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High Pressure Cement Based Grouts

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1 Generalities

Nearly two centuries ago the first cement slurry has been injected into ground to stabilize a rock that had settled.

Since that time the grouting techniques have been improved largely and were used in numberless civil structures in particular. Most of the improvements have been made in the development of the grouting equipments whereas the design criteria have been, for a long time, essentially based on empirical approaches resulting in some "grouting dogmas" sometimes difficult to eliminate (4).

The purpose of the present document is to summarise the present status of high-pressure cement grouting in Switzerland. The presently theoretical background adopted in practice for the design of grouting works will be described in Chapter 2. Recent experiences using high-pressure cement grouts will be described in Chapter 3, with particular emphasis on the Alptransit Railway Tunnels presently under construction and for which very high quality standards were necessary.

Some considerations on the contracting methods adopted in Switzerland are discussed in Chapter 4 highlighting the advantages and disadvantages of the various approaches.

A summary of the salient aspects of the recent Swiss experiences in high-pressure cement grouts and some information on the ongoing developments complete the present report.

It has to be mentioned that the present report will only deal with cement based grouts, with a particular emphasis on high-pressure grouts, in a fissured rock mass. Grouts in soils including high-pressure jetting will, therefore, not be taken into consideration.

A bibliography on the relevant recent Swiss literature in the field of cement grouts are given in Appendix 1. Some of the mentioned reports are unpublished and have been prepared as part of the construction works and site supervision activities.

The author would like to express his gratitude for the received support by various Swiss agencies and in particular to AlpTransit AG for having accepted to publish some relevant technical data related to the grouting works carried out recently.
2 Theoretical background for cement-based grouts

2.1 The aim of grouting

Within any grouting project it is essential to clearly define the purpose of the grouting works. In general the aims of grouting are to (3, 8):

- increase the strength
- reduce the deformability
- decrease the permeability

of a given rock mass.

The aim of the grouting works may be limited to one of these points or, in some cases, to a combination of them. The more clearly it is possible to define the objective to be attained, the better a correct design and an appropriate quality control will be possible. As for any other civil structure, it is felt of primary importance to clearly define the result to be achieved at the end of the grouting works already in the design stage.

Once the purposes to be achieved are clearly identified it has to be evaluated if cement based grouts are the appropriate method to reach them. It often occurs that unsatisfactory results are not due to an inaccurate grouting work done by the contractor but simply because cement based grouts are unsuitable for the purposes to be achieved. The first basic design step consist thus in the selection of the most suitable grout product which, in some cases, might be rather difficult. Although it is not the purpose to discuss all the aspects to be taken into consideration for the selection of the most suitable grout product, some relevant limits of cement based grouts are described in the next paragraph.

2.2 Limits of cement based grouts

When considering the limits of the use of cement grouts in rocks the following aspects have to be considered:

- the properties of the grout slurry during the injection process
- the properties of the hardened grout
- the properties of the rock mass.

Although a cement-based grout slurry presents some fluid properties, the behaviour in terms of penetrability depends significantly on the properties of the solid component, i.e. the cement. During the grouting process, the yield stress (or cohesion of a Bingham fluid) as well as the maximum size of the cement particles define essentially the penetrability of a grout mix for a given crack width and a given grouting pressure.

However, considering that the maximum size of cement particles range between 10-15 microns for an ultra fine cement to 100 micron for a common cement, the minimum crack width to be grouted using cement slurries is of the order of 0.05 to 0.1 mm. Cracks of opening less than 0.2-0.3 mm might only be filled with ultra fine cements, independently from the considered grouting pressure. The filling of cracks of less than 0.1 mm represents a difficult task which might be achieved only under favourable conditions. The "Dynamic Injection Method" is only exceptionally used in Europe. The opinion is that a step-by-step procedure involving a gradual
increase of the grouting pressure leads to similar final results as the dynamic method. In fact, in Europe, the grouting process is generally defined as a function of the injection rate, meaning that lower pressures are adopted in the case of high rates.

It has to be kept in mind that grout slurries behave differently from water in particular when fine cracks are considered. Water absorption tests, as often carried out in the investigation phase for grouting works, have thus to be used with care because the hydraulic behaviour of water differs largely from the behaviour of a cement slurry.

The second aspect refers to the properties of the hardened grout. Improving the penetrability of a slurry often results in a reduction of the mechanical properties of the hardened grout. In order to satisfy both requirements additives such as superplasticizers can be used. The requirements of the hardened grout are generally referred to:

• limited bleeding
• minimum strength

and under some particular conditions the

• porosity.

In order to avoid any excessive bleeding, the use of unstable cement based grouts is presently an exception in Europe. Commonly used criteria for the limits between stable and unstable grouts does, however, not exist. The stability requirements of a slurry is thus defined for each project and may also vary depending on the grouting purposes. Therefore, the bleeding requirements have to been taken into consideration already in the design phase in order to meet the project requirements.

As far as the strength and the porosity are concerned the requirements to be adopted will significantly influence the design of the slurry and the associated costs. It is therefore important to accurately evaluate the requirements in order to identify the most suitable slurry design. Excessive strength requirements might result in slurries of less penetrability. It has to be mentioned that the strength and porosity properties of concretes are essentially defined by the water cement (W/C) ratio.

The most difficult aspect to be evaluated are obviously the properties of the rock mass. However, it is essential to be aware of the limits for using cement grouts to improve the properties of a rock mass (1).

A rock mass typically includes families of fractures of variable dimensions and in some cases faulted zones. When grouting works are considered (in particular cement based grouts) a distinction has to be made between fractures and faults.

Fractures represent a discontinuity of the rock matrix generally oriented along preferred directions. They might be filled with air, water or fault gouge with variable characteristics.

Grouting of rock fractures using cement based slurries is possible as long as the crack width exceeds approximately 0.1 mm as previously mentioned. However, it has to be considered that the crack width varies for the same crack and for the various crack families. The penetration distance might thus vary significantly depending on the effective crack width. Therefore, the penetration distance should be looked at under a probabilistic perspective rather than following a deterministic evaluation. In addition, it has to be considered that the filling of one crack with cement slurry might, under some conditions, affect the nearby cracks by promoting their
closure. A contemporary grouting of cracks of various openings is rather difficult to achieve in particular when very fine cracks are considered.

The increase of the strength of a fractured rock mass by grouting the fractures is therefore only possible under exceptional conditions. In general, only the widest cracks will be filled thus having little effect on the strength properties of the entire rock mass.

Fault zones may show highly variable properties and are, in some cases, more similar to soils rather than rocks. The groutability of these zones may thus vary significantly depending on the properties of the fault material. In addition, it should be considered that these properties may vary within the fault zone since the borders often represent a less developed fragmentation than the fault centre.

Although it is not possible to consider all the possible cases, it is felt important to point out that cement grouts may improve both the mechanical and permeability properties of faulted zones. An increase of the strength as well as a reduction of the deformability are, in general, easier to achieve in the case of faults as compared to a fractured rock mass. In addition, it has to be considered that fine spaces (indicatively less than 0.1 mm) might not be grouted using cement based slurries - independently from the used pressure.

The distinction between the treatment of a fractured rock mass or the grouting of faults is therefore important when defining the purposes of a grouting work.

### 2.3 Outdated practices

In relation to high-pressure cement based grouts, it is felt important to comment some practices commonly used in the past and sometimes in recent works (7).

Although rarely used in Europe, the practice to use various mixes starting from a thin highly unstable mix to progressively thicker ones is not yet completely eliminated.

As previously mentioned, the groutability of a given crack essentially depends on the size of the largest cement particle in relation to the crack width. Although a high grouting pressure might increase the crack width, its influence is basically limited to the progress of the grout penetration. In case relatively fine cracks have to be grouted, the use of a thin-to-thick process is thus useless as long as the same cement material is used. In the contrary, the poor mechanical properties of the unstable slurries might result in filling the cracks with a relatively poor material.

A variation of the slurry composition only makes sense if progressively finer cements are used. This procedure may allow to start with the grouting of the wider cracks followed by the filling of progressively fines ones.

A second concept widely used in the past is the "presso-filtration" assuming that the excess water will be pressed out into fine cracks. Considering again that the groutability of a given crack is essentially given by the relation between the maximum size of the cement grain and the crack width, pressing out the water will not improve the groutability. In the case of high-pressure grouting, the concept of "presso-filtration" is progressively abandoned also.

Another widely used concept is the "refusal pressure" defined as the pressure for which the rock mass "refuses" to absorb more grout.
The experiences as well as theoretical considerations show that the grouting process can be continued by increasing the pressure. The limit is thus given by the equipment and the purpose of the grout but not by the rock mass itself. On the other hand, the grouting process may be stopped by reducing or limiting the grouting pressure.

Due to the variability of the rock fractures the relation between the volume taken by the rock mass and the grouting process applied is rather difficult to quantify. In general terms, for a given slurry and a grouted segment a limiting pressure exists for which the grout flow becomes zero.

This situation clearly shows the difference in the hydraulic behaviour between water and a cement based slurry. For water, a limiting pressure for zero flow does not exist (except in the theoretical case of a watertight rock).

The last widely used concept to be discussed refers to the "hydro-fracturing". Confusion is often made between hydro-jacking, which is the opening of existing joints or cracks and the hydro-fracturing, which is the formation of new cracks.

If cracks exist in a given direction it is more probable that existing joints or cracks are opened rather than forming new ones. The formation of a new crack is thus highly improbable except in the case of particular stress conditions within the rock mass.

One should consider that grouting is done by introducing a grout mix into cracks, thus a certain opening of the cracks is unavoidable and is even desired as considered in the French "clauquage" method (REF?).

2.4 The GIN Approach

The GIN approach was introduced approximately 20 years ago (REF) and is based on the idea that the possible heave of the ground, the risk of hydro-fracturing, the amount of hydro-jacking, the reach of the grout slurry and, in general, the reduction or the benefit achieved with the grouting is a combination of the properties of the grout mix, the applied pressure and the volume of injected grout. It is obvious that damages may occur in case of high pressures in combination with high injection volumes. On the other hand, high pressures with no injected grout or high injected volumes at low pressures can hardly be of harm.

In fact, neither the pressure itself nor the volume itself are determining factors but only their combination is of primary interest. Based on these considerations the concept of "Grouting Intensity" has been developed approximately 20 years ago and is actually widely used in many cement based grouting of fractured rocks. The Grouting Intensity Number or G.I.N.–value has been defined as

\[ GIN = p \cdot V \]

usually expressed as bar·lt/m

The GIN principle, as shown in Figure 1, is based on limiting the grouting pressure according to a given GIN value (red curve) which prevents any excessive combination of pressure and volume and thus any unwanted damage.
Figure 1: Principle of the GIN concept. The red curve indicates a constant GIN value (7).

The GIN value in conjunction with a maximum pressure and a maximum volume has to be selected for each case taking into account:

- the geological conditions
- the scope of the grouting works
- the geometrical definition.

Note that the correct application of the GIN method is not sufficient to insure a satisfactory result (5). Various aspects have to be defined within the design of the grouting process. This includes in particular:

- the selection of the correct grout slurry
- the selection of the distance between the drilling holes
- the grouting process (downward or upward with single or double packer)
- the definition of the minimum and maximum grouting volume per time unit
- the minimum pressure to be reached for a grouted hole to be considered satisfactory.

The complete design of a grouting work thus requires the definition of several parameters which cannot be selected casually or simply based on "experience". Test grouts are essential to optimise the numerous parameters required for a successful grouting considering costs, schedule, and durability.
2.5 Controls during and after the grouting

2.5.1 Controls during the grouting process

The purpose of the quality controls is to verify that the works are carried out according to the specifications and according to the requirements given by the engineer. During the grouting process it is often useful to collect the grouting parameters in order to optimise some parameters as well as the entire process. The importance of this very essential task is often neglected by the engineer. It is useless that the contractor feeds the engineers with a huge amount of data (drilling and grouting records) if no appropriate analysis of these data is carried out afterwards. Note that the documentation of the grouting parameters is of particular use for optimisation purposes during the grouting process. It is useless to evaluate the data only once the grouting work has been completed because no measures can be taken in order to improve the work.

The parameters measured during the drilling and grouting process are basically the following (6):

- Drilling parameters with some data on drilling deviations
- Grout cohesion (with PCM), and Marsh cone time
- Bleeding after 15, 30, 60 and 90 min.
- Grout temperature and specific weight
- In some cases, mechanical properties of the hardened grout (7, 14 or 28 days)
- Pressure and volume, GIN value per grouted step.

The last point is essential to evaluate if the grouting is carried out properly with a good probability of achieving the required result. The grout take is usually defined as litres per m$^3$ of grout take and expressed as lt/m$^3$. Basically, the following situations might occur:

a) Low grout take (typically less than 15-20 lt/m$^3$)

In case the grout take is very low, the penetration of the grout might be insufficient or only a very limited number of cracks are filled. A small distance between the injection boreholes or a modification of the slurry would be helpful in case also thin cracks have to be injected.

b) Average grout takes (typically 40-100 lt/m$^3$)

In case the GIN value is reached for grout takes in the range of typically 40-100 lt/m$^3$, the grouting parameters are usually adequate. In this case a secondary grouting phase with drilling holes in between the primary boreholes will show a reduction of the take demonstrating that a progressive filling of the rock fractures is achieved.

c) High grout takes (typically above 150 lt/m$^3$)

In this case the grout mix may need to be adapted in order to limit the useless propagation of the slurry. Only in very highly fractured rocks the average grout take exceeds 150 lt/m$^3$ with a rather confined grout propagation. In general, in order to better confine the grout, a “thicker” slurry has to be used.
The adopted unit corresponds to litres of grout per metre borehole (lt/m). The injected cement amount per metre depends obviously from the adopted w/c ratio of the slurry. The previous indicative values apply only for stable grout mixes.

The analysis of the grouting parameters is not only important during the initial working phase but during the entire grouting process. In fact the properties of the rock may vary (variable fractures intensity, faults, etc.) leading to an adaptation of the grouting to the variable conditions.

2.5.2 Controls after the grouting

The purpose of the controls after completing the grouting work is to verify that the objectives have been reached. The control measures to be carried out, therefore, depend on the main purposes of the grouting activities.

The most common control procedure is to verify the reduction of the permeability of the grouted body. The reduction of the permeability can be quantified by permeability tests carried out before and after the grouting work or directly by measuring the inflow into a drainage borehole. In addition, some useful information is also provided by carrying out transmissivity tests as discussed in chapter 3 in more detail.

A proper interpretation and quantification of the controls carried out at the end are only possible if the objectives to be reached have been clearly defined. The possibility to quantify the objectives of the grouting work is essential to quantify and evaluate the success of the operation. In other words, a vague definition of the purposes of an injection does not allow quantifying any obtained result.
3  Recent Swiss experiences in high pressure cement grouts

3.1  Generalities

High-pressure cement grouts have been used recently in Switzerland in particular within the construction works of the new AlpTransit railway tunnels for both Lötschberg and Gotthard. The 17 km long Lötschberg tunnel has been completed and will soon go into complete commercial operation, whereas the 52 km long Gotthard tunnel is still under construction. Approximately 2/3 of the total tunnel excavation works have been carried out to date.

Cement based grouts have been used for the construction of both tunnels, although the more extensive and difficult grouting works have been done within the construction of the Gotthard base tunnel. In the present chapter the relevant aspects of the grouting works carried out at the Gotthard base tunnel will be summarized. Some aspects related to the works carried out at the Lötschberg tunnel will complete the chapter.

3.2  Grouting at the Gotthard base Tunnel

3.2.1  Purposes of the grouting at the Gotthard base tunnel.

Figure 2 shows the schematic alignment of the Gotthard Base Tunnel (GBT) with the relative position of the Nalps, St. Maria and Curnera water reservoirs. These reservoirs used for power generation are impounded by double curvature arch dams of 117 to 153 m height.

Following the experience of the Zeuzier dam (1978) and the recently collected data following the construction of the Gottard highway tunnel (Survey of 1999), it is proven that the drainage of a rock mass causes deformations at the surface in the same way as it occurs for saturated
soils. Within the GBT project a specific survey system has thus been installed at the surface along the tunnel alignment for the monitoring of the surface behaviour. The measurements carried out since 1999 have shown a close relation between the progress of the tunnel excavation and the permanent deformations at the surface. As shown in Figure 3, the settlements at the surface are also inducing horizontal movements of both valley flanks (9). These horizontal movements originating in the curvature of the settlement surface, together with some relative deformations represent the most severe conditions for the arch dams located 1200 to 1400 m above the tunnel.

At present, the distance between both valley flanks at the crest elevation of the Nalps dam has been reduced by 13 mm leading to a modification to the operational conditions of the dam structure.

![Figure 3: Schematic layout of the settlement caused by the drainage of a rock mass.](image)

The settlements are caused by a modification of the stress distribution in the rock mass due to the increase of the effective stresses induced by the reduction of the water pressure. The increase of the effective stresses induces a closure of the fractures in the rock mass causing the observed deformations at the surface.

Although it is not the purpose in the present report to explain in detail the phenomena, the observed behaviour might be modelled using the FES (Fissured, Elastic, Saturated) model developed nearly 20 years ago (11).

The extent of the surface deformations is clearly related to the amount of water drained from the rock mass. In fact, assuming a saturated rock mass body, the amount of water drained from the rock mass (tunnel inflows minus regeneration of the water table from the surface) has to correspond to the settlement volume. The only method to limit the settlements and the deformations at the surface is to reduce (as much as possible) the inflows into the tunnel both during and after the excavation progress. In the vicinity of the arch dams particularly restrictive conditions with respect to the maximum allowable inflows to the tunnel have thus been imposed to the contractor with the purpose of keeping the observed horizontal deformations at the dam
crest within an acceptable range. The acceptable range depends from dam to dam. In the case of the Nalps dam the maximum allowed distance reduction at the crest elevation was set to 50 mm at final stage.

More information to the observed phenomena and the relations between the inflow rates and the observed settlements on the surface are given in /9/ and /10/.

3.2.2 Inflows occurred in September 2006

During the excavation progress approximately 300 m before underpassing the Nalps arch dam a permeable fault zone was reached which was not identified with the systematic investigation drillings (13). On September 13th 2006 up to 12 l/s were flowing into the tunnel from the fault zone (N°44). The flows were leaking through the temporary shotcrete lining. Figure 4 shows the conditions at the tunnel front a few days crossing the fault zone.

![Figure 4: Tunnel front after crossing the fault zone. Water inflows are located on the right side of the tunnel profile.](image)

This relatively small inflow has no direct effect on the excavation progress but caused immediate deformations at the surface approximately 1400 m above the tunnel. The horizontal deformation rate measured at the surface reached 2.5 mm/month at the dam crest. The measured non-reversible deformations at the surface are shown in Figure 5.
Due to the difficulty to predict the long-term behaviour of the settlements, the tunnel excavation was immediately stopped and a preliminary investigation program was started. The purpose of this program was mainly to identify the geometry of the fault and to identify its relevant characteristics in order to be able to define the most suitable grouting procedure. Only once the results of the investigation program became available, the definitive decision for a grouting campaign was discussed within the expert team.

3.2.3 Preliminary investigations

The purpose of the preliminary investigations was to obtain more precise information of the fault zone. The investigation consisted essentially in 3 core drillings located outside the tunnel profile with a maximum length of 150 m and crossing the fault zone. Based on these drillings the geological model was established as shown in Figure 6.

From the geological point of view, the fault zone is located in gneissic rocks and consists mainly of highly fractured hard rocks up to some mylonitic zones. The width of the fault zone is variable and it is estimated to be between 5 to 10 m. The hydrostatic water pressure in the rock mass prior to the excavation was estimated to be about 120 bars, whereas during the core drillings a maximum pressure of only 50 bars was measured due to drainage effect of the tunnel. Note that all drillings were carried out with a preventer protection in order to avoid any possible uncontrolled water release.
After the completion of the core drillings the boreholes were used to carry out hydrotests in order to evaluate the transmissivity of the fault zone. These tests showed a transmissivity of approximately $2.4 \times 10^{-7} \text{ m}^2/\text{s}$. To mention that within the fault zone the rock was completely disrupted with no defined structure.

According to the results of the preliminary investigations showing a well defined permeable fault zone, the decision was taken on October 11th 2006 to proceed with an extensive grouting campaign in order to reduce the flow rate to the tunnel. The requirements of the grouting were to reduce the steady state inflow into the tunnel from 8 l/s ideally to 2 l/s with a maximum acceptable value of 3 l/s on the threatened tunnel stretch. The expert team recommended to the owner to reduce the permeability of the fault in order to limit the risk of any excessive horizontal deformations at the surface and thus the risk to damage the Nalps dam. The limit of 3 l/s was considered acceptable to avoid the risk of an excessive surface deformation in the dam area.

### 3.2.4 Design and test activities

Based on the above mentioned objective, the following main activities were carried out or supervised by the design team:

- Completion of the tunnel excavation through the entire faulted zone
- Reinforcement of the lining in order to support high grouting pressures
- Preliminary selection of the grout slurry and grouting parameters

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**Figure 6:** Geological model of the fault zone as established according to the 3 core drillings carried out outside the tunnel profile.
• Definition of grouting tests
• Definition of grouting scheme and procedures
• Implementation of the design measures and field controls.

Regarding the preliminary tests on the grout slurries, the following requirements were defined by the designer:

• Very stable grout with less than 3% bleeding after 2 hours
• Flow time according to Marsh funnel 36 sec<T<42 sec
• No use of bentonite
• Use of conventional and ultrafine cement in order to achieve the project requirements.

Since the fault zone was disclosed during the excavation progress (and not as part of investigation drillings) it was considered preferable for operational reasons to complete the excavation of the fault zone before starting with the grouting works.

During this design step various options for grout products other than cement based products were evaluated but, finally, only cement based grouts were considered.

Considering the high temperatures (approximately 40°C) preliminary tests on the grout slurries were carried out directly in the tunnel using the water and the grouting equipment of the contractor. The reduction of the temperature of the rock mass to be grouted was not considered feasible.

Preliminary laboratory tests were carried out using the ultrafine cement SPINOR A12 with a maximum grain size of 12 micron and a Blaine value of 12’000 cm²/g, and a high Puzzolane Type III cement (GEOROC 50) with a Blaine value of 7’000 cm²/g. Note that, although the Blaine value of the pozzolanic cement is relatively high, the size of the maximum grain is approx 100 Micron thus equal to a common cement. The second cement was selected due to the high content of fine particles reducing the bleeding of the slurry.

Based on these preliminary tests various grout slurries were tested in order to identify the most suitable one. In order to fulfill all the requirements it was finally decided to prepare two slurries. A first slurry was used to confine the slurry and to seal the largest fractures (extensive contact grouting), whereas the second and finer slurry was used to grout the finer cracks in the faulted zone. Following various tests the following composition of slurries were finally selected:

• SPINOR A12 at w/c=1.0 and 2% superplasticizer, and
• Georoc H50 at w/c=0.9 without any additive

The GEOROC cement is commonly used in Europe for cement grouts due to the high fine content resulting in a good stability of the slurry. The SPINOR A12 is an ultrafine cement with a very small maximum grain size thus allowing the injection of very fine fissures. It is important to notice that the tests have been carried out using the same water and the equipment (turbomixer, grouting pumps, etc.) as used by the contractor in order to insure that the obtained results are representative for the subsequent application conditions.

With regards to the grouting procedure a maximum pressure of 150 bars was selected considering a natural watertable at 120 bars. The selection of the maximum pressure of 150 bars was based on the concept to insure some "crack opening”. Therefore pressure conditions slightly
above the "natural" conditions were selected. Due to the mylonitic character of the fault which
cannot be directly compared with a fractured rock mass, the GIN concept could not be applied.
It was decided to inject into each hole until the maximum pressure was reached in order to fill as
many as possible voids.

**Grouting and drilling tests** carried out in the fault zone above the tunnel showed a satisfactory
injectability so the adopted slurries were considered adequate for the grouting.

The generally adopted grouting concept is shown in **Figure 7** indicating the contact grouting,
the investigation and drainage drillings and the main grout curtains.

The developed concept consisted in the realisation of an inner ring of contact groutings (using
the GEOROC slurry) with the purpose of confining the grout into the fault zone. Once this
confinement was realized it was possible to inject into the fault at very high pressures (150 bars)
using the Ultrafine Cement. Two rings or curtains have been planned, depending on the
obtained permeability reduction after the grouting of each ring.

In a first step, it was necessary to reinforce the existing lining because the contact grouting had
to be carried out at a maximum pressure of 30 bars. 4-5 m long boreholes were drilled radially
and systematically grouted from the bottom to the top of the tunnel profile.

**Figure 7:** General grouting concept including the drainage drillings (3 drillings used as
investigation core drillings), the contact grouting and the main grout curtains
(yellow and pink rings).

**Figure 8** shows the reinforced lining and the schematic layout of the contact groutings.

Commercial-in-Confidence
Figure 8: Reinforced lining and schematic layout of the contact grouting.

The position of the main grouting curtains carried out in the fault zone is shown in Figure 9. Note that Figures 8 and 9 do not show the drainage drillings located outside of the grouted areas.

Figure 9: Schematic view of the grouted zone with the position of the grout curtains into the fault zone.
The drainage boreholes were kept open during the entire drilling process. Only exceptionally some cement slurry was found in the drainage water. The schematic view of the tunnel section after the grouting is shown in Figure 10.

Some difficulties were encountered during the drilling works which had to be carried out under difficult conditions. It has also to be mentioned that all the groutings were carried out without the use of double packers because they are difficult to install under such high pressures.

The boreholes of the grout curtain were equipped with 10 m long grouted standpipes in order to limit grout losses near the lining. The packers were placed into the standpipe. Note that the inflation pressure of the packer was 250 bar. At these high pressures, a packer placed directly into the drilling hole would be rapidly damaged.

Figure 10: Schematic cross section of the tunnel with the various grouting works.

3.2.5 Tests and final results

Following the grouting of the first ring, the efficiency test was performed in the closure of the drainage boreholes and the control of the total leakage into the tunnel. This relatively simple procedure allowed a quick verification of the efficiency of the grout works in terms of permeability reduction. After the first test carried out between February 2nd and February 5th 2007, the measured leakage was 4.1 l/s. Based on this result additional groutings were carried out to further reduce the leakage. The second and final test was made between March 7th and March 9th 2007 and found a total leakage of slightly less than 3.0 l/s. Although the optimum value of 2.0 l/s was not achieved the obtained value was acceptable and, therefore, the groutings were suspended.
The total grout take of both contact and fault grouting is shown in Figure 11. The figure shows the final results after some permeability tests have been carried out.

**Figure 11:** Total grout take at the fault zone N° 44. Tm means tunnel meter.

The distribution of the grout take on the tunnel profile is shown in Figure 12, where only the take of the fault grouting is displayed. The Table below the figure indicates the totally drilled meters; the grouted quantities; and the time required for the works.
<table>
<thead>
<tr>
<th>Measure</th>
<th>Drillings [m]</th>
<th>Grouted quantity [ltr]</th>
<th>Work Duration [Days]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contact grouting</td>
<td>3'900</td>
<td>70'700</td>
<td>42</td>
</tr>
<tr>
<td>Fault grouting</td>
<td>8'900</td>
<td>103'600</td>
<td>95</td>
</tr>
<tr>
<td>Total</td>
<td>12'800</td>
<td>174'300</td>
<td>102</td>
</tr>
</tbody>
</table>

**Figure 12**: Distribution of the grout take around the tunnel for the fault grouting and table of total quantities of drilling length, grout quantity and work duration.

### 3.3 Conclusions on the grouting of the fault N°44

The very high requirements as well as the exceptional conditions for the grouting of the fault zone N°44 at the Gotthard base tunnel required relatively unusual design solutions and the use of relatively heavy equipment in particular with regards to the drilling machines. However, despite exceptional requirements and difficult operational conditions the collaboration between the designer and the contractor was successful and allowed to quickly optimise the various design and operational aspects involved in the realisation of the grouting works. The actual inflow in the fault zone is nearly constant at approx. 2.8 l/s.

### 3.4 Grouting at the Ferden station in the Lötschberg tunnel

At the Ferden section, the Lötschberg railway tunnel is underpassing the thermal springs of Leukerbad. In this highly sensitive area a fault zone was encountered located relatively close to the area assumed to supply the springs. The water and rock temperature was approx. 43 °C. The tunnel owner defined that the maximum inflow into the tunnel in this section had to be limited to 2 l/s at a natural water table pressure of 120 bars.

As a consequence, the following work sequence was adopted (12):

- The excavation was stopped 50 m before the expected fault zone.
- An investigation borehole was drilled in the tunnel axis (with preventer).
- The excavation was continued up to 30 m before the fault zone.
- Drilling and grouting of the first external ring indicated in grey colour in Figure 13.
- 4 investigation and test boreholes were drilled at the inner side of the grouting ring to check the result.
- Drilling and grouting of the second ring indicated in red colour in Figure 13.
- 4 test drillings inside the future tunnel profile.
- Excavation of the tunnel through the fault zone.

The total inflow into the tunnel in case no grouting would have been carried out was not estimated.

**Figure 13** shows the general layout of the grouting scheme in the fault zone (both rings).
Figure 13: Grouting concept of the fault zone at the Lötschberg tunnel.

In total 4 different slurries have been used within the various grouting phases. The overall characteristics of the used slurries can be summarized as follows:

- Ultrafine Cement with silikafume
- w/c = 0.55-1.0
- Marsh cone time 30-41 s
- Bleeding < 2-3 %

The maximum grouting pressures were limited to 200 bars and some Lugeon tests were carried out before and after the grouting in order to check the grouting results. According to the Lugeon and the transmissivity tests, the expected inflow after the tunnel excavation was estimated according to usual approaches.

After the grouting a total inflow of 2-2.2 l/s was measured along the grouted section.

Figure 14 shows the installed equipment for the drilling and grouting works.
Figure 14: Photo of the drilling and grouting installations at the Lötschberg tunnel in the Ferden area.

In total, nearly 6’000 m of drillings were necessary with a totally injected volume of 233’000 l in order to reduce the inflows to the required value. The total cost of the drilling and grouting work was approximately 7 Mio. US$ of which nearly 55 % was for the drilling works, 20 % for the installations and only about 25 % for materials supply and the actual grouting.
4 Contracting methods

4.1 Generalities

Usually, during the design phase of the grouting works, the engineer in collaboration with the owner, has to identify and to select the most appropriate contracting form. The following two approaches are possible and have been adopted in Switzerland on various opportunities:

- Contract and payment exclusively according to quantities (usually drilled lengths and grouted slurry).
- Separate payment for the installation of the equipment and for the operation of the contractor’s machines. In addition, payments are due according to quantities (usually drilling length and grouting volume).
- Lump sum payment without any consideration on quantities.

Lump sum contracts in the area of grouting are rather unusual in Switzerland and are generally confined to very limited works.

The specifications for grouting works are often defining a range of possible mixes and cement properties with some indication on maximum pressures. Sometimes the GIN value is defined. Only in recent years some outdated and "standard" specifications have been progressively abandoned although many grouting works are still carried out according to old practices.

In many cases, the final selection of the grout mix is left to the contractor or to some product supplier who, however, are not aware of the purposes and the requirements of the grouting work.

The optimum design of the entire grouting process can only be achieved through a close collaboration between the engineer and the contractor. This collaboration does, however, not influence the responsibilities of each partner within the construction works. In many cases the distinction between the design and the realisation of a grouting works is not clear as for other civil works.

4.2 Responsibility of the contractor and the designer

Due to the sharing of the responsibility to attain the expected result within a given time and cost frame, the situation is often unclear and causes many disputes between the owner, the engineer and the contractor. It is seen as important for a grouting work that all participants are aware of their part of responsibility in the progress of the grouting works. It should also be mentioned that no "European standard" exists that would clearly define the responsibility sharing between the various partners.

According to the author, the following aspects have to be considered:

- Grouting works require (as any other underground activity) large experience and should, in any case, be designed and carried out by experienced partners. Grouting involves knowledge of geology and rock mechanics, design of grout mixes and evaluation of the properties of the various products, drilling works, grouting procedures and control measures. Inexperienced designers and/or contractors are the best premises for disputes and misunderstandings during the work.
Similarly to any other civil work, the design responsibility should be in the hands of the engineer except in case of a General Contractor Agreement. Even in this latter case the risk that the General Contractor will attribute unexpected activities or delays to the geological conditions of the rock mass to be grouted is high.

The experience of specialized contractors provides an important contribution to the optimization possibilities of a given work particularly with regards to the definition of the most appropriate procedures, grouting mixes, additives and equipments. However, this contribution does not reduce in any form the design responsibility of the engineer.

According to the previous observations, the following aspects should be considered in the definition of a contract related to grouting works:

- Who is in charge of the design and takes the design responsibility?
- What will be the extent of the responsibility of the contractor?
- Who will take the responsibility for the geological risk and what shall be included in this risk?

Only once the responsibilities are clearly defined, it is possible to complete the various documents constituting a contract for grouting works. It is felt reasonable to distribute the responsibilities according to the following general guidelines. Too often it occurs that the causes of unsatisfactory grout works (in terms of final results, costs or schedules) are attributed to the contractor who often has much less information than the owner or the engineer.

Based on the recent Swiss experience, especially in the case of exceptional conditions, the following guidelines for the sharing of responsibility are recommended:

- The design of the grouting works including the definition of all procedures shall be carried out and under the responsibility of a specialized designer. He shall also identify all preliminary investigations on the rock and on the grout process in order for the contractor to be able to select the appropriate equipment.
- Preliminary grouting tests are essential for the optimization of the mix and for the procedures. These tests may be carried out before or after the contract award. Both alternatives involve some advantages and disadvantages. If the tests are carried out before the contract award, the results are available to all the bidders. However, this procedure is time consuming and requires a separate contract. In case the test grouting is carried out after the contract award, the final design can only be defined after the contract award. If the tests are performed by the contractor to be charged with the grouting works is advantageous for the test validity.
- The contractor must be informed on all available activities and information and he must have the possibility to suggest some optimizations. However, the design responsibility remains in the hands of the designer. The responsibility of the contractor is essentially to provide the adequate equipment and the experienced staff that is capable of carrying out all work phases planned.
- For unpredictable geological conditions neither the contractor nor the engineer can be held responsible. However, it is within the obligations of the engineer to inform the client on the possible risks and carry out the investigations required to minimize the risks (e.g. unforeseen events) to a reasonable low level.
5 Conclusions

The recent experience in the design and implementation of high-pressure cement based grouts in Switzerland can be summarized as follows:

- Appropriate design, combined with preliminary tests is essential to rapidly optimize the grouting procedures. A clear definition of the purposes and the final results to be achieved, in combination with appropriate investigations and preliminary tests, are the best premises for successful grouting works. The design of the grouting work include the definition of the slurry, applied grouting pressures and volumes, GIN value, drilling schemes and sequences, controls during and after the grouting, etc.

- Grouting works require experienced engineers and contractors, especially in case of difficult conditions in terms of geology, logistics and schedules. The lack of experience of the engineer or the contractor often causes misunderstandings, excessive expectations and subsequent disputes between the various partners.

- With the increase of grouting pressures, higher risks are usually related to the drilling works which, in some cases, have to be carried out under preventer protection. The drilling works are generally more time consuming than the grouting itself.

- As far as the grouting is concerned, the allowable maximum pressure gradient in a rock must be considered also in case of high pressure grouts. Assuming a typical maximum gradient of 10bar/m results in 20 m long drillings for a 200 bar grouting. High pressure grouting thus usually require long grouting boreholes.

- No optimum contracting form exists and each option involves advantages and disadvantages. The risk of poor results or unexpected cost increases or deviations from schedule can be reduced if an experienced engineer and an experienced contractor is involved. In any case, the responsibility of each party in the design and realization of the grouting works have to be defined clearly.

From the methodological point of view, some new concepts have been introduced during the last 20 years in Europe and these concepts are progressively applied in practice abandoning some outdated "dogmas". However, it should be considered that grouting works, especially under difficult conditions, require specific experience from both the design and the realization points of view.
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