linked to more porous area or in case of leakage, in applying constant pressure the grout flows constantly or even with increasing trend. It happens also where fracture aperture increases i.e. jacking of the fracture (Fig.8). Therefore to evaluate the performance of the theoretical approach in estimation of fracture dilation, the deviation point of recorded and predicted flow is compared with the corresponding time at which the path of normalized pressure-normalized spread reach the elastic jacking limit.

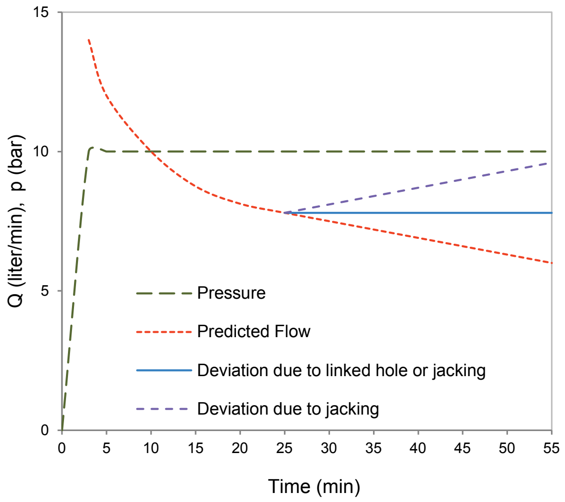

Fig. 8. Jacking to be detected by grout flow in principle. If jacking occurs, the increase in grout flow should be observed. Constant flow in constant pressure indicates linked hole or jacking (Tsuji, et al., 2012). Conversion factors 1 bar = 0.1 MPa and 1 liter/min = 1.67x10-4 m3/s.

4 Studied Projects

In this research work, the theoretical approach has been verified with data from different projects. Data from Gotvand Dam project has been used to evaluate performance of the analytical solution in estimation of penetration length and fracture dilation (Paper I and II). Furthermore, grouting works in THX Project has been examined in respect to Jacking phenomena (Paper II). In the papers III and IV, Data from City Line project has been used. Below, Geological characteristics of each project as well as the used material and considered stop criteria have been given.

4.1 Gotvand Dam Project

Geology

At Gotvand project, a rock fill clay core dam has been constructed on the sedimentary rock. The geology of the region is consists of two different formations: "Bakhtiari" formation which is situated in upper level and the "Agha Jari" formation which is laid beneath (rafi 2010). These formations consist of lime stone, sand stones and in upper levels, conglomerates. At the top level, there is a layer of dislocated rock mass which is the most permeable section. Density of rock is 24 kN/m^3 and modulus of elasticity is 4 GPa. The goal is to reduce conductivity to less than $3*10^{-7}$ which is corresponding to water loss in control holes of 3 Lugeon.

Material

Grout with water cement ratio of 2 has been used. This high ratio selected since the geology is sedimentary rock and grout mixture can penetrate smaller fissures. d_{95} of the cement is 32µm, yield stress is 0.35 Pa and viscosity is 0.0043 Pas. Regarding to the results in Paper I, this mixture has been penetrated far away. Sealing efficiency of this material is a point of question as well (see Paper I)

Stop criteria

The gradient of pressure and flow is considered as stop criterion in grouting works of Gotvand dam project and grouting is stopped when the flow is less than a minimum value in a defined pressure. The order of setting pressure according to consultant document is as below:

- During the first 10 minutes of grouting the exerted pressure gradient is 0.5bar/min and the manner continue if the flow is more than 8lit/5min.
- If the flow is lower than 8lit/5min the exerted, increase pressure gradient to 1 bar/min.
- If the flow is near zero, increase the pressure Gradient to 2-3bar/min.

4.2 THX Project

Geology

At THX project, which is about building a concrete dam, 10 rock core obtained and analyzed with aim of representing the geological logs thus describing the lithological characteristics of the rock mass and evaluating the rock cores in terms of strength, weathering and degree of fracturing (consultant documents). Density of rock mass is 26 kN/m^3 and modulus of elasticity is 4 GPa. In this research work cases from the right bank of the dam situated between chainage 60 to 190 have been studied. According to consultant documents, sandstone and conglomeratic sandstone prevalence in the dam foundation. Mudstone is mainly found in the foundation in the upper parts of the right bank, abutments and the central part (table 3).

Table 3. geological properties in THX Project

From Chainage	*To Chainage*	*Main Lithology*
60	190	Mudstone/Sandstone layers
190	220	Sandstone
220	300	Sandstone/Conglomerate layers
300	400	Mudstone/Sandstone layers
400	545	Sandstone with mudstone layers

Material

In THX dam project, thicker gout mix in compare with Gotvand project has been used. Grout has been mixed with the water cement ratio of 0.75 which gives yield stress of 0.7 Pa and viscosity of 0.02 Pas.

Stop criteria

GIN method has been used to distinguish refusal point. In studied cases, the GIN value of 1000 has been used.

4.3 City Line Project

Geology

In the City Line project in Stockholm (Citybanan), currently grouting is conducted in the service tunnels at Stockholm city station. The dominant rock types are mainly gray granite or reddish gray gneiss with minor feature of pegmatite or amphibolite. Thus mechanical property of the rock is expected to be modulus of elasticity of 40000 MPa and density of 28 KN/m3. The hydrogeological pre-investigation shows the rock mass conductivity varying between 10^{-7} and10^{-8} m/s and the ground water level is considered to be equal to the ground surface level as elevation is close to the sea level in the Stockholm area. The ground water head of the area ranges between 32 and 33m. Furthermore, the fracture mapping shows that vertical fractures are dominant in the studied tunnel. Grouting is performed in fans of boreholes with inclination of between 10 to 20 degrees and the length is often between 20 and 25 m in general.

Material

The Injecteria 30 is used as the cement and mixed with the water in the ratio of 0.8. The rheological properties of the final product are yield stress of 6 Pa and viscosity of 0.02 Pa.s.

Stop criteria

According to the documents from consultant the following practical criteria are applied in grouting of this borehole.

Grouting is normally conducted with planned pressure for 20 minutes. Grouting is completed when any of the following stop criteria is achieved within that time:

(a) When the grout flow is less than 1 liter/min and sustained for 5 minutes.

(b) When the grouting volume is above 500 liters (exclude hole filling volume), change the grout mix to W/C= 0.5 and after the thick grout volume is above 150-200 liters.

5 Discussion of the results

The grouting works is categorized as difficult since in all studied cases the goal is high reduction in conductivity and therefore there is a need to use more precise procedure in design and execution. It has been depicted that using empirical methods in studied projects resulted in unsuccessful grouting. There were over spread of grout in studied cases in Gotvand Project and jacking has happened in fractures of THX Project. However, in City Line project, the theory confirms usage of high pressure which decreases the grouting time.

In studied cases with data from the sedimentary rock of Gotvand Dam (Paper I), there is a well convergence of the recorded and predicted flow trends which indicate well performance of the analytical method. In this study, size of the largest existing fracture based on flow of grout has been estimated. This fracture is dominant for most of grout flow and penetration length of grout has been estimated in this fracture. Results indicate overflow of grout in the studied cases since the used grout is very thin and grout has been injected for a long time. Shorter penetration length is achieved with a thicker mix in the same time. It has been shown that by using penetration length as the stop criteria, grouting time and material properties can be optimized. Considering long duration of grouting at each borehole, using this theory can shorten the grouting time at curtain of this dam.

In the other research work (Paper II), jacking of the fracture has been studied. In this purpose, elastic and ultimate jacking limits have been established based on the estimated grout spread. Studied cases with data from sedimentary rock of Gotvand Dam and THX Project confirm well performance of this approach. in all the studied cases, Deviation of recorded and predicted flow happens about the same time at which deformation of the fracture starts (see section 3.3). Uncertainties in measurement of the pressure as well as the depth of the fracture (the distance of the surface to the largest fracture which will jack) are among the reasons for the small difference. As a further study, a methodology for deciding grouting pressure based on the required penetration length has been proposed. From the results it could be concluded that recorded pressures are higher than the one proposed by the theoretical approach. The empirical approach was found conservative in some cases (Fig 9).

In the studied case at City Line project (Paper III and IV), considering a single horizontal fracture in depth of 30 meters below the surface, the high applied pressure resulted in small deformation in the assumed fracture. Geology of the studied area consists of mostly vertical fractures, and as discussed before, much larger pressure is required to jack these vertical joints. Thus the theory confirms usage of high pressure which shortens the grouting time while monitoring the grouting work with theoretical approach is suggested.

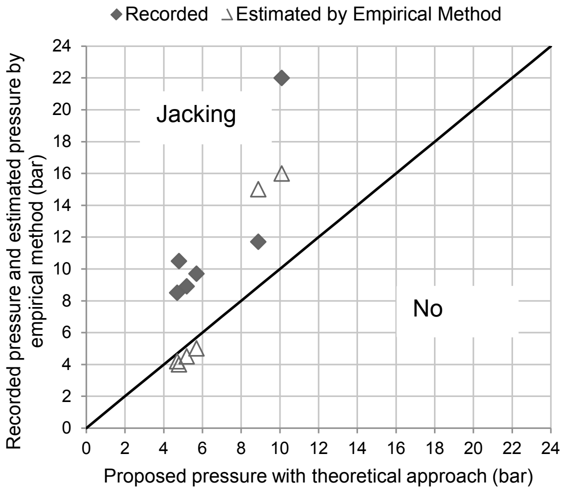

Fig. 9. Comparing the pressure proposed by the empirical method and also the recorded pressure during grouting with the ones obtained from the proposed methodology at the studied boreholes. Applied pressures are higher than the one proposed by the theoretical approach and lead to deformation of fractures. The empirical approach is conservative in some cases. Conversion factors 1 *bar* = 0.1 MPa.

6 Conclusion

Theoretical approach in grouting design which is called "Real Time Grouting Control Method" has been studied in this research work. Performance of this approach regarding estimation of grout penetration length has been validated through the cases from 3 different projects. Furthermore, the possibility of using the estimated penetration length for identifying jacking of the fracture has been verified. It has been shown that performance of grouting work can be controlled and design variables can be optimized based on the predicted values for grout spread and fracture deformation, which are provided by this method.

Uncertainties connected to geological properties of the rock mass can be mentioned as the main obstacle in implementation of this theory. A single horizontal fracture has been assumed to be dominant for the grout flow. Furthermore, variation of the fracture aperture size as well as material properties has been neglected. Despite assumptions and simplifications, the results are promising.

Currently this theory has been applied in different geologies and performance of it has been verified. To enable using this approach as a tool in future grouting works, there is a need for more case studies in projects with different geological properties which make it possible to improve and customize this analytical solution to be installed on grouting machineries. Studying pros and cons of elastic deformation of fracture will optimize the stop criterion. Furthermore, examining the empirical approaches closely in compare with theoretical approach provides better understanding about obstacles of current practices and will be helpful in developing the theory. Applying this theory in to highlight and decrease ambiguities in grouting works is another aspect of future work.

Reference List

a, n.d. *pin,* s.l.: s.n.

Bonin, G. R., Rombough , V. T., Carter, T. G. & Jefferies, M. G., 2012. *Towards Better Injection Control and Verification of Rock Grouting.* New Orleans, s.n.

Brantberger, M., Stille, H. & Eriksson, M., 2000. Controlling Grout Spreading in Tunnel Grouting Analyses and Development of the GIN-method. *Tunneling and Underground Space Technology,* 15(4), pp. 343-352.

Carter, T. G., Dershowitz, W., Shuttle, D. & Jefferies, M., 2012. *Improved Methods of Design for Grouting Fractured Rock.* New Orleans, Proceeding of 4th International Conference on Grouting and Deep Mixing, pp. 1472-1483.

Dalmalm, T., 2004. *Choice of Grouting Method for Jointed Hard Rock based on Sealing Time Predictions,* Stockholm: Royal Institute of Technology.

Dalmalm, T. & Stille, H., 2003. *Some aspects on grouting time modelling.* New Orleans, ASCE.

Dershowitz, W. S., Doe, T. W., Uchida, M. & Hermanson, J., 2003. *Correlation between fracture size, transmissivity, and aperture.* Cambridge, MA, Soil Rock America: Proceeding of the 39th US Rock Mechanics Symposium, pp. 887-891.

Eklund, D. & Stille, H., 2007. Penetrability due to filtration of cement base grouts. *Tunneling and underground space technology.*

Eriksson, M. & Stille, H., 2003. *A method for measuring and evaluating the penetrability of grouts.* NewOrleans, Proceeding pf 3th International Conference on Grouting and Deep Mixing.

Ewert, F., 1996. The GIN Principle-a helpful method for rock grouting? part1 and 2. *International Water Power and Dam Construction,* February and April, 48(2), pp. part 1: pp17-19, Part 2: pp 36-40.

Fransson, Å., 2001. Characterisation of a fractured rock mass for a grouting field test. *Tunneling and underground space technology ,* Volume 16, pp. 331-9.

Fransson, Å., 2008. *Grouitng design based on characterization of the fractured rock,* s.l.: Swedish Nuclear Fuel and Waste Managment Co. (SKB).

Fransson, Å. & Gustafson, G., 2005. The use of the Pareto distribution for fracture transmissivity assessment. *Hydrogeology Journal,* Issue 14, pp. 15-20.

Fransson, Å., Tsang, C.-F., Rutqvist, J. & Gustafson, G., 2007. A new parameter to assess hydromechanical effects in single-hole hydrualic testing and grouting. *International journal of Rock Mechanics and Mining Sciences,* Volume 44, pp. 1011-1021.

Fransson, Å. et al., 2012. *A swedish grouting design concept: Hydrualic testing and selection of grout.* New Orleans, proceeding of 4th International Conference on Grouting and Deep Mixing, pp. 1691-1700.

Funehag, J. & Gustafson, G., 2008. *Injekteringsförsök med Cembinder® U22 i Hallandsås,* Göteborg, Sweden.: Department of Geoengineering. Chalmers University of Technology.

Gothäll, R. & Stille, H., 2009. Fracture dilation during grouting. *Tunnelling and underground space technology,* Volume 24, pp. 126-135.

Gothäll, R. & Stille, H., 2010. Fracture-Fracture intraction during grouting. *Tunnelling and Underground Space Technology,* Volume 25, pp. 199-204.

Gustafson, G. & Stille, H., 2005. Stop criteria for cement grouting. *Felsbau,* 23(3).

Gustafsson, G. & Stille, H., 1996. Prediction of Groutability from Grout Properties and Hydrogeological Data. *Tunneling and underground space technology,* Volume 11, pp. 325-332.

Hast, N., 1958. The measurement of rock pressure in mines. *Sveriges Geol. Undersokn,* 52(3).

Hoek, E. & Brown, E., 1980. *Underground Excavation in rock.* London: Institute of Mining and Metallurgy.

Houlsby, A., 1990. *CONSTRUCTION AND DESIGN OF CEMENT GROUTING, A guide to grouting in rock foundations.* New York: John Wiley and Sons.

Håkansson, U., Hässler, L. & Stille, H., 1992. Rheological properties of microfine cement grout. *Tunneling and Underground Space Technology,* Volume 7, pp. 453-458.

Hässler, L., 1991. *Grouting of Rock - Simulation and Classification- PhD thesis,* Stockholm: Royal Institute of technology.

Hässler, L., Håkansson , U. & Stille, H., 1992. Computer-simulated flow of gorut in jointed rock. *tunneling and underground space technology,* 7(4), pp. 441-446.

Ivanetich , K., Bryant, K. & Gularte, F., 2012. *The Use the Optical Televiewer in Geotechnical Construction.* New Orleans, Proceeding pf 4th International Conference on Grouting and Deep Mixing.

Kobayashi, S. & Stille, H., 2007. *Desgin for rock grouting based on analysis of grout penetration,* Stockholm, Sweden: Swedish Nuclear Fuel and Waste Managment Co (SKB).

Kobayashi, S., Stille, H., Gustafson, G. & Stille, B., 2008. *Real Time Grouting Control Method, Development and application using Äspö HRL data,* Stockholm, Sweden: Swedish Nuclear Fuel and Waste Management Co (SKB).

Lombardi, G., 1997. GIN Principle Revised. *International Water Power and Dam Construction.*

Lombardi, G. & Deere, D., 1993. Grouting Design and Control Using the GIN Principle. *International Water Power and Dam Construction,* 45(6), pp. 15-22.

Palmström, A. & Stille, H., 2010. *Rock Engineerng.* s.l.:Thomas Telford.

Priest, D. S., 1993. *Discontinuity analysis for rock engineering.* London: Capman & Hall.

Rafi, J., 2010. *Applying "Real Time Grouting Control Method" in Gotvand Dam,* Stockholm, Sweden: Royal Institute of Technology.

Rafi, J. & Stille, H., 2012. *Applying "Real Time Grouting Control Method" in Sedimentary Rock with Gtvand Dam Data.* new orleans, s.n.

Rafi, J. & Stille, H., 2013. Controlling Jacking of Rock considering Spread of Grout and Grouting Pressure. *Tunneling and underground space technology.*

Rafi, J., Stille, H. & Bagheri, M., 2012. *Applying Real Time Grouting Control in Sedimantry Rock.* New Orleans, Proceeding pf 4th International Conference on Grouting and Deep Mixing, pp. 1450-1459.

Rombough, V., Bonin, G. & Shuttle, D., 2006. *Penetrability Control of GIN Mixes During Fractured Rock Grouting.* 1-4 October, Vancouver, Canada, Proceeding of Sea to Sky Geotechnique: 59th Canadian Geotechnical Society Conference, pp. 528-535.

Shuttle , D., Rombough, V. & Bonin, G., 2007. *GIN Distilled.* [Online]
Available at: http://www.groutline.com/articles.htm

Snow, D., 1965. *A parallel plate model of fractured permeable media,* s.l.: Unversity of california, Berkeley.

Stille, B., 2010. *Experiences With Real Time Grouting Control Method.* Stockholm, Bergmekanikdag, pp. 119 -132.

Stille, B., Stille, H., Gustafson, G. & Kobayashi, S., 2009. Experience with the real time grouting control method.. *Geomechanics and Tunneling,* 2(5), p. 447–459.

Stille, H., 2012. *Rock Grouting in Tunnel Construction-models and design.* Stockholm, EURock.

Stille, H., Gustafson, G. & Hassler, L., 2012. Application of New Theories and Technology for Grouting of Dams and Foundations on Rock. *Geotechnical and Geological Engineering.*

Terzaghi, K. & Richart, F., 1952. Stresses in rock about cavities. *Geotechnique,* Volume 3, pp. 57-90.

Tsuji, M. et al., 2012. *Optimization of Grouting Procedure with RTGC and data from trialgrouting at Citiybanan in Stockholm,* Stokholm: SKB.

Wei Jiang, X. et al., 2009. Estimation of rock mass deformation modulus using variations in transmissivity and RQD with depth. *International Journal of Rock Mechanics and Mining Science,* Volume 49, pp. 1370-1377.

Wilson, D., Dreese, T. L., Heenan, D. & Cockburn, J., 2002. *Apparatus and method for computerized data collection, monitoring, analysis, and control of grouting operations.,* USA: s.n.

Published Articles

Paper I*: Rafi, J, Stille, H, Bagheri, M, 2012. Applying “Real Time Grouting Control Method” in Sedimentary Rock, in *4th International Conference on Grouting and Deep Mixing*. 16-18 February, New Orleans-USA.

Paper II*: Rafi J, Stille H, 2013. Controlling jacking of rock considering spread of grout and grouting pressure, Accepted to be published in *Journal of Tunneling and Underground Space Technology*.

Paper III*: Rafi J, Tsuji M, Stille H, 2013. Theoretical Approaches in Grouting Fractures of the Rock Mass: Theories and Applications. Accepted in *the 47th US Rock Mechanics / Geomechanics Symposium*. 23-26 June, San Francisco, CA, USA.

Paper IV: Rafi J, Tsuji M, Stille H, 2013. Theoretical approaches in grouting design: estimation of penetration length and fracture deformation in real time, Presented in *Bergmekanikdagen*, 11 March, Stockholm-Sweden.

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BERÄKNING AV BRUKS SPRIDNING OCH SPRICKDEFORMATIONER; REALTID FÖR STYRNING AV INJEKTERINGFÖRLOPPET

Theoretical approaches in grouting design: estimation of penetration length and fracture deformation in real time

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SAMANFATTNING
Analytiska lösningar för uppskattning av injekteringsbrukets inträngning i realtid har utvecklats (Gustafson och Stille 2005). Denna metod ger ett robust stoppkriterium baserat på brukets inträngning i sprickan. Metoden har validerats genom tillämpning i ett flertal olika projekt med olika typer av berggrund, såsom hård kristallin berggrund och sedimentär berggrund. Resultaten bekräftar att erforderlig tätningseffekt uppnås med bruk- och tidssåtgång som sannolikt ligger när den optimala.

Genom att ersätta den injekterade volymen injekteringsbruk (vilket för närvarande görs i dagens empiriska metoder) med den uppskattade inträngningen av bruket gör det även möjligt att uppskatta sprickans mekaniska spänningstillstånd i realtid. I tidigare studier har metodens har metodens tillämpning studerats. I denna studie har metodens effektivitet i en av Citybanans tunnlar utvärderats. För detta syfte har gränser för brott- och elastiska deformationstillstånd i den injekterade sprickan beräknats tillsammans med sprickans tillstånd i realtid. Trots att höga injekteringstryck används visar analysen att sprickans beräknade deformationer är lägre än de som krävs för ett uppspräckningsbrott. De deformationer som inträffar i sprickan kan i vissa fall vara fördelaktiga då de kan öka inträngningen och minska injekteringstiden.

SUMMARY
Analytical solution for estimation of grout penetration in real time has been introduced (Gustafson and Stille 2005). This method gives a robust stop criteria based on required penetration through the fracture of grout mixture. This method has been validated through application in several case studies with data from tunnels in hard rock, pre-Cambrian rock and sedimentary rock. It confirms the requested sealing efficiency while avoiding loss of material and extra process time.

Substitution of injected grout volume (currently used in empirical methods) with the estimated grout spread provide an interesting opportunity to estimate state of fracture in real time as well. In previous studies applicability of this method in estimation of penetration length has been verified. In this study, efficiency of this approach at one of the tunnels in Citybanan project is examined. For this purpose ultimate and elastic jacking limits based on the spread of the grout has been established and state of the fracture in real time has been determined. Despite high applied pressure, the

estimated state is much lower than ultimate jacking limit although the fracture has been opened up. This deformation can be beneficial since it would increase penetrability and shorten grouting time.

INTRODUCTION

Selection of grouting pressure in order to have adequate grout spread while avoiding undesirable deformations is point of interest. Jacking of the fracture due to injection pressure will be beneficial since penetrability increase and the required spread will be achieved in shorter time. On the other hand larger induced pressure will cause larger deformations which affects sealing efficiency of grouting works. Continuing the process with this pressure may lead to uplift of the rock mass and cause damages to the on-ground structures.

At empirical method, grout spread and jacking are controlled by limiting injection pressure in tight rock and grout volume in larger fractures (Houlsby, 1990). To avoid jacking, Lombardi & Deere (1993) introduced limitation on production of pressure and volume to control the induced energy in the fracture. There are difficulties and ambiguities in practicing empirical methods which among them, lack of information about the amount of grout spread as well as fracture deformation due to injection of grout material are considerable.

The objective of this article is to verify performance of the theoretical approach which is an analytical solution for estimation of grout penetration length and state of fracture in real time. Advantages of practicing this method in compare with empirical methods have been discussed and applicability of the method at grouting works of Citybanan project has been examined.

GROUTING SPREAD

Different strategies and stop criteria have been proposed in order to control injected volume of grout while achieving required penetration length. Houlsby (1990) suggests stopping injection when a fairly substantial amount of cement has been injected. But in this case it is not known if the grout has been spread enough around the borehole. Considering a certain low flow rate or certain maximum pressure would not fulfill the spread requirements either. Analytical solution has been proposed by Gustafson and Stille (2005) which enables estimation of penetration length in real time. This theory which is called “Real Time Grouting Control Method” first developed in channels (Hässler, 1991) by considering properties of Bingham Fluid. Grout flow in the fracture has been simplified to flow in channels where grout spread in parallel lines (on dimensional flow) or flow in a disk around the borehole (two dimensional flow).

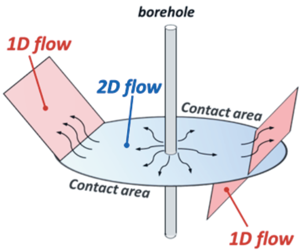

Figure 1 Dimensionality of the grout flow

Fracture aperture is estimated based on the flow of Bingham fluid in the fracture (void filling aperture). Since transmissivity of the rock mass is correlated to the cubic size of aperture, a fracture with the largest aperture is dominant for most of the flow. Thus 70-80% of sum of the apertures is considered as the largest existing aperture.

To solve the numerical approach in analytical solution, relative time and penetration are defined by Gustafson & Stille (2005) according to dimensionality of fractures. Thus at every moment, based on characteristics of the material and applied pressure, relative penetration for each dimensionality is available (figure 2). Relative penetration is independent of fracture aperture and is the same for every fracture. It implied the ratio of grout spread to maximum possible penetration length. Since the relative penetration and the fracture aperture are available, flow of grout mix at every interval is predicted.

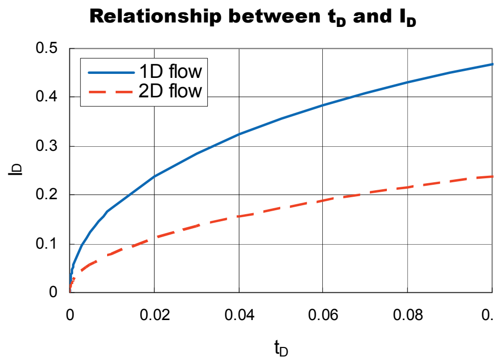

Figure 2 The relative penetration as a function of the relative grouting time in normal x-axis (Kobayashi, et al., 2008).

GROUTING PRESSURE
As a rule of thumb, the overburden pressure is principally considered in selection of pumping pressure to limit potential for uplift or displacement. Furthermore the rock strength, apparently independent of depth, should be the limiting factor for pressure (Weaver, 1991). Empirical methods propose usage of moderate pressure in stressed rock, called as penetration grouting by Houlsby (1990), which is helpful to avoid disruption while in deeper levels of ground (displacement grouting), higher pressure which open up the fractures to facilitate penetration of grout is suitable. For this purpose graphs for selection of the proper pressure based on quality of rock mass and the depth where fracture situated in have been established (for example figure 3.). The difficulty of using this method is categorizing geology of the grouting zone according to suggested qualities in these graphs.

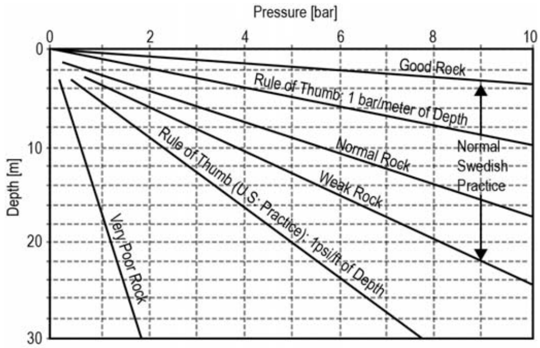

Figure 3 Grouting pressure according to practice in Sweden and U.S. (Weaver 1991). Both rock quality and depth of grouting are important factors in determining the grouting pressure.

As it is mentioned by Houlsby (1990), the energy regarded to combination of volume and pressure should be controlled, Lombardi & Deere (1993) introduced Grout intensity number (GIN) as a curvature trimming rectangle of maximum pressure-maximum volume (fig 4). It implies that in tight rock mass where there is low grout take, high pressure up to maximum limit is applicable while in larger fracture where grout spread in far distances, lower pressure must be applied to avoid undesired deformations or even uplift of the rock. In practicing this method, state of the fracture when GIN curve arrives is not known. Furthermore the amount of spread at this point is indefinite. Thus there are ambiguities in selection of GIN number and applicability of this method has been questioned (Ewert, 1996; Rombough, et al., 2006; Shuttle , et al., 2007).

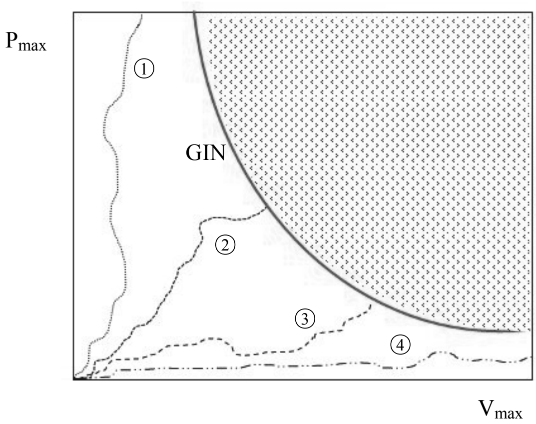

Figure 4 GIN limit curvature trims the rectangle of P_{max}-V_{max}. Due to size of the fractures, grouting is stopped at different points. (from ① to ④, larger fractures are grouted. Dotted zone is the danger zone)

The reason to use volume instead of theoretically correct penetration in empirical methods was practical. (Stille, et al., 2012). As a development to GIN method, Brantberger, et al. (2000) correlated GIN value with spread of grout. Since the grout pressure acting as a cone on the fracture, it should be at least 3 times the overburden to lift the rock mass above (figure 5).

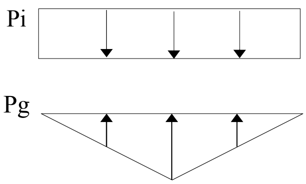

Figure 6 A cone of pressure acting on walls of the fracture

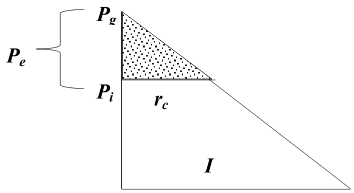

Figure 5 Excess pressure (P_e) is due to the difference of grouting pressure and critical pressure (Dotted zone).

The normalized pressure is defined as $P_n = P_g/3P_i$, thus the ultimate jacking occurs at P_n> 1 where deformations are permanent. This is valid in the larger grout spread. When the penetration length is small in compare with the depth of the fracture below ground surface, due to the geometry of the lifted cone, much larger pressure is applicable.

It has been shown by Gothäll & Stille (2009) that when the grouting pressure (P_g) exceed the critical pressure (P_i), The exceeding pressure P_e loading the rock mass in radius of r_c (figure 6) and the grouted fracture will open up. In other word, as soon as the $P_n<1/3$, the fracture starts to dilate. The deformation at this point is elastic and is recoverable if the injection pressure is released and the grout mixture can be pumped out.

In case of allowing larger deformations, higher pressure is applicable. Both serviceability and ultimate limit states have been depicted in figure 7. Over the acceptable jacking limit, dilation of the fracture is larger than acceptable deformation but it is recoverable. It continues up to ultimate jacking limit where beyond that, deformations are permanent i.e. rock will be uplifted.

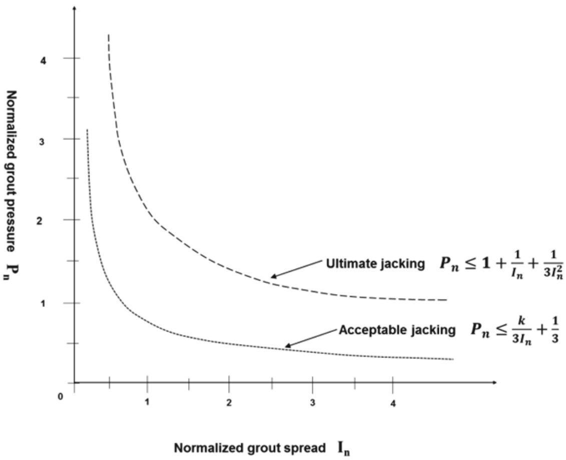

Figure 7 Relation between relative pressure and relative penetration related to ultimate jacking and acceptable jacking (Stille, et al. 2012). (K is the stiffness of the loaded area.)

METHODOLOGY

Substituting injected grout volume with penetration length provides an interesting chance to develop Real Time Grout Control method in order to estimate state of the fracture in real time. This method is practiced through two processes 1) estimation of the grout spread and 2) establishing serviceability and ultimate limits based on the estimated grout spread to predict state of the fracture and decide pumping pressure.

At first process, pumping pressure and grout flow which are recorded by Logac system in addition to properties of the grout material are inputs for estimation of the grout spread. Relative penetration is obtained through figure 2 and by determining fracture aperture size penetration length of grout in real time is estimated. Stop criteria considered as "reaching a certain maximum spread in largest fracture and certain minimum spread in smallest fracture" (Kobayashi, et al., 2008). Therefore, by considering the required penetration length, grouting time at which the procedure should be stopped is achieved. Convergence of predicted grout flow with the recorded one is an indicator to verify efficiency and accuracy of this process.

To establish elastic and ultimate jacking limits, the estimated penetration length in first process as well as geological properties of rock mass, depth of the fracture aperture under the ground surface and decided acceptable deformations are used. Examining the state of fracture in this span as well as adjusting the initial design inputs (Pumping Pressure, Material Properties and Refusal Point) is of interest. The procedure at "Real Time Grouting Control Method" to estimate spread of the grout and state of the fracture in real time has been depicted in figure 8

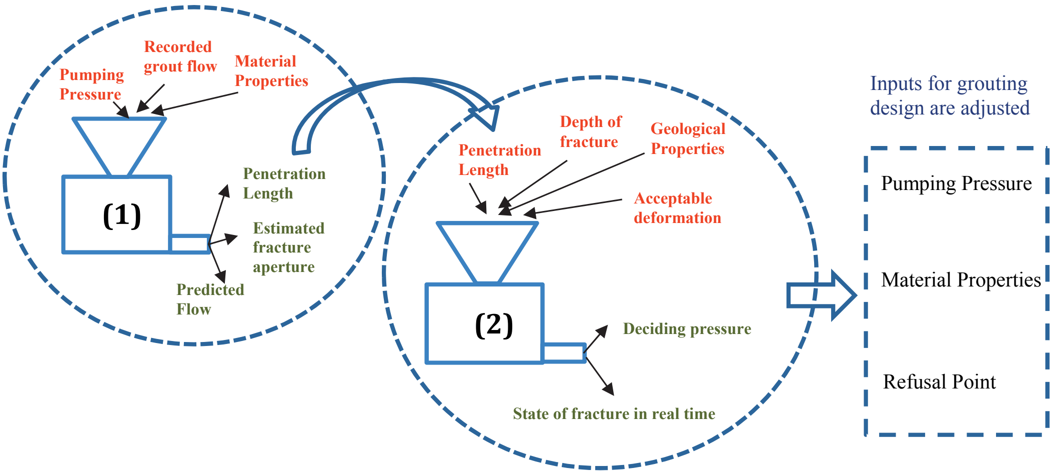

Figure 8 Procedure of Real Time Grouting Control method is practiced in two processes. Penetration length which is outcome of the first process is used for the estimation of state of the fracture. By this procedure, initial design inputs are adjusted.

To verify efficiency of this theory at estimation of the grout spread and defining stop criteria based on the theory, case studies have been performed with data from different projects

(Fransson, et al., 2012; Tsuji, et al., 2012; Stille, 2010; Rafi & Stille, 2012). In this study grouting work at one of the tunnels of Citybanan project has been examined to study the complete procedure.

CITYBANAN PROJECT

Project configuration

In the City Line project in Stockholm (Citybanan), currently grouting is conducted in the service tunnels at Stockholm city station. Grouting is performed in fans of boreholes with inclination of between 10 to 20 degrees and the length is often between 20 and 25 m in general. The tunnels are excavated in hard rock of Stockholm with Modulus of elasticity of 40000 MPa.

Material properties and stop criteria

The Injecteria 30 is used as the cement and mixed with the water in the ratio of 0.8. The rheological properties of the final product are yield stress of 6 Pa and viscosity of 0.02 Pa.s. According to the documents from consultant the following practical criteria are applied in grouting of this borehole.
1. Grouting is normally conducted with planned pressure for 20 minutes. Grouting is completed when any of the following stop criteria is achieved within that time:

a) when the grout flow is less than 1 lit/min and sustain for 5 minutes.

b) When the grouting volume is above 500 liters (exclude hole filling volume), change the grout mix to W/C= 0.5 and after the thick grout volume is above 150-200 liters.

Analyzing grouting work

To verify efficiency of Real time grouting control method, grouting at the borehole number 4 in fan 1522 is examined. It is assumed that largest fracture which is dominant for most of the grout is horizontal and situated in the middle of the borehole. Based on the drawings form consultant, this fracture is at the depth of 30 meters under the ground surface. Refusal point has been arrived at the maximum injected volume.

Injection pressure and flow of grout has been registered during the grouting work (figure 9). High constant pressure has been applied for around 10 minutes. The amount of pressure has been decreased and continued constantly up to refusal point. As expected, grout flow in decreasing trend up to 3 minutes. Largest fracture aperture has been estimated based on the grout flow in this zone as 0.1 mm. At this point sudden increase of flow can indicates opening of new fractures (hydro fracturing) in this tight zone. According to Houlsby (1990), the signs indicate that rock has probably moved are sudden increase in grout take for no apparent reason or sudden loss of the pressure in the hole. Therefore, jacking may start at the moment when recorded and predicted flow data are deviating and continue as long as grout flow in a trend other than predicted one. The sharp increase at around ten minutes with no big drop afterwards can be due to existence of connected fractures where the mixture can flow in large amount. Flow of the grout has been estimated with the initial estimated fracture aperture and in first 3 minutes, it converges with the recorded flow. Extension of the predicted flow implies that the grout take has been much larger than the expected amount. The grout flow is steady or even increase while the grout pressure is approximately constant. It means that the fracture aperture may opening up i.e. jacking has happened.

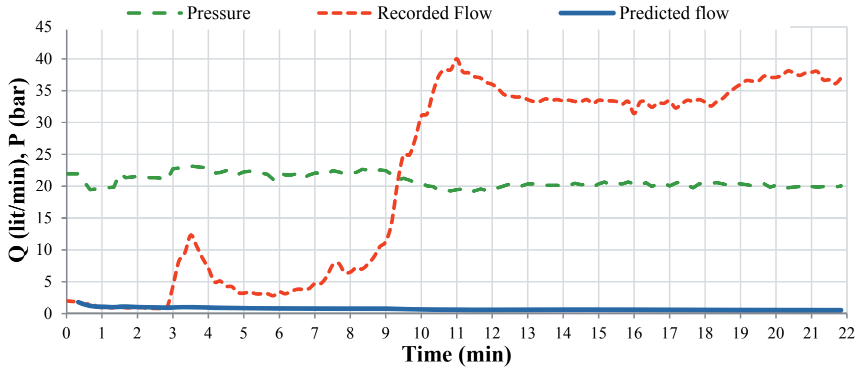

Figure 9. Recorded grout pressure and flow and predicted flow obtained from analytical solution. Hole filling period has been excluded.

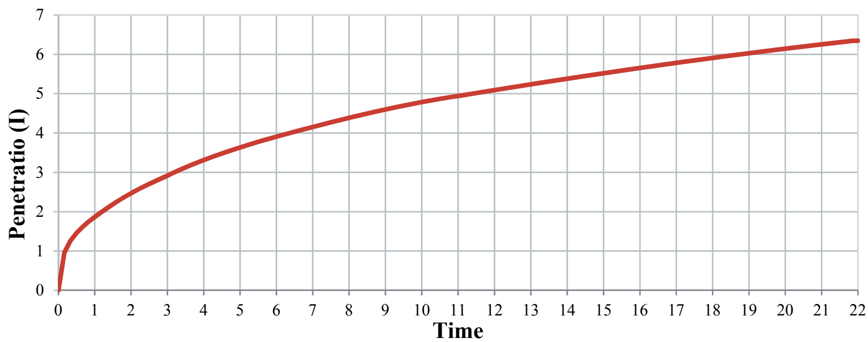

Figure 10 Penetration length of grout in assumed single horizontal fracture.

Critical pressure (Pi) is the overburden of the rock mass and would be 8.4 bar which is considerably less than the applied pressure. Thus it is mostly possible for the fracture to open up. To examine the state of the fracture during grouting, penetration length of the grout in real time has been estimated (figure 10) and ultimate jacking limit has established in span of 11 to 22 minutes (figure 11). It should be noticed that the estimated penetration length is based on the initial estimated fracture.

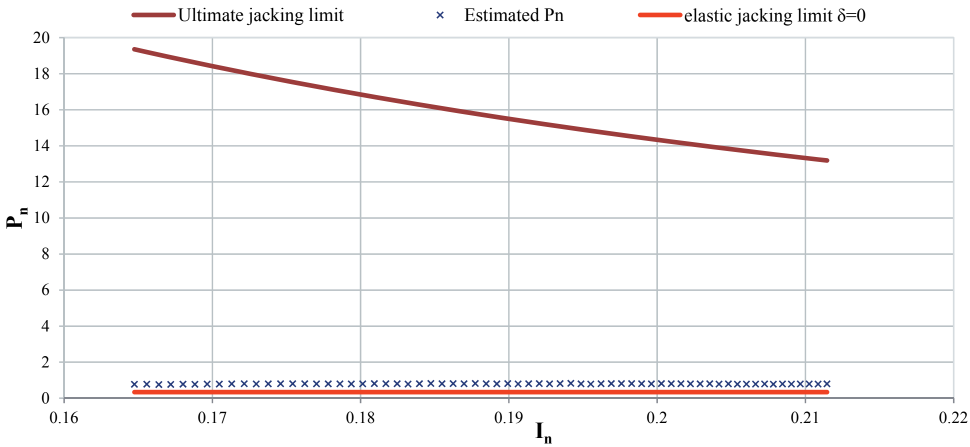

Figure 11 Estimated state of the fracture in compare with ultimate and elastic jacking limits.

From figure11 it is depicted that the grouting has been stop much sooner than arriving the ultimate limit i.e. since the fracture has been situated at deeper part, ultimate jacking would not happen in short spread. Due to the larger applied pressure in compare with the overburden, the fracture has opened up, but since the deformation is elastic, in releasing pumping pressure, this deformation will be recovered if the grout can be pumped out.

Due to existence of tectonic stresses that are in result of tectonic activities, the total horizontal stress is much higher than the stress induced by gravity (Palmström & Stille, 2010). "k" value is used as the ratio of horizontal stress to vertical stress and at shallow and moderate depth values of k are high (Hoek & Brown, 1980). Thus it is reasonable to apply a grouting pressure much higher than overburden. Since no deformation may happen in vertical fracture, in applying the pressure for long time, grout may flow out of the fracture (leakage). In the studied case, fracture has been dilated (horizontal fracture) or grout has been flowed out (vertical fracture), thus the pressure is needed to be reduced or grouting should be stopped sooner.

CONCLUSION

It was demonstrated that theoretical approach is robust method in estimating the spread and the state of the fracture in real time. Thus it allows grouting with highest possible pressure up to the point where spread requirements are fulfilled and no undesirable jacking happens. Estimating theoretically correct spread and formulating jacking limits based that will clarify ambiguities of the empirical methods.

Development of Real Time Grouting Control method in order to establish jacking limits and to estimate state of the fracture based on the grout spread was discussed and application of this theory was validated with data from a grouting work at Citybanan project. In this case, by considering grouting at a horizontal fracture, theoretical approach could estimate dilation of the fracture. Since the depth of the fracture from the surface is large in compare with penetration length, there is a large gap between elastic and ultimate jacking limits. Therefore the energy induced by applied pumping pressure in the fracture in span of the grout spread is much less than the energy that is required for the uplift. However the applied pressure is larger than the overburden and the fracture has been dilated. This elastic dilation may be beneficial as it can improve the penetrability and shorten the grouting time.

In developing this method, there were uncertainties connected to the geology and the material properties. Variation in rheological properties of the grout mix has been neglected. The variation of fracture aperture due to jacking has been neglected as well. Furthermore, discontinuities connected to the studied borehole have been simplified to one single horizontal fracture which is dominant for most of the grout flow. In case of vertical fractures, since horizontal stresses are much larger than the stresses due to overburden, elastic and ultimate jacking limits are larger and no deformation may occur. In Citybanan project, there are many vertical fractures under high horizontal stress, thus applying high pressure while using this theory to modify grouting procedure in real time (reducing pressure or stopping grouting when jacking or leakage occurs) is the optimum design.

REFERENCES

Brantberger, M., Stille, H. & Eriksson, M., 2000. Controlling Grout Spreading in Tunnel Grouting Analyses and Development of the GIN-method. *Tunneling and Underground Space Technology,* 15(4), pp. 343-352.

Ewert, F., 1996. The GIN Principle-a helpful method for rock grouting? part1 and 2. *International Water Power and Dam Construction,* February and April, 48(2), pp. part 1: pp17-19, Part 2: pp 36-40.

Fransson, Å. et al., 2012. *A swedish grouting design concept: Hydrualic testing and selection of grout.* New Orleans, proceeding of 4th International Conference on Grouting and Deep Mixing, pp. 1691-1700.
Gothäll, R. & Stille, H., 2009. Fracture dilation during grouting. *Tunnelling and underground space technology,* Volume 24, pp. 126-135.
Gustafson, G. & Stille, H., 2005. Stop criteria for cement grouting. *Felsbau,* 23(3), pp. 62-68.
Hoek, E & Brown E.T., 1980. *Underground Excavation in Rock*. Institute of Mining and Metallurgy, London
Houlsby, A., 1990. *CONSTRUCTION AND DESIGN OF CEMENT GROUTING, A guide to grouting in rock foundations.* New York: John Wiley and Sons.
Hässler, L., 1991. *Grouting of Rock - Simulation and Classification- PhD thesis,* Stockholm: Royal Institute of technology.
Kobayashi, S., Stille, H., Gustafson, G. & Stille, B., 2008. *Real Time Grouting Control Method, Development and application using Äspö HRL data,* Stockholm, Sweden: Swedish Nuclear Fuel and Waste Management Co (SKB).
Lombardi, G. & Deere, D., 1993. Grouting Design and Control Using the GIN Principle. *International Water Power and Dam Construction,* 45(6), pp. 15-22.
Palmström, A. & Stille H., 2010. *Rock Engineering*. London: Thomas Telford Limited.
Rafi, J. & Stille, H., 2012. *Applying "Real Time Grouting Control Method" in Sedimentary Rock with Gtvand Dam Data.* new orleans, s.n.
Rombough, V., Bonin, G. & Shuttle, D., 2006. *Penetrability Control of GIN Mixes During Fractured Rock Grouting.* 1-4 October, Vancouver, Canada, Proceeding of Sea to Sky Geotechnique: 59th Canadian Geotechnical Society Conference, pp. 528-535.
Shuttle , D., Rombough, V. & Bonin, G., 2007. *GIN Distilled.* [Online] Available at: http://www.groutline.com/articles.htm
Stille, B., 2010. *Experiences With Real Time Grouting Control Method.* Stockholm, Bergmekanikdag, pp. 119 -132.
Stille, H., Gustafson, G. & Hassler, L., 2012. Application of New Theories and Technology for Grouting of Dams and Foundations on Rock. *Geotechnical and Geological Engineering,* Volume 30, pp. 603-624.
Tsuji, M. et al., 2012. *Optimization of Grouting Procedure with RTGC and data from trialgrouting at Citiybanan in Stockholm,* Stokholm: SKB.
Weaver, K., 1991. *Dam Foundation Grouting.* New York, USA: ASCE.

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