İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

PRE AND POST INJECTIONS IN TBM EXCAVATION IN DIFFICULT GROUND CONDITIONS, TWO EXAMPLES FROM TURKEY

M.Sc. Thesis by
Suat SEVEN

Department : **Mining Engineering**

Programme : **Mining Engineering**

JUNE 2010

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Suat SEVEN
(505071016)

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Supervisor : **Prof. Dr. Nuh BILGIN (ITU)**
Members of the Examining Committee : **Prof. Dr. Hasan ERGIN (ITU)**
Assoc. Prof. Dr. Atac BASCETIN (IU)

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TBM PROJELERİNDE ZORLU ZEMİN KOŞULLARINDA ÖN VE SONRADAN ENJEKSİYON UYGULAMALARI, TÜRKİYE'DEN İKİ FARKLI ÖRNEK UYGULAMA

YÜKSEK LİSANS TEZİ
Suat SEVEN
(505071016)

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Tez Danışmanı : Prof. Dr. Nuh BİLGİN (İTÜ)
Diğer Jüri Üyeleri : Prof. Dr. Hasan ERGİN (İTÜ)
Doç. Dr. Ataç ÇETİN (İÜ)

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FOREWORD

First and foremost I offer my sincerest gratitude to my supervisor, Prof. Dr. Nuh BILGIN, who has supported me throughout my thesis with his patience and knowledge whilst allowing me the room to work in my own way. I attribute the level of my Masters degree to his encouragement and effort and without him this thesis, too, would not have been completed or written. One simply could not wish for a better or friendlier supervisor. I also would like to make a special reference to Mr. Selahattin YILMAZ who was the TBM manager of Mavi Tunnel project. Without his corporation I could not have gotten such relevant data. Finally pleasure to thank those who made this thesis possible such as my wife who gave me the moral support, my parents for supporting me throughout all my studies at University and my classmate mechanical engineer Sami Enis ARIOGLU who shared his valuable knowledge with me.

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Mining Engineering

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ABBREVIATIONS

TBM : Tunnel Boring Machine
EPB : Earth Pressure Balance
DS : Double Shield
PU : Polyurethane
PUR : Polyurea
WPT : Water Pressure Testing
APT : American Petroleum Instıtute
Kpf : Coefficient of pressure filtration
ISO : International organization for standardization
ISRO : International Society of Rock Mechanics
HK : Herrenknecht
RETC : Rapid Excavation &Tunnelling Conference
kW : Kilowatt
kN : Kilonewton
psi : Pound per square inch
ft : Feet

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PRE AND POST INJECTIONS IN TBM EXCAVATION IN DIFFICULT GROUND CONDITIONS, TWO EXAMPLES FROM TURKEY

SUMMARY

For over than 30 years TBM's were successfully utilised all around the world in a variety of geological formations. However in some circumstances TBM's required the execution of by-pass tunnels and other special hand mining works to face special extremely poor conditions, and mainly: important and rapid squeezing phenomena, very unstable tunnel faces (flowing ground) and combinations of the two.

To advance the TBM's under these conditions it is necessary to make large use of expanding foams to fill large voids in front and over the cutterhead. In addition, several patterns of cement or chemically grouted micro-piles should be executed from inside the TBM shield to improve the ground characteristics and stability ahead of the machine.

Despite these extreme conditions and the restrictions to the tunneling operations with the help of today's advanced technology the TBM's are able to advance without requiring any bypass or hand mining excavation and without the shield to get trapped.

This thesis describes the technologies and methods adopted to overcome these special geological conditions in different TBM projects as well as the possible improvements and modification points to the machines, which will ease the operation after facing the problem. Additionally two case study projects from Turkey is analyzed with details.

TBM PROJELERİNDE ZORLU ZEMİN KOŞULLARINDA ÖN VE SONRADAN ENJEKSİYON UYGULAMALARI, TÜRKİYE'DEN İKİ FARKLI ÖRNEK UYGULAMA

ÖZET

30 yılı aşkın süredir TBM'ler dünyanın her tarafında farklı jeolojik formasyonlarda başarıyla kullanıldılar. Buna rağmen aşırı zayıf zemin şartlarında ve özellikle hızlı deformasyon ve sıkışmanın yaşandığı zeminlerle stabil olmayan ayna (akan zeminler) ya da her ikisinin beraber yaşandığı bazı durumlarda by pass tünelleri ya da elle galeri açmak zorunluluğu doğmuştur.

Bu şartlarda TBM'lerin ilerleyebilmesini temin edebilmek için kesici kafanın önünde oluşan büyük boşlukları yüksek miktarlarda şişen köpük malzemeleriyle doldurmak gerekliliği doğmuştur. Buna ek olarak zemin karakteristiklerini iyileştirmek ve makine önündeki stabiliteyi sağlayabilmek amacıyla TBM içerisinden farklı paternlerde çimento ve kimyasal grout ile doldurulmuş mikro-kazık uygulamaları yapmak durumunda kalınmıştır.

Tünelciliği etkileyen tüm bu aşırı zorlu şartlara rağmen günümüz ileri teknolojisi sayesinde TBM tünellerinde by pass tüneller, elle açılan galeriler ve şıkışma problemleri olmadan da ilerleme yapmak mümkün hale gelmiştir.

Bu tez farklı TBM projelerinde yaşanan özel jeolojik şartların aşılmasına yönelik teknoloji ve metodları, sorunla karşılaşıldıktan sonra operasyonu kolaylaştırmaya yönelik makinalarda yapılabilecek iyileştirme ve modifikasyonları incelemektedir. Tüm bunlara ek olarak Türkiye'den iki örnek vaka detaylı analiz edilmiştir.

1. INTRODUCTION

Hard rock TBMs are very often open machine layouts, with just a part roof shield over the front of the machine. The whole system is designed for fast face advance and the economy of the project often depends entirely on the rate of advance. Typically, hard rock conditions will demonstrate stable and good ground for a major part of the tunnel. However, short sections of crushed shear zones with clay and gouge material may cause serious time delays. Spiling rock bolting has been proven to be very efficient under such circumstances, provided the fully grouted rebar bolts can be placed in a controlled and practical manner.

Environmental restrictions are becoming more and more a part of tunnel excavation projects. Even hard rock TBM tunnelling may require a strict ground water control, because of consequences on the ground surface from lowering the ground water level. Such consequences may be surface settlement, negative influence on ground water resources or damage to vegetation.

1.1 Objective Of The Thesis

Pressure injection ahead of the advancing tunnel face (named hear as pre-injection), using microfine cement and chemical grouts, is today a proven technology. As examples can be mentioned about 100 km of pre-injected tunnels below Oslo City and about the same length of subsea tunnelling elsewhere in Norway. Examples from other parts of the world are also available, like Stockholm, Sweden, the Inland Feeder project and Hollywood Metro both in Southern California and Arrowhead Tunnels Project San Bernardino, as well in California. To take advantage of spiling rockbolts and pre-injection, it is an obvious prerequisite to be able to drill the necessary boreholes in the right positions and at the correct angle. In drill and blast excavation this is simple, but in TBM projects it has repeatedly turned out to be difficult. More than once owners have accepted bids containing reassurance from the contractor that the drilling method will be sorted out later. Experience shows that if 'later' means after start of the TBM operation, it is often too late.

The governing economic factor in all modern tunnelling is the speed of tunnel advance. This fact is closely linked to the very high investment in tunnelling equipment, causing high equipment capital cost. Added to this is that the limited working space at the tunnel face will normally allow only one work operation to take place at a time.

The face advance rate depends on the number of hours available for actual excavation works. When one hour of face time has a value of USD 1000 to 2000 it becomes necessary to determine the cost-benefit of all face activities. From this, it can be seen that injection in a tunneling environment is basically different from injection for dam foundations and surface ground treatment. The drive for more efficient methods, materials and equipment has therefore been very strong and substantial progress has been made [1].

2. REASONS FOR GROUTING IN TUNNELLING

2.1 Objectives

Tunnel excavation involves a certain risk of unexpected ground conditions. One of the risks is the chance of hitting large quantities of high pressure ground water. Also smaller volumes of ground water ingress can cause problems in the tunnel, or in the surroundings. Water is the most frequent reason for grouting the rock surrounding tunnels. Ground water ingress can be controlled or handled by drainage, pre-excavation grouting and by post-excavation grouting.

Rock or soil conditions causing stability problems for the tunnel excavation is another possible reason for grouting. Poor and unstable ground can be improved by filling discontinuities with grout material with sufficient strength and adhesion.

2.2 Grouting Types In Tunneling

Pressure grouting in rock is executed by drilling boreholes of suitable diameter, length and direction into the bedrock, placing packers near the borehole opening (or using some other means of providing a pressure tight connection to the borehole), connecting a grout conveying hose or pipe between a pump and the packer and pumping a prepared grout by overpressure into the cracks and joints of the surrounding rock.

In tunnel grouting there are two fundamentally different situations to be aware of:

Pre-excavation grouting, or pre-grouting, where the boreholes are drilled from the tunnel excavation face into virgin rock in front of the face and the grout is pumped in and allowed to set, before advancing the tunnel face through the injected and sealed rock volume. Sometimes, such pre-excavation grouting can be executed from the ground surface, primarily for shallow tunnels with free access to the ground surface area above the tunnel.

Post-excavation grouting, or post-grouting as shown in the Figure 2.1, where the drilling for grout holes and pumping in of the grout material takes place somewhere along the already excavated part of the tunnel. Such locations are mostly selected where unacceptable amounts of water ingress occur.

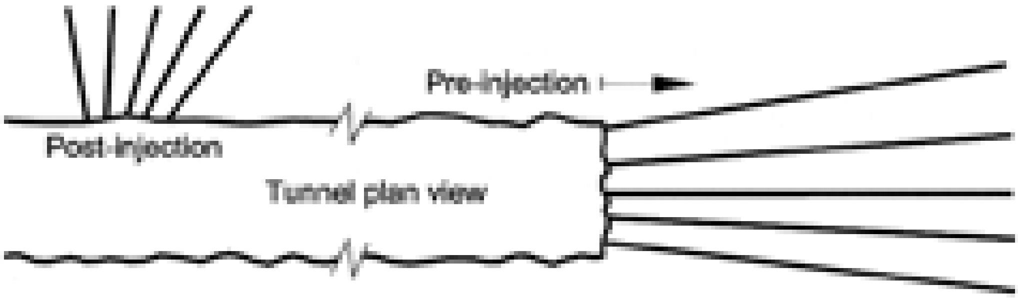


Figure 2.1 : Pre-excavation grouting (injection) and post-grouting (injection).

The purpose of tunnel grouting in a majority of the cases is ground water ingress control. Improvement of ground stability may sometimes be the main purpose, but will more often be a valued secondary effect of grouting for ground water control. Cement based grouts are clearly used more often than any other grout material in tunnel injection, but there are also a number of useful chemical grouts and mineral grouts available. Pressure grouting (injection) into the rock mass surrounding a tunnel is a technique that has existed for more than 50 years and it has developed rapidly during the last 15 years. An important part of developing this technology into a high-efficiency economic procedure has taken place in Scandinavia. Pressure injection has been successfully carried out in a range of rock formations, from weak sedimentary rocks to granitic gneisses and has been used against very high hydrostatic head (up to 500 m water head), as well as in shallow urban tunnels with low water head [2].

The effect of correctly carrying out pre-grouting works ranges from drip free tunnels (less than 1.0 l / min per 100 m tunnel, [3]), to ground water ingress reduction that only takes care of the larger ingress channels. As an example, sub-sea road tunnels in Norway are mostly targeting an ingress rate of about 30 l / min per 100 m tunnel, since this produces a good balance between injection cost and lifetime dewatering cost [4].

3. CEMENT BASED GROUTS

3.1 Traditional Cement Based Grouting Technology

Pressure grouting into rock was initially developed for hydro power dam foundations and for general ground stabilization purposes. For such works there are normally very few practical constraints on the available working area. As a result, grouting was mostly a separate task and could be carried out without affecting or being affected by other site activities.

The traditional cement injection techniques were therefore applicable without too much of disadvantage. The characteristic way of execution was:

Extensive use of Water Pressure Testing (WPT) on short sections of boreholes (3 – 5 m) for mapping of the water conductivity along the length of the hole (Figure 3.1). This process involves carrying out water pressure tests at regular intervals along the borehole to see what the overall water loss situation is, i.e. which sections of the borehole are watertight and which sections allow more or less water to escape. The results were used for decision making regarding cement suspension mix design like water/cement ratio (w/c-ratio by weight) and to choose between using cement or other grouts.

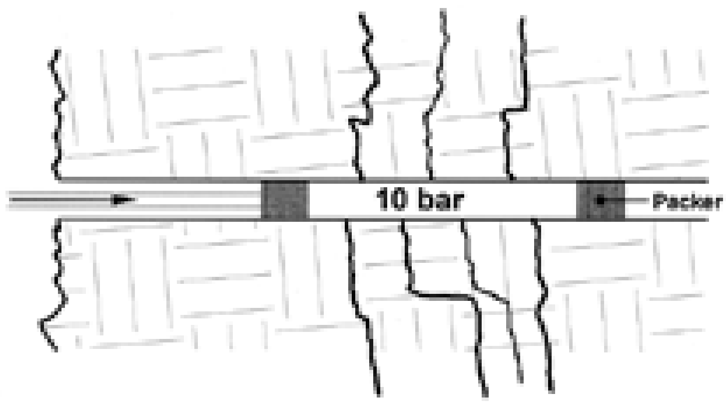


Figure 3.1 : Water pressure testing (WPT).

Use of variable and mostly very high w/c-ratio grouts (up to 4.0) and «grout to refusal» procedures, the latter expression meaning that grout is pumped into the rock until the maximum pre-determined pressure is reached and no more goes in.

Use of Bentonite in the grout to reduce separation (also called bleeding) and to lubricate delivery lines. Bentonite is a special kind of clay with favourable properties in this respect.

Use of stage injection (in terms of depth from surface), low injection pressure and split spacing techniques (new holes drilled in the middle between previous holes). One way of stage injection involves drilling to a certain depth and then injecting the grout for that length. Next, this length gets re-drilled and the hole made longer, followed by a new round of grouting. This process is repeated in steps until full length has been reached. Split spacing as described above is a different way of doing stages injection. Holes drilled to full length may also be stage grouted by moving the packer in stages up the hole, or down the hole using double packer.

The typical overall effect of the above basic approach was that injection operations were quite time consuming. WPT every 5 m; pumping of a lot of water for a given quantity of cement; the need for counter pressure (i.e. grout to refusal or no more take) causing unnecessary spread of grout; holding of constant end pressure over some time (typically 5 – 10 minutes) to compact the grout and squeeze out surplus water; slow strength development and complicated work procedures. The last point is arising from the constant change of w / c-ratio during the pumping to thicken up the grout and reduce unnecessary spread. It all added up to a very long execution time. Due to the very limited maximum grouting pressure being allowed, the efficiency of the individual grouting stages would be limited. This would lead to more drilling of holes and further injection stages to reach a required sealing effect or ground tightness.

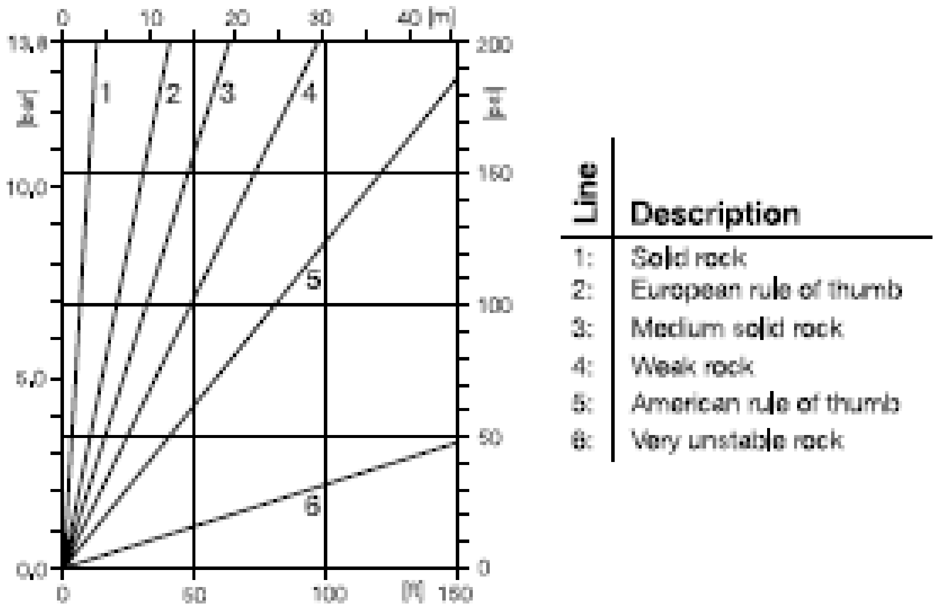


Figure 3.2 : Relation between rock cover and admissible grouting pressure.

From figure 3.2 can be seen that a lot of grouting would be carried out at less than 5 bar pressure.

Summing up; The traditional cement injection technique, as described above and for the reasons given, is rather inefficient when considering the time necessary and the resources spent in reaching a specified sealing effect. Time efficiency is especially important when considering working at a tunnel face and the rock cover and limited free surface area would then normally allow the use of much higher pressure without the same risk of damage.The tunnel environment therefore begs a different approach.

3.2 Basic Properties Of Cement Grouts

3.2.1 Cement particle size, fineness

Any type of cement may be used for injection purposes, but coarse cements with relatively large particle size can only be used to fill larger openings. Two important parameters governing the permeation capability of cement are the maximum particle size and the particle size distribution. The average particle size can be expressed as the specific surface of all cement particles in a given weight of cement. The finer the grinding, the higher is the specific surface, or Blaine value (m^2/kg).

For a given Blaine value, the particle size distribution may vary and the important factor is the maximum particle size, or as often expressed the d95.

The d95 gives the opening size where 95 % of the particles would pass through (are smaller) and conversely, the remaining 5 % of the particle population is larger than this dimension. The maximum particle size should be small, to avoid premature blockage of fine openings, caused by jamming of the coarsest particles and filter creation in narrow spots.

The typical cement types available from most manufacturers without asking for special properties are shown in Table 3.1.

Table 3.1 : Fineness of normal cement types (largest particle size 40 to 150 μm).

Cement type/specific surface	Blaine (m^2/kg)
Low heat cement for massive structures	250
Standart portland cement (CEM I 42.5)	300-350
Rapid hardening portland cement (CEM 52.5)	400-450
Extra fine rapid hardening cement (CEM 52.5)	550

The cements with the highest Blaine value will normally be the most expensive, due to more elaborate grinding process.

Table 3.2 gives examples of particle size of cements commonly used for pressure injection. Please note that the actual figures are only indications based on information from the manufacturers [5]. There will always be some variation when testing depending on the batch.

Table 3.2 : Particle size of some frequently used injection cements.

Cement type	Particle size μm	Blaine
Cementa Anleaggningscement	120 (d_{95}), 128 (d_{100})	300-400
Cementa Anleaggningscement	64 (d_{95}), 128 (d_{100})	600
Cementa Anleaggningscement	30 (d_{95}), 32 (d_{100})	1300
Meyco UGC Rheocem 650	16 (d_{95}), 32 (d_{98})	650
Cementa Ultrafin cement 16	16 (d_{95}), 32 (d_{100})	800-1200
Spinor A16	16 (d_{98})	1200
Dyckerhof Mikrodur P-F	16 (d_{95})	1200
Meyco UGC Rheocem 800	13 (d_{95}), 20 (d_{100})	820
Cementa Ultrafin cement 12	12 (d_{95}), 16 (d_{100})	2200
Meyco UGC Rheocem 900	8 (d_{95}), 10 (d_{98})	875-950
Spinor A12	12 (d_{98})	1500
Dyckerhof Mikrodur P-U	9.5 (d_{95})	1600

From an injection viewpoint, these cements will have the following basic properties: A highly ground cement with small particle size will bind more water than a coarse cement.

The risk of bleeding (water separation) in suspension created from a fine cement is therefore less and a filled opening in the ground will stay more completely filled also after setting.

The finer cements will normally show quicker hydration and a higher final strength. This is normally an advantage, but causes also the disadvantage of shorter open time in the equipment. High temperatures will increase the potential problems of clogging up of lines and valves. The intensive mixing required for fine cements must be closely controlled to avoid heat development caused by friction in the high shear mixer and hence even quicker setting.

The finer cements will mostly give better penetration into fine cracks and openings, but be aware that a small number of maximum size particles may negatively dominate even if the average size is favourable.

The advantage of fine particles will only be realized as long as the mixing process is efficient enough to separate the individual particles and properly wet them with water. In a pure cement and water suspension, there is a tendency of particle flocculation after mixing, especially with finer cements and this is counter-productive. It is commonly said that the finest crack injectable is about 3x the maximum particle size (including the size of flocculates). For standard cements, this means openings down to about 0.3 mm while the finest micro cements may enter openings of 0.06 mm.

The question is sometimes raised, what is the definition of micro cement. Unfortunately, there is no such agreed definition that that has any official international stamp of approval. As an informative indication of minimum requirement to apply the term micro cement, the following suggestion may be used: Cement with a Blaine value > 600 m2 / kg and minimum 99 % having particle size < 40 μm.

The above «definition» fits quite well with the International Society for Rock mechanics reference [6]: "Superfine cement is made of the same materials as ordinary cement. It is characterized by a greater fineness (d95 < 16μm) and an even, steep particle size distribution."

The effect of water reducing admixture (or dispersing admixture) when mixing a micro cement suspension can be seen in Figure 3.3 [7]. It is quite evident that the reduction of d85 by the use of a dispersing admixture from about 9µm to 5µm will strongly influence the penetration of the suspension into the ground. If these figures are put into the soil injection criteria of Mitchell [8], a good injection result with this cement without admixture could be achieved in a soil with d15 > 0.22 mm. With admixture the same result could be obtained in a soil with d15 > 0.12 mm. Also in rock injection the effect of admixture would be significant.

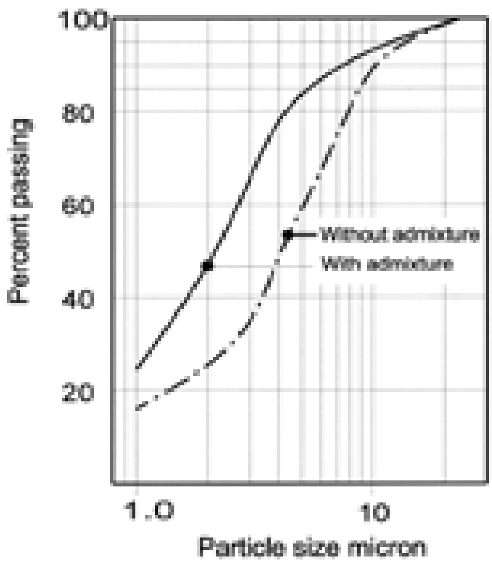


Figure 3.3 : Dispersing effect of admixture when using micro cement [7].

Another important effect of the dispersing admixture is the lowered grout viscosity at any given w/c-ratio. The effect of lower water content is improved final strength of the grout, but more important is lower permeability and better chemical stability. The compressive strength of a pure water and cement mix using a standard OPC is about 90 MPa at w/c-ratio of 0.3 (which will be far too stiff to be used for any normal injection work). Already at w/c-ratio of 0.6 the strength will drop to 35 MPa and when using w/c-ratio above 1.0 the strength is finally in the range of 1.0 MPa or less. (RHEOCEM® 650 with 1.5 % admixture and w/c-ratio of 1.0 reaches 10.0 MPa compressive strength after 28 days). More important in cases with even higher water content is that the permeability is pretty high and the strength is so low that if any water flow takes place, it can lead to chemical leaching out of hydroxides (hydration products from cement reacting with water) and eventually mechanical erosion.

3.2.2 Cement-Bentonite Grouting

A bentonite grout backfill consisting of just bentonite and water may not be volumetrically stable and introduces uncertainty about locally introduced pore water pressures caused by the hydration process. Introducing cement, even a small amount, reduces the expansive properties of the bentonite component once the cement-bentonite grout takes an initial set. The strength of the set grout can be designed to be similar to the surrounding ground by controlling the cement content and adjusting the mix proportions. Controlling the compressibility (modulus) and the permeability is not so easy. Weaker cementitious grouts tend to remain much stiffer than normally consolidated clays of similar strengths. The bentonite solids content has the greatest influence on the permeability of cement-bentonite grout, not the cement content.

Cement-bentonite grouts are easier to use than bentonite grouts, provide a long working time before set and are more forgiving should the user deviate from the design recipe or mixing equipment and method. It is easier to adjust the grout mix for variations in temperature, pH and cleanliness of the water.

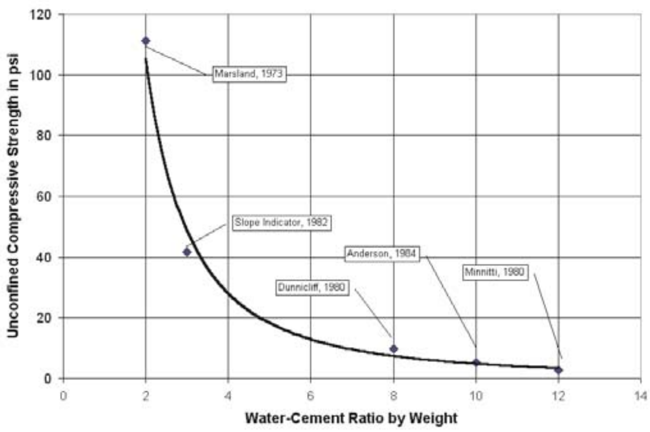


Figure 3.4 : 28-day cement-bentonite grout strength vs. water-cement ratio[9]

The general rule for grouting any kind of instrument in a borehole is to mimic the strength and deformation characteristics of the surrounding soil rather than the permeability. However, while it is feasible to match strengths, it is unfeasible with the same mix design to match the deformation modulus of cement- bentonite to that of a clay for example. The practical thing to do is to approximate the strength and minimize the area of the grouted annulus. In this way the grout column would only contribute a weak force in the situation where it might be an issue. [9]

Strength data collected informally from various sources by the author over the years are summarized in Figure 3.4 trend line drawn through the data points illustrates the decrease in strength with increasing water-cement ratio. The water- cement ratio controls the strength of the set grout (Marsland, 1973). Marsland's rule-of-thumb is to make the 7-day strength of the grout to match one quarter that of the surrounding soil. Water and cement in proportions greater than about 0.7 to1.0 by weight will segregate without the addition of bentonite or some other type of filler material (clay or lime) to suspend the cement uniformly. In all cases sufficient filler is added to suspend the cement and to provide a thick creamy-but-pumpable grout consistency.

3.2.3 Rheological behaviour of cement grouts

Cement mixed in water is an unstable suspension or a stable paste (in terms of water separation) and behaves according to Bingham's Law. Water and true liquids have flow behaviour according to Newton's Law. These laws are as follows (see Figure 3.5):

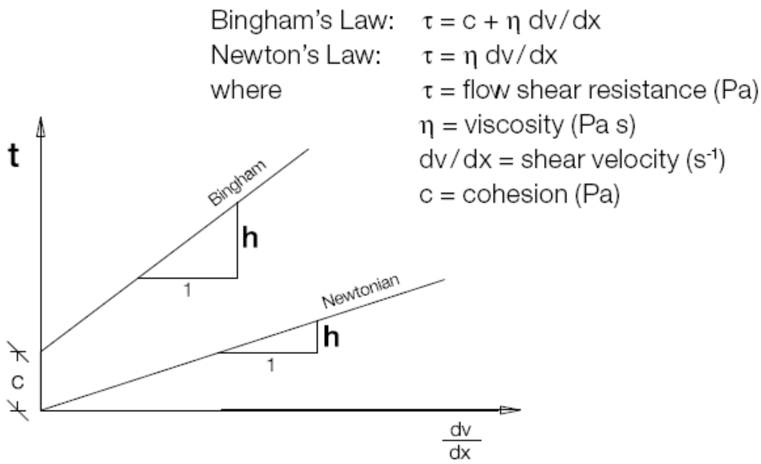


Figure 3.5 : Rheological behavior of Newton and Bingham fluids.

When a stable grout has a very low w / c-ratio, or when ground mineral powder or fine sand has been added, the grout may also have an internal friction. To cover this property, Lombardi has proposed the following rheological formula [10]:

$$\tau = c + \eta dv / dx + p \tan \phi \qquad \textbf{(3.1)}$$

Where p=Internal pressure within grout, ϕ = angle of internal friction of the grout.

A true liquid will flow as soon as there is a force creating a shear stress.Water in a pipe will start flowing as soon as there is an inclination. Liquids that have higher viscosity than water will also flow but at a lower velocity.

A cement suspension or a paste, will demonstrate some cohesion. The difference to liquids is that the cohesion has to be overcome for any flow to be initiated. If the internal friction is negligible, the paste will thereafter behave in a similar manner as a liquid with the same viscosity. The rheological parameters of cement suspensions can be influenced by w / c-ratio, by chemical admixtures, by Bentonite clay and by other mineral fillers. As an example, it is possible and often useful, to create a grout with a high degree of thixotropy. This means a paste with low total flow resistance while being stirred or pumped, but shortly after being left undisturbed, it shows a very high cohesion.

3.2.4 Pressure grouting

Pressure grouting consist of material (grout) under pressure,so as to fill joints and other defects in rock, soil concrete, masonry and similar materials. It can also modify soil through the filling of pore space or compaction into a denser state.In addition grouting is used to fill and bond cracks and defects in structural concrete and masonry.

In new construction, it is employed to ensure complete filling under precast members, base plates and similar assemblies. It can be used to cast concrete in place, wherein a cementitous grout is pumped into previously placed large aggregate (preplaced aggregate concrete). Severeal specialty geotechnical techniques use preesure grouting as a pricipal component.

These involve a wide spectrum of operations, ranging from the grouting of foundation piles to encapsulation or immobilization of contaminated soil through jet grouting or deep soil mixing with grout.

The installation of soil anchors, rock bolts and foundation piles can be facilitated and their capacity significantly increased, with supplementary grouting.

The main ingredient of a grout may be cementitious material, a liquid or solid chemical including hot bitumen, or anyone of a number of different resins. A combination of two or more components often used. A wide variety of different filler materials may be included.

Grout can have virtually any consistency, ranging from a true fluid to a very stiff mortarlike state. It will generally harden at some point after injection, so as to be come immobile, and can be designed to have a wide variety of both bond and compressive streng. Grouts are available that will cure into a solid, a flexible gel or a light weight foam.

The application of grout and the use of grouting methods have proven to be advantages in many andevours. For the beneficial modification of geo-materials they are now routinely used to achieve any of the following:

Block the flow of water and reduce seepage

Strengthen soil, rock, or combinations thereof

Fill massive voids and sink holes in soil or rock

Correct settlement damage to structures

Form bearing piles

Support soil and create secant pile walls

Install and increase the capacity of anchors and tie-backs

Immobilize hazardous materials and fluids

As it is apparent, the beneficial applications of pressure grouting are no longer limited to the control of water seepage, but now facilitate strengthening and improving virtually all geo and other materials. They are also useful, and in some cases mandatory, for the repair and rehabilitation of concrete and masonry structures. Although the reasons for grouting, as well as the materials and procedures available, are many, the principles of injection are the same. [11]

3.2.5 Setting of cement grouts

When cement and water are mixed, chemical reactions start immediately and continue fairly actively for the nex few hours and then with increasingly less activity for over a year.

The reactions should eventually produce setting and hardness. Setting can be thought of in terms of (a) initail set, where the grout firms up, but is not really hard, and (b) final set where it becomes hard.

The minimum w:c ratio able to provide the quantity of water needed for the cement to fully react ("hydrate") is around 0,2:1 by weight. This is 0.3:1 by volume and is too thick to be a practical grout. The thickest mix able to be handled by the best mixers is 0,5:1 and this particular grout has limited application. All the practical grouts have much more water than is needed merely fort he chemical action. This excess water slows down the setting.

For instance, compare the ASTM Type I specified initial set time of 0,75 hours and final set of 10 hours at paste consistency. Even 3:1, a grout recommended as the thinnest desirable for durability can take 16 hours for initial set. Firm set takes much longer. This is one of the reasons for using the thickest grout practicable.

In the case of the thinner grouts, setting time is critical if the grouting is being done in a foundation where groundwater is flowing. These grouts do not have the benefit of thixotropic stiffening and can be washed away if set is too slow. This has happened at at least one dam where 8:1 grout was unwisely used witout realization of its slow set, all of the carefully placed grout was washed away in a matter of days. Sadly, this was not recognized until a great deal of grouting has been done.

However, it is not only those flowing water conditions that can be troublesome if grout is slow to set. A more common situation is disturbance from water originating from nearby drilling, water testing, and grouting operations, such water is liable to remove or damage grout that is stil sof tor perhaps even stil fluid. [12].

3.2.6 Durability of cement injection in rock

There is a large volume of hard rock tunneling with extensive use of pre-injection as part of the tunnel design and as the sole measure of permanent ground water control. The primary experience basis is probably in Scandinavia, where Norway alone has close to 100 km of subsea tunneling with pre-grouting. Even though some of these tunnels go down to as much as 260 m below sea level and the grout injection works carried out are of a permanent nature, there is no report indicating that grout has degraded.

The Norwegian Public Roads Administration operates 20 sub-sea tunnels of various different ages (the oldest tunnel goes to Vardoe island and was commissioned in 1981) excavated through quite variable ground conditions. In fact, the general trend reported is a slow reduction of water ingress over the years. If any such degradation of grout would be a problem, one would expect an increase in water ingress levels. Melby [13] presents a paper dealing with 17 different projects totaling 58.6 km of tunneling. A comparison of water ingress at the time of opening and measurements made in 1996 shows the average in 1996 to be only 62.9 % of the ingress recorded when the tunnels opened. None of these tunnels showed an increase in the leakage rate.

The Norwegian national oil company Statoil constructed three pipeline sub-sea tunnels amounting to a total of 12 km, going down to 180 m below sea level. The tunnels are crossing Karmsund, Foerdesfjord and Foerlandsfjord. During more than 15 years of operation Statoil has recorded the energy-consumption expended in pumping of ingress water from the deepest point in the tunnels to sea level discharge. Statoil states that there has been no increase in ground water ingress, since the energy consumed for pumping has not increased [14].

Very important for the quality and durability of cement grouts is the w / cratio and whether the grout is stable or segregating (bleeding). Modern grouting technology in tunneling means stable grouts and thus also w/c-ratio below a certain limit, depending on the type of cement and the admixture used. This view is supported by the ISRM, Commission on Rock Grouting, Final Report, which states in Chapter 4.2.6 [6]: «Stable or almost stable suspensions contain far less excess water than unstable ones.

Hence, grouts with a low water content offer the following advantages:

During grouting:

- higher density, hence better removal of joint water and less mixing at the grouting front
- almost complete filling of joints, including branches
- the reach and the volume of grout can be closely delineated
- grouting time is shortened because little excess water has to be expelled
- the risk is reduced that expelled water will damage the partially set grout

After hardening:

- greater strength
- lower permeability
- better adhesion to joint walls
- better durability.

3.3 Accelerators For Cement Injection

It is frequently claimed that there is no need for relatively fast setting cement for rock injection, because it is possible to use an accelerator to compensate for slow cements, when needed. This is partly correct, but when considering the use of microfine cement in combination with quick setting to speed up tunneling, then the picture is different.

The main point is that accelerators will cause flocculation of the cement particles when added to the grout. In the case of a micro cement, this is defeating the purpose of paying for and using a microfine cement. Furthermore, in most cases part of the injection will be carried out without accelerator and the accelerator is only added for special local purposes. The downside of this is that at the end of injection, there will be some grout setting fast, but most of the volume is slow and this volume will be dimensioning for the waiting time.

Cements mentioned in table 3.1. are fast setting without any added accelerator, thus overcoming the above described problems. Even when using a quick setting grout, there are situations where accelerated setting can be necessary. This will typically be in post-grouting cases for backflow cut-off, but also in pre-injection, backflow may happen through the face. If for any reason the grout is pumped into running water, or pressure or channel sizes are extreme, accelerated grout may become necessary.

The best option in combination with micro cements is alkali free accelerators used for sprayed concrete This products seems to not create flocculation before setting is initiated and then it sets pretty fast. The standard products can be diluted by adding 50 % water before using it with the grout. Normal dosage (calculated on weight of un-diluted product) will be in the range of 0.1 to about 3.0 % by weight of cement. Low dosages can be added to the grout in the agitator (but it is not recommended), while higher dosages must come through a separate hose to the packer head. A non-return valve is needed for use with a dosage pump for alkali free accelerators through a separate hose to the packer head. Setting time of cement grout with inreasing accelerator dosage is shown in the figure 3.6.

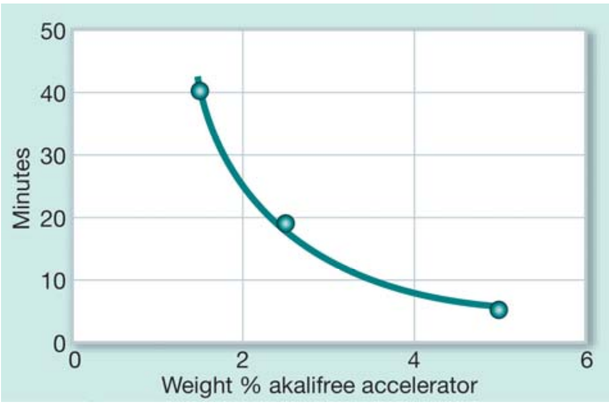


Figure 3.6 : Setting time of cement grout with increasing accelerator dosage.

Practical experience has shown that this system works very well and can compete with other alternatives for backflow cut-off (like quickfoaming polyurethane), mostly even without loosing the borehole for further injection without accelerator. It should be noted that this very practical and efficient solution may not work very well with slower cements and blended cements.

4. CHEMICAL GROUTS

Chemical grouts consist of liquid-only components which lead to quite different behaviour compared to cementitious grouts. Chemical grouts are Newtonian fluids, demonstrating viscosity but no cohesion (see Figure 3.5). Therefore the penetration distance from a borehole and the placement time for a given volume only depend on the viscosity of the liquid grout and the injection pressure used. Chemical grouts available include silicates, phenolic resins, lignosulphonates, acrylamide and acrylates, sodium carbomethylcellulose, amino resins, epoxy, polyurethane and some other exotic materials.

For practical purposes there are two main groups of chemical grouts available:

1. Reactive plastic resins

2. Water-rich gels

The reactive resins may be monomers or polymers that are mixed to create a reaction (polymerization) to stable three-dimensional polymeric end product. When short reaction times are used, normally such products are injected as two-component materials, mixing taking place at the injection packer upon entry into the ground. At longer reaction times even two-component materials can be injected by a one-component pump. Mixed batches only have to be small enough and with long enough open time to be injected before the polymerization reaction takes place. Such products will not be dissolved in water, but they may react with water. For proper reaction and quality of the end product the right proportioning of the components is important. Two-component pumps must function properly at all times for this requirement to be satisfied.

The gel forming products are dissolved in water in low concentrations and the liquid components therefore show a very low viscosity (often almost as fluid as water). When the polymerization takes place, an open three dimensional molecular grid is created, which binds a lot of water in the gel. The water is not chemically linked to the polymeric grid, but is locked within the grid by adsorption.

4.1 Polyurethane Grouts

Among the chemical solution grouts employed for water control, those based on polyurethane chemistry are by far the most extensively used, especially in the correction of leakage in to structures. These are highly versatile compositions are of many different types and supplied in various forms. They can provide a wide range of properties in both the fluid and hardened states. Some are furnished as single solutions that need only be combined with water, where as others consists of two separate solutions that are premixed, the reaction being completed upon exposure to moisture. Different formulations will react to form gels, solids or foams.

The strength of the gel, or density of the foam is dependent on the particular formulation and the amount of water with which it has combined. As with most compositions involving chemical reaction, temperature has a significant influence on both the viscosity of the individual components and the mixed grout. Time to gelling and /or curing of the mixed resins is also sensitive to temperature. Urethane grouts are generally very stable once in place. They provide high resistance to acids, but only fair resistance to strong alkalis, and are subject to degradation upon exposure to ultraviolet light. Some formulations react completely upon coming in contact with moisture, but others require thorough mixing with a certain minimum amount of water. Single-component urethanes are prepolymers; that is, initial polymerization has already taken place, but the process can not be completed without their coming in contact with moisture. The amount of water required for complete polymerization in the required time frame will vary and is dependent on the individual formulation. Because the stability and quality of the reacted material are totally dependent on the inclusion of a sufficient amount of water, knowledge of that required, and positive assurance that is provided, are crucial. Many cases of less – than –acceptable grout performance have been attributed to lack of sufficient moisture. Urethane grouts can be divided into two main classes, hydrophobic and hydrophilic formulations react with water, but only a very small amount is required for their proper cure. Once that amount has been combined, their repel further moisture to which they are exposed.

Because of a limited amount of water is allowed to contact the polymer during the reaction, gas is trapped in the polymerized matrix, which develops as a foam in an unrestrained form.

Hydrophobic formulations tend to produce relatively rigid foams and are available in a wide variation of densities. The actual density results from the particular chemical formulation and, of course, exposure to sufficient moisture. To enhance wetting of contacted surfaces and stimulate the water-grout reaction, surfactants are frequently used, resulting in an increased chemical bond.

Figure 4.1 : Typical volume expansion of PU grouts

Because they repel excess water, the ability of hydrophobic formulations to bond to wet surfaces is not great; however, this is not a limitation in soil pore space or random cracks with rough surfaces. It can become a problem in generally smooth walled joints in rck or concrete or in a void in soil immediately adjacent to a smooth interface. Some hydrophobic formulations will immediately (within 15-20 seconds) foam on contact with water as shown in figure 4.1. This can be beneficial in stopping flows of moving water. Because reacted hydrophobic urethanes take in little outside water in cured state, they are generally free of shrinkage, even when allowed to dry. They can also be formulated to be of very low viscosity, which allows penetration in to the finest cracks and defects and in to virtually any permeable soil. Even though initial contact with water is required for reaction, once it is started, the grout will be resistant to further dilution. Hydrophobic resins can be formulated to have a very high expansion potential, and grouts with unrestrained expansion of nearly 30 times their original volume have been reported. Although the expansion within a fine fissure or soil pore space will be considerably reduced because of frictional constrained, a high expansion rate will lower the cost of material for a given amount of work.

Hydrophilic formulations also react with water, but many continue to attract it after completion of the initial reaction. These formulations often continue to expand if possible, forcing the resulting gel into pore spaces and voids beyond those originally filled. Because they attract moisture, they tend to be drawn in to water-filled pore space and micro cracks on the surfaces of the filled defects. In addition they develop a much grater bond to those surfaces as long as they are not excessively diluted. The quality of the cured resin is totally dependent on the amount of water it contains, so it is important to supply only that which is required for the desired gel or foam. These formulations are thus best avoided for use in areas that are likely to dry periodically. Because some hydrophilic formulations require a minimum amount of water to react fully, good practice dictates that the grout is stream-mixed with water during injection with the use of an appropriate dual pump proportioning system as shown in the figure 4.2. [11].

4.1.1 Pumping equipment

For two-component PU products it is necessary to use a custom design two-component PU pump. These are normally prepared for 1:1 ratio of the A and B components (by volume). One of the most popular pumps available for under ground use can be seen in figure 4.2.

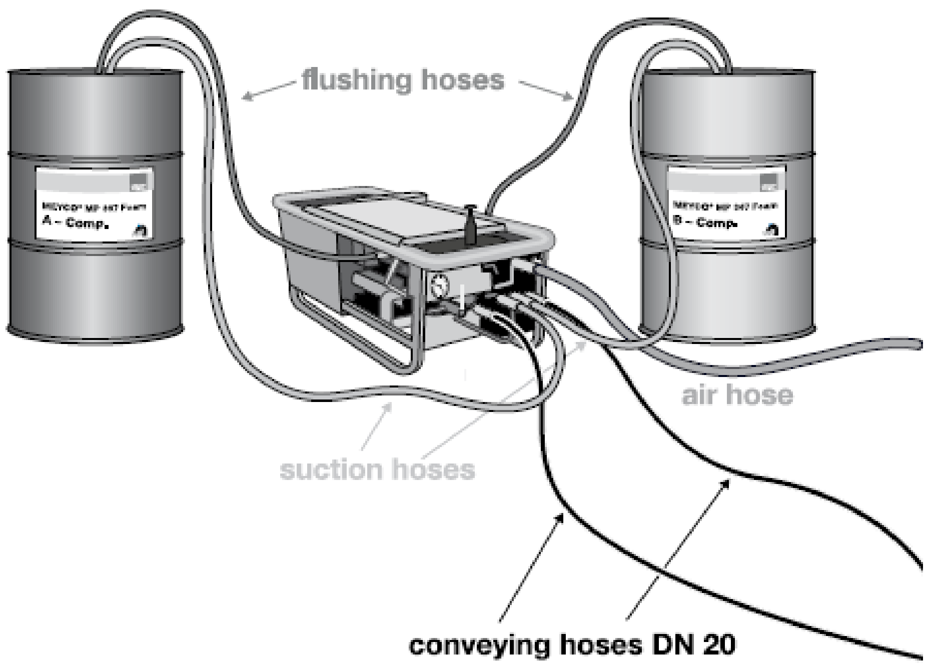


Figure 4.2 : Air-driven 2-component PU pump.

4.2 Silicate Grouts

Sodium silicate has been used for decades, mostly in soil injection. There are also examples of silicate injection in rock formations. The main advantage of silicate grouts is the low viscosity and the low cost. It may also be added that apart from the pH of typically 10.5 to 11.5 (causing it to be quite aggressive) there are small problems with working safety and health. On the other hand, this grout is questionable in relation to the environment, since the gel is generally unstable and will dissolve over time.

The main characteristics of a silicate grout in its pregel state are the following:

- Density is linked to the silicate compositions and relative amount.
- Initial viscosity depends mainly on the silicate Rp and concentration
- Evolutive viscosity changes until gel point and strongly influences injection time
- Setting time (gel point) is defined when the grout becomes hard enough that it can not be poured. Setting time depends on the quality or quantity of reagent and varies inversely with temperature. It can vary from a few to 120 minute and clearly influences the period of injectability.

In its hardened state, the main characteristics are the following:

- Mechanical strength is rarely measured on the gels because of its irrelevance but rather measured on permeated soil samples. It varies with reagent and silicate concentrations, chemistries, and degree of neutralization.
- Syneresis is the expulsion of water (usually alkaline) from the gel, a companied by gel contractions. This may continue for up to 40 days after gel setting. The extent varies with the nature and concentration of the components and on the granulometry and PH of the soil (progressively less in finer soils).
- Resistance to washout, along with gel dissolution, depends on silicate concentration. [15].

4.3 Colloidal Silica Grouts

This product bears no resemblance to the chemical silicate systems described above. The colloidal silica is a unique new system with entirely new properties and can be considered safer and more environment friendly than even cement. Colloidal silica is a water suspension of nanometric silica particles.

The water dispersion contains discrete, non-aggregated spherical particles of 100 % amorphous silicon dioxide in a 30 % solids suspension. Table 4.1 give an idea about the particle size, remember the frequently made statement that silica fume particles are like cigarette smoke (Figure 4.3).

Table 4.1 : Comparison of particle size for silica products.

Product	Particle size µm	Blaine (m^2/g)
Colloidal silica	0.016	80-900
Silica fume	0.2	15-25
Precipitated silica	5	10-15
Cystalline silica (mesh 200)	15	0.4

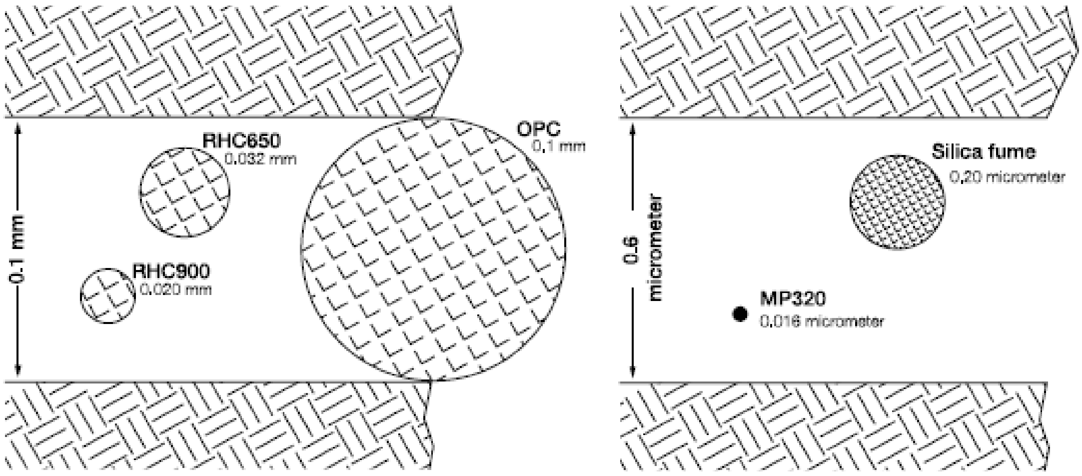


Figure 4.3 : Injection products particle size.

4.4 Acrylic Grouts

The acrylic grouts came in use already 50 years ago and for cost reasons the first products were based on acrylamide. The toxic properties of such products have over the years stopped them from being used any more. The last known major application was in the Swedish Hallandsasen railway tunnel, where run-off to the ground water caused pollution downstream and poisoning of livestock. However, it is not necessary to include this dangerous component (acrylamid) in an acrylic chemical grout.

Polyacrylates are gels formed in a polymerization reaction after mixing acrylic monomers with an accelerator in aqueous solution. In the construction industry, acrylic grouts are used for soil stabilization and water proofing in rock. Polymerized polyacrylates are not dangerous to human health or the environment. In contrast to that, the primary substances (monomers) of certain products can be of ecological relevance before their complete polymerization. Injection materials polymerize very quickly – as a rule, within some minutes. Before the monomers completely polymerize, a considerable amount can be diluted by the ground water (especially if water is actually flowing), subsequently leading to contamination.

Because of such effects in practical injection works under ground and because of the working safety of personnel, the use of products containing acrylamide (which is a nerve poison, is carcinogenic and with cumulative effect in the human body) must be completely rejected.

Products are available that are based on methacrylic acid esters, using accelerator of alconal amines and catalyst of ammonium persulphate. These products are in the same class as cement regarding working safety and can be used under ground, provided normal precautions are taken. Acrylic gel materials are very useful for injection into soil and rock with predominantly fine cracks due to the low viscosity (Figure 4.3). Normally such products is injected with less than 20 % monomer concentration in water and the product viscosity is therefore as low as 4 to 5 cP. This viscosity is kept unchanged until just before polymerization, which then happens very fast. This is very favorable behavior under most conditions. The gel-time can typically be chosen between seconds and up to an hour.

When using acrylic grouts, be aware of the exothermic reaction and itspractical effects. Testing the gel-time in a small cup (like an espressocup), the gel-time will be substantially longer than the same test carried out in a double size coffee mug. This happens because of self-acceleration driven by the heat development (10o C higher temperature will cut gel-time by about 50 %). This is important when working with such products, because slow penetration through large diameter boreholes may cause premature gelling.

Figure 4.4 : Acrylic grout injected in sand.

The strength of the gel will primarily depend on the concentration of monomer dissolved in water, but also which catalyst system and catalyst dosage that is being used. The gel will normally be elastic like a weak rubber with compressive strength of about 10 kPa at low deformation. Injected sand can reach compressive strength of 10 MPa.

If a gel sample is left in the open for some time under normal room conditions, it will loose the adsorbed water trapped within the polymer grid, shrink and become hard and rigid. If placed in water again, it will swell and regain its original properties. In underground conditions this property of an acrylic gel will seldom represent any problem, but be aware that if an unlimited number of drying / wetting cycles must be assumed, the gel will eventually disintegrate. The chemical stability and durability of acrylic gels are otherwise very good and can be considered satisfactory for permanent solutions.

4.5 Epoxy Resins

Epoxy products can have some interesting technical properties in special cases, but the cost of epoxy and the difficult handling and application are the reasons for very limited use in rock injection under ground. Epoxy resin and hardener must be mixed in exactly the right proportions for a complete polymerization to take place. Any deviation will reduce the quality of the product. The reaction is strongly exothermic and if openings are filled that are too large (width > a cm or so) the epoxy material will start boiling and again quality will be reduced. Epoxy viscosity is relatively high, unless special solvents are used.

Working safety and environmental risk are additional aspects of epoxy injection that makes the product group of marginal interest for underground rock injection.

4.6 Urea-silicates

Lately the mining industry has started to use a combined system of urea-silicate instead of traditional polyurethane grouts. The product will be handled as a two component material, where one component is a special silicate and the other component a special developed pre-polymer MDI-system. The two individual components are normally mixed at a ratio of 1:1. Urea-silicate has a lot of advantages compared to traditional polyurethane grouts such as a high fire resistance low flammability and an extremely fast curing.

The system has a very low reaction temperature, typically below 100°C and clearly below normal polyurethane systems with reaction temperatures between 150° – 180°C. The system can be designed to react with or without foaming and react even with or without being in contact with water. These systems are used for cavity filling, stabilizing of fractured rock, sands and soil and for repair of underwater structures.

5. DIFFERENT GROUTING APPLICATIONS IN VARIOUS TBM PROJECTS

5.1 The Oslo Sewage Tunnel System, Norway

The tunnel system consists of about 40 km of sewage transport tunnels and an underground sewage treatment plant. Construction was undertaken around 1980. The tunnels were constructed by TBM to avoid the vibration problems when passing below an urban area. Pre-injection was mandatory because a major part of buildings and infrastructure are founded on marine clay. Even a minor lowering of the pore pressure in the clay basins would cause settlements up to several hundred metres away from the tunnel alignment.

The first contract let was based on a 3.5 m diameter hard rock Robbins TBM. The contractor stated that the probe drilling and injection drilling would be solved 'later'. The method finally adopted was very poor. The TBM was backed up a couple of metres, people and equipment were passed through the cutterhead and drilling was carried out by manually operated pneumatic jackleg drills. This system was grossly unsatisfactory and caused the owner to reject all bids for subsequent project sections, if the detailed pre-injection drilling solution was not included and acceptable.

The largest single tunnelling contract covered 14.2 km of 3.5 m diameter TBM tunnel and a 900 m drill and blast access tunnel at Holmen. The two Robbins TBMs were manufactured to accommodate two hydraulic booms with Montabert H 70 hydraulic borehammers. The feeder length was 10 feet (3 m).

During planning of the works, it was found that all components had to be adapted to work with each other. This included the TBM. The final outcome provided 17 locations around the periphery where boreholes could be started. From each location drilling could be carried out in variable direction. The starting points were 3 m behind the face.

The equipment layout can be seen in Figures 5.1; 5.2; 5.3. The normal borehole angle relative to the tunnel contour was 4°. By drilling 27 m boreholes, losing 3 m between the starting point and the actual face and by 4 m overlap to the next drilling, a net length of 20 m tunnel was pre-injected per round (Figure 5.4).

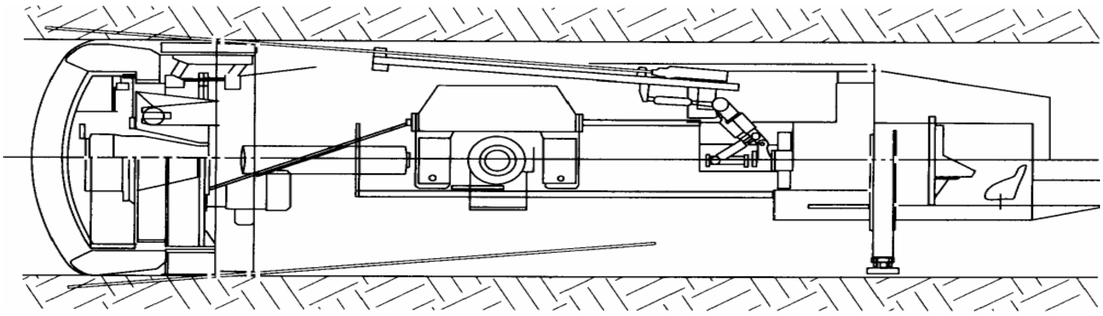


Figure 5.1 : Custom design drilling booms.

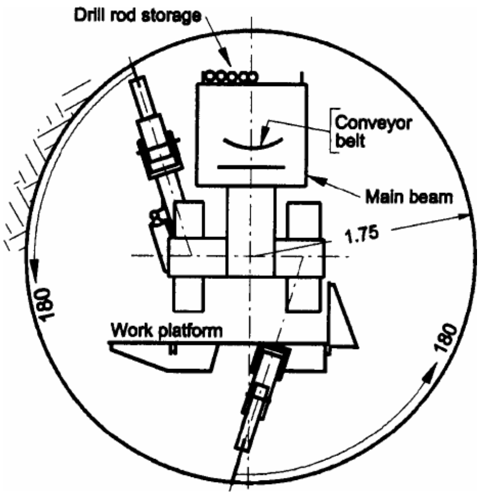


Figure 5.2 : Drilling boom movement range.

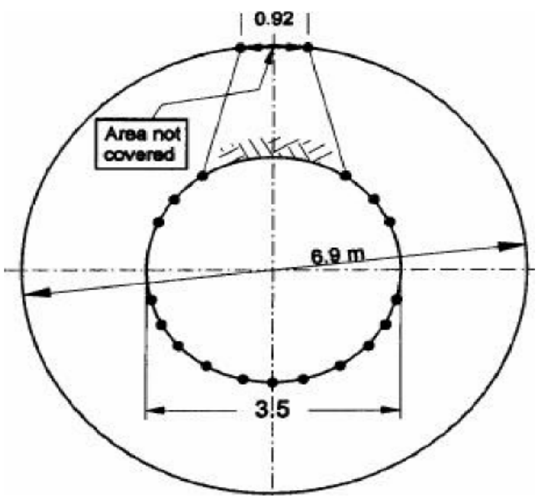


Figure 5.3 : Borehole starting position.

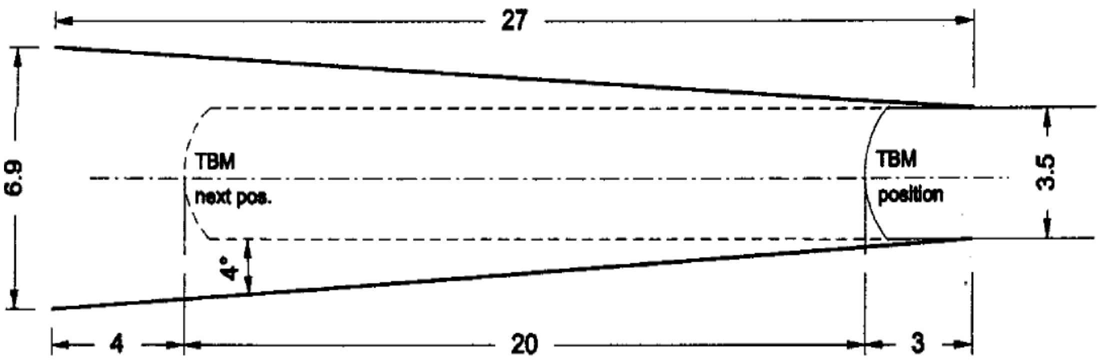


Figure 5.4 : Plan view with net drilling length.

The grouting equipment consisted of colloidal mixer, agitator and grout pump from Montanburo of Germany. Standard units were modified to fit into the back-up system about 25 m behind the face. Maximum pump output was 50 1/mm. and maximum pressure was 50 bar.

Work in the tunnel was organized in two shifts per day; each shift was 7.5 hours. The routine drilling of 4 ca 27 m long holes normally required three to four hours, including setup and clearing away. The injection time was highly variable depending on the quantities injected. It is therefore no surprise that weekly face progress also varies accordingly.

The overall average results are given in Table 5.1.

The total cost of pre-injection, including drilling ahead, was 38 per cent of the total Contractor's cost per metre of tunnel [16].

Table 5.1 : Average figures for both tunnel headings.

	Heading Bjerkas	Heading Sandvika
Tunnel length (m)	7211	7067
TBM net penetration (m/machine hour)	3.72	3.69
Boreholes per metre of tunnel	6.95	6.48
Cement consumption (kg/m)	121.3	54.1
Chemical grout consumption (I/m)	6.3	5.9

5.2 Limerick main drainage water tunnels, Ireland

The Limerick Main Drainage scheme is a major environmental and infrastructure project which provides a new drainage system and treatment facility for the City and its surrounding area. The new system is linked to a state-of-the-art waste water treatment plant, there by eliminating untreated discharges to the rivers.

Murphy Tunnelling were the Contractor for the project, involving the construction of

2.55km of 2.82m ID Lovat EPB TBM driven, segmentally lined tunnel from Corcanree Business Park to Harvey's Quay in the City Centre. Thirteen segmentally lined shafts sunk to depths of 15m provide both access and connection points for existing sewers.

Ground conditions along the route of the tunnel are complex and difficult ranging from very hard limestone bedrock to over laying very soft alluvial and glacial deposits. The close proximity to the River Shannon and the high water table added to the construction problems. For one of the shafts, ground stabilisation of water bearing fine sands was required to allow safe break-in and break-out of the TBM.

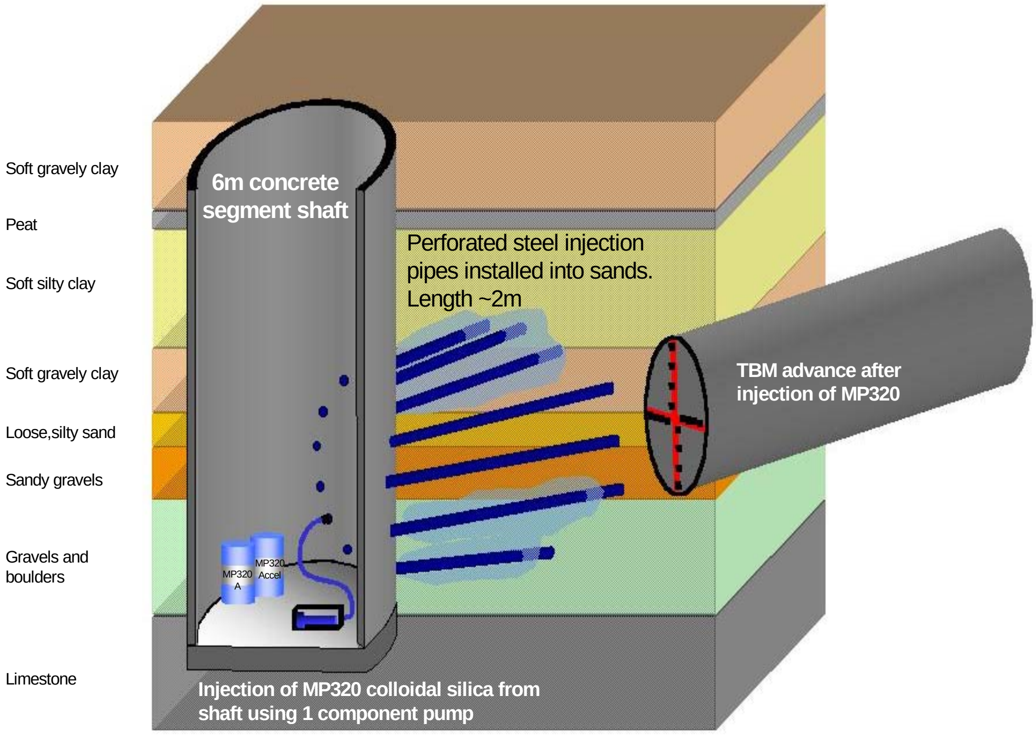


Figure 5.5 : Fine sand stabilization from shaft to permit safe entry of EPB TBM.

To facilitate injection, a series of one and two metre long perforated steel pipes as shown ın the Figure 5.6, with one-way valves fabricated on site were rammed into the sands through pre-drilled holes in the shaft segment lining at positions marked on the outer circumference of the proposed TBM breakthrough.

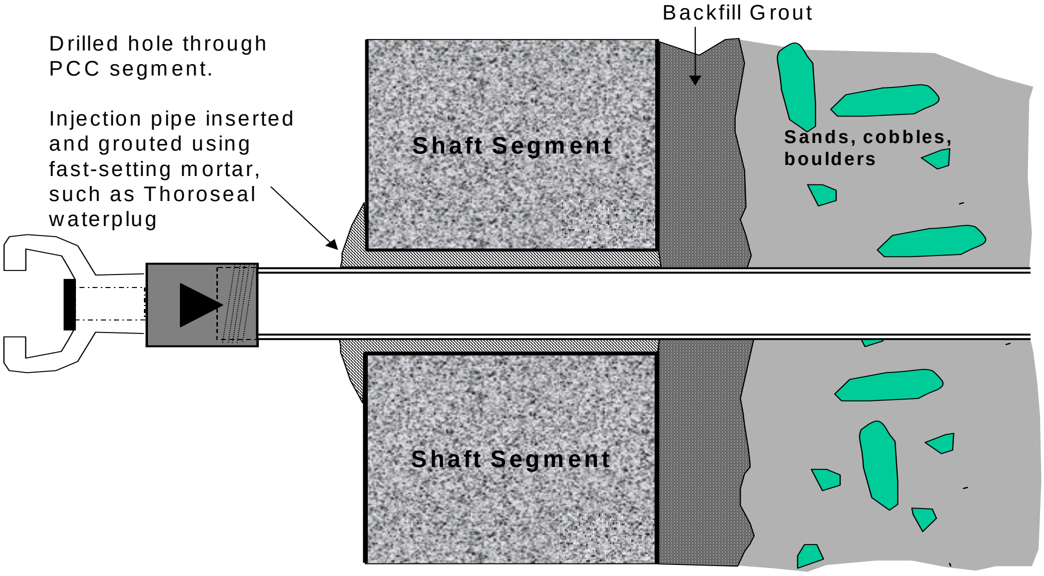


Figure 5.6 : Fabricated on-site steel injection tubes.

After removal of the shaft segments to allow the safe breakthrough and breakout, the sands were seen to be effectively treated and stable with the colloidal silica as indicated in the Figure 5.5, and offered no resistance with the TBM excavation.

5.3 The Abdalajis Project, Spain

The Abdalajis Tunnel East is one of the two 7,1 km long railways tunnels part of the new high speed railway line from Cordoba to Malaga and its alignment is shown in Figure 5.7 .

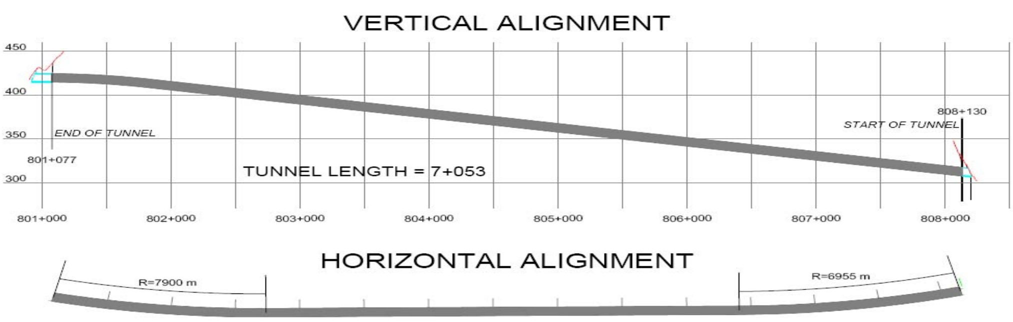


Figure 5.7 : Tunnel alignment.

The excavation diameter is 10 meter and the lining is the tapered ring type made of 45 cm thick precast segments (Figure 5.8) with a final internal diameter of 8,80 m. The adjacent segments are connected with bolts and the adjacent rings with plastic connectors. An EPDM profile is used to seal the joints between segments. The gap between the segments and the rock surface is filled with pea-gravel and in a second stage the pea-gravel is grouted with a cement based mixture.

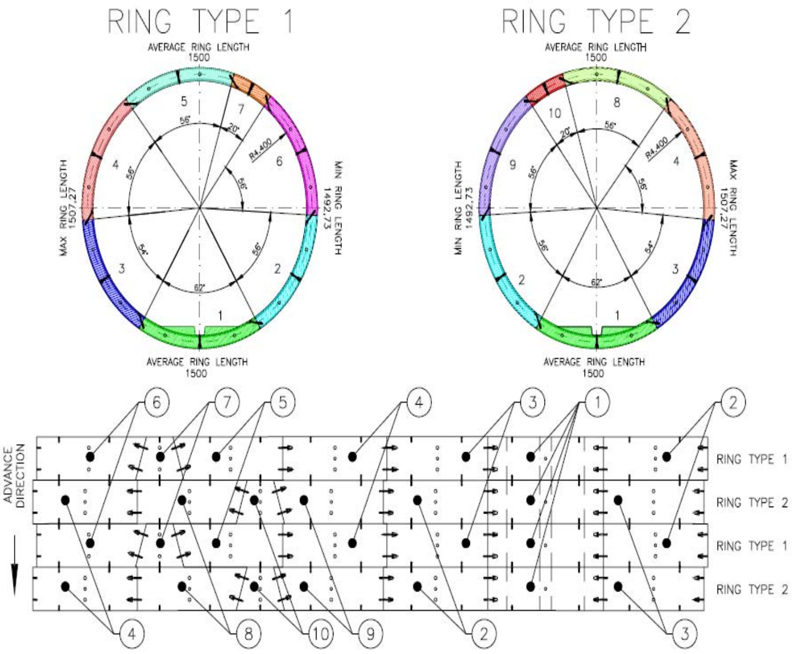


Figure 5.8 : Typical tunnel section of abdalajis project.

The Abdalajis tunnel geology can be divided in two sections:

• the first section (Figure 5.9) in weak and very weak formations having a total length of about 2 km

• the second section in more competent sedimentary rocks under high overburden

Grouting was related mainly with the excavation of the first section, characterised by the following formations:

• phillade and quartzite - Formacion Tonosa

• slate and sandstone –Formation Morales

• conglomerate –Formacion Almogia

• argillite - Arcillas Variegadas

The rock formations in this section of the tunnel were expected to be of very low geo-mechanical characteristics, and were considered by many technicians an impossible task for a TBM, due to the high and rapid convergence and the potential face instability phenomena.

At the same time high pressure water flows were expected in the fractured limestone formations to be encountered after the argillite formations.

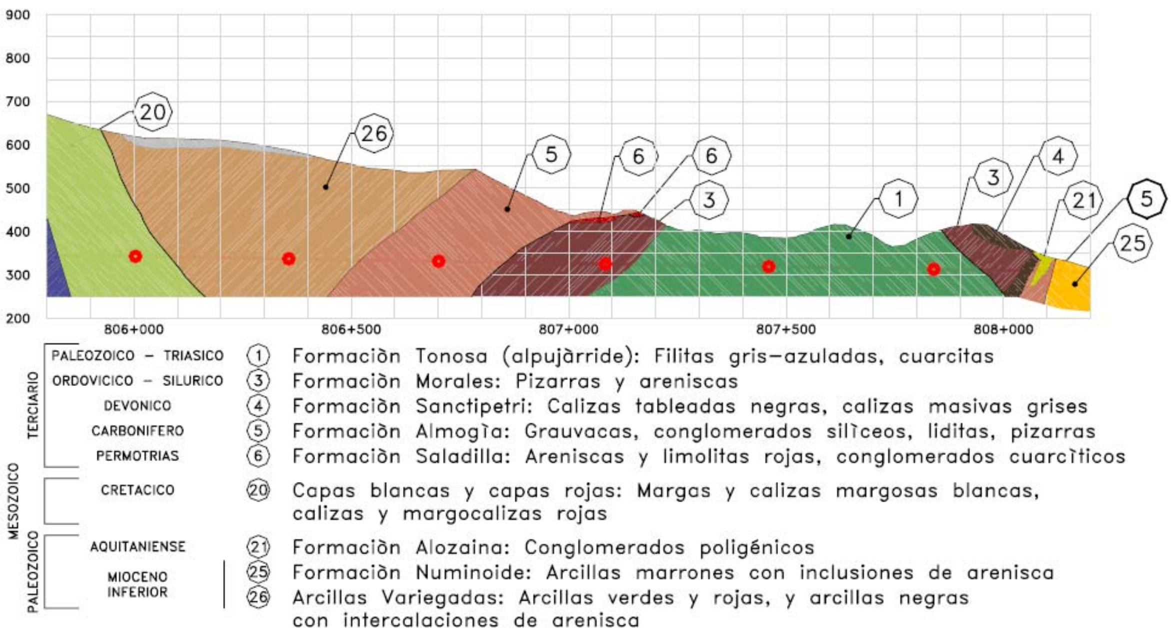


Figure 5.9 : The abdalajis project-Tunnel geology of first section.

The DSU design concepts and characteristics have been described in detail in the article “New Design a 10 m universal double shield TBM for long railway tunnels in critical and varying rock conditions” published in the RETC 2003 Proceedings by Wolfgang Gutter and Paolo Romualdi.

This new type of TBM as shown in the figure 5.10 was developed and tested the first time by modifying in the tunnel an existing 3,7 m diameter Wirth DS TBM that was stuck by the squeezing ground in a fault area under high cover in the Val Viola tunnel project, in the Italian Alps. Basically the existing TBM was modified by :

- Increasing the diameter of excavation to increase the overcutting in respect of the rear shield and of the segmental lining
- Increase the diameter of the front shield to eliminate the step-back joint between the front and the rear shield,

These rather simply modifications allowed the TBM to restart the excavation and to achieve over 25m/day of production in the same ground that was trapping the TBM before. The most special characteristics and design features of these TBMs are:

- The extremely high power and torque for a 10 m diameter rock TBM: 4900 kW and 18700 kNm
- The high main and auxiliary thrust, respectively 82500 kN and 152500 kN
- The 30 cm standard clearance between the excavation and the segmental lining outer diameter
- The possibility of additional over-cutting of 20 cm on diameter
- The possibility to displace vertically the cutterhead in respect of the shield
- The no.29 through the shield probe/pipe positions in the upper 180° of the gripper shield drill at a 7° angle to the tunnel wall
- Tunnel convergence was one of the biggest concerns in the project design phase since the large tunnel diameter and the presence of argillite rock formations under high cover. The DSU TBM proved however to be able to cope with the rapid convergences of the tunnel walls without any problem and without having to use the full overboring capability of the machine, thanks to the particular shield design of this new type of TBM (Figure 5.10).

Figure 5.10 : The abdalajis project-Double shield TBM.

This achievement represent a very important step in the tunnelling technology since it will allow the utilization of mechanized excavation in many critical tunnels to be bored under high cover in weak rock formations were before only the conventional method was adopted, with the consequent high costs and long construction times.

A long section of the tunnel in the argillite (Arcillas Variegadas formation) was characterised by large instabilities of the tunnel face.

In the most critical sections the ground was behaving like a non cohesive ravelling ground.

Under these conditions, without taking special measures, the over-excavations and face collapses would have buried the TBM in the tunnel.

For this reasons the following measures were implemented in several critical sections:

• Face stabilization treatments - (Figure 5.11) - when the over-excavation at the face was increasing above a couple of meters in front and above the cutterhead, the TBM advance was stopped, the voids filled with resin foams and the collapsed material in front of the face consolidated with chemical grout mix. Than the TBM was advanced for few strokes until the treatment had to be repeated.

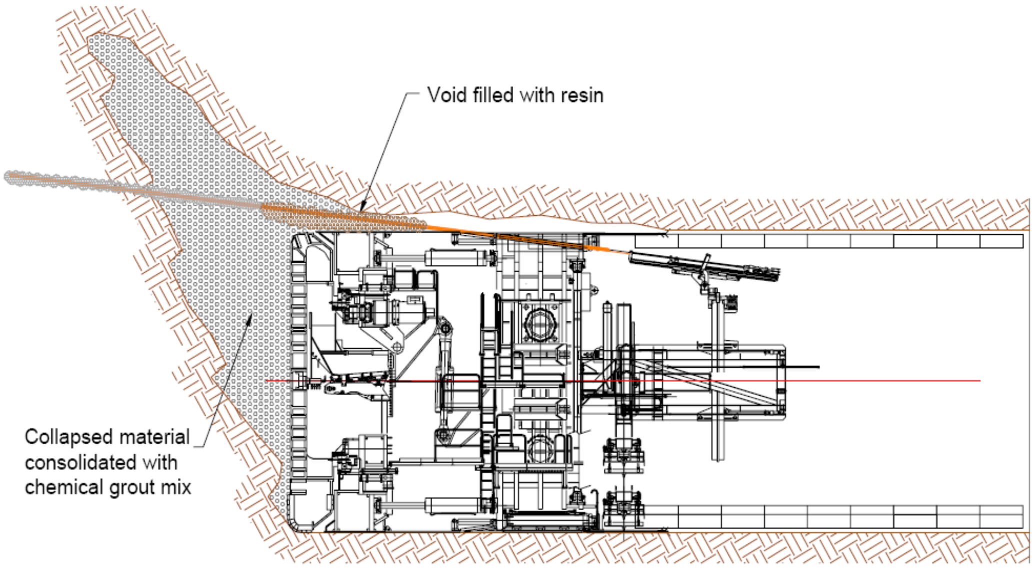


Figure 5.11 : Face stabilization treatment.

• Pre-treatment of the ground ahead of the face - (Figure 5.12) - in the most critical sections the weak argillite was behaving like a flowing gravel and even the above described face stabilization treatments were not sufficient to control the face collapses and the ground itself had to be treated in front of the TBM.

A pattern of fiberglass pipes were executed (Figure 5.13 and 5.14) through specific holes in the rear shield of the TBM in order to stabilise the crown of the tunnel. These piles were than grouted with chemical grouting mix (GEOFOAM or MEYCO), very successful in penetrating in the argillite formations and increase the cohesion of the ground enough to avoid face collapses. In the worst tunnel sections this treatments have been repeated each 3-5 m [17].

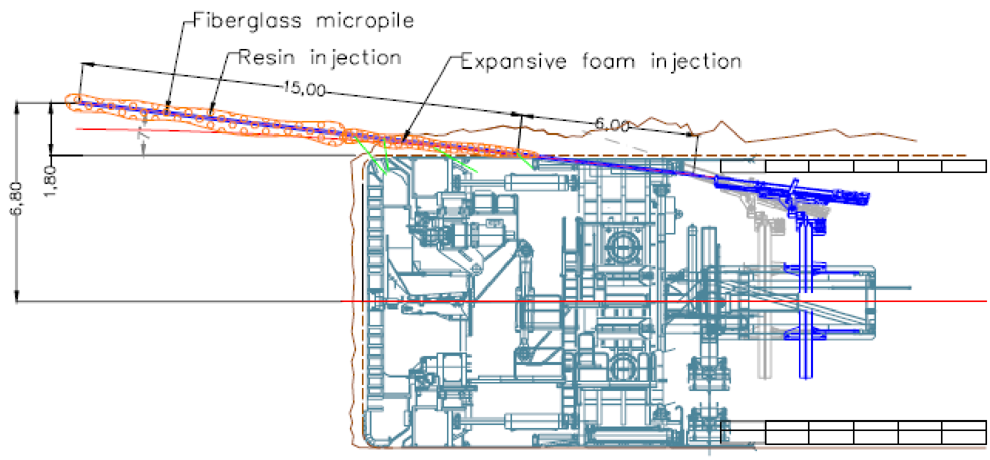


Figure 5.12 : Pre-treatment (Pre-injection) of the ground ahead of the face.

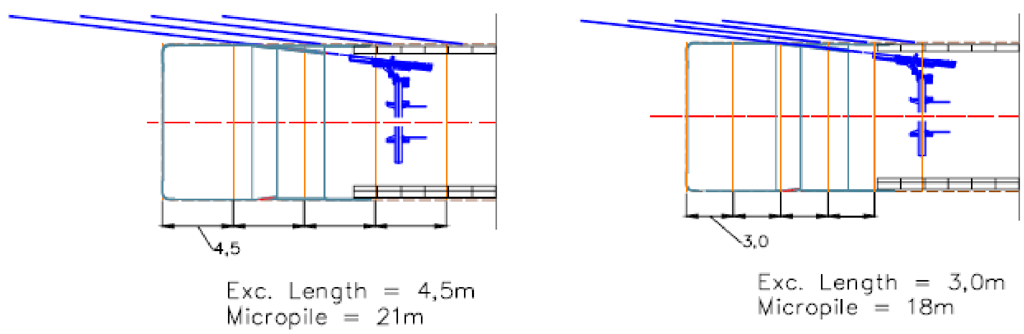


Figure 5.13 : Pre-injection pattern of the ground ahead of the face.

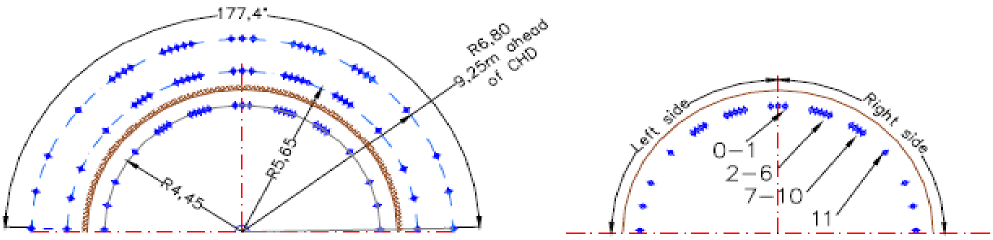


Figure 5.14 : Pre-injection pattern in front of the cutterhead.

5.4 Arrowhead Tunnels Project, Southern California, USA

Conditions faced by crews on the Arrowhead tunnels in Southern California were more difficult than anyone could have predicted.

Construction of the three major tunnels on the 44 mile (70km) Inland feeder conduit (Figure 5.15) to transfer water from the Colorado and Northern California aqueducts to the Metropolitan Water District's new Diamond Valley Reservoir had started by 1999, but a report in mid-1999 started that the first TBM in the first tunnel (Arrowhead East) was on stop while an extended program of grouting was progressed to limit excessive groundwater ingress. A later story reported termination of the Arrowhead East and Arrowhead West contracts ahead of complete redesign and rebid of the work.

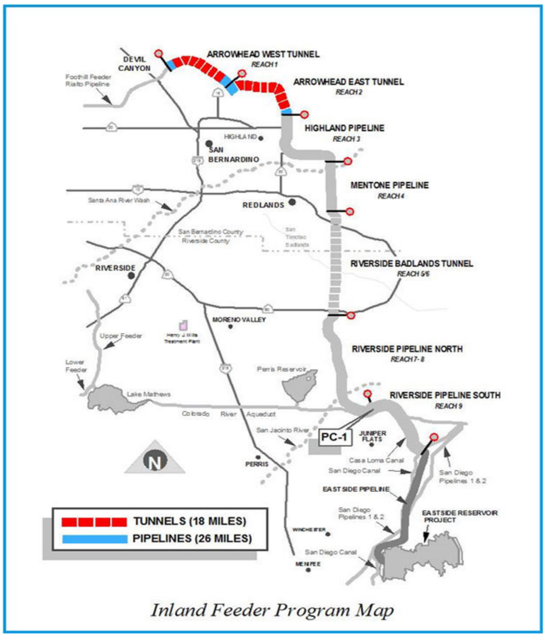


Figure 5.15 : Inland feeder program map [18].

The Shea/Kenny JV won the redesigned contract for the Arrowhead Tunnels in May 2002 with the lowest conforming competitive bid of $242 million. To meet the higher redesign specifications, the JV bought two new hard rock shielded TBMs from Herrenknecht (Figure 5.16) to erect a 5.8m (19ft) o.d. precast concrete segmental lining and complete the 6km (19,890ft) West Tunnel heading and the remaining 6.8km (22,443ft) of the East Tunnel heading toward the 2.4km (8,000ft) completed at the opposite end by the original Shank/Balfour Beatty contract.

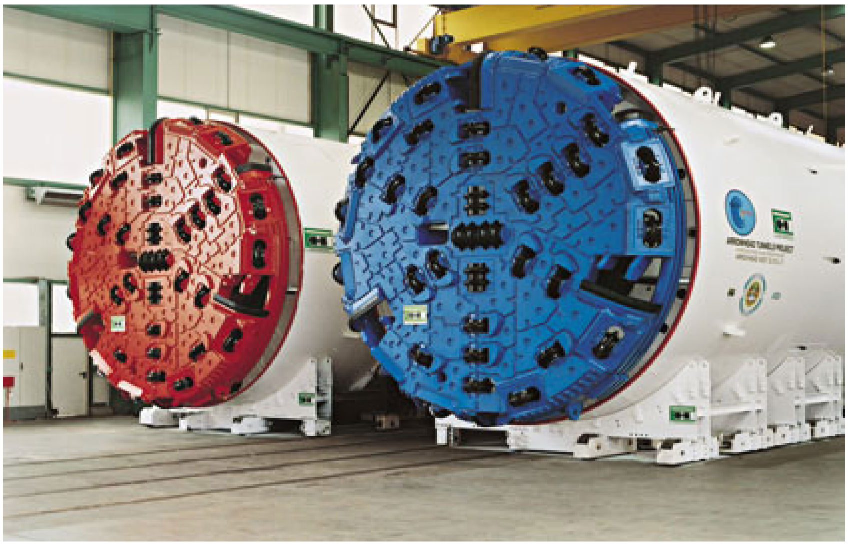


Figure 5.16 : Herrenknecht Arrowhead Hard Rock Machines [18].

The groundwater problems that caused cancellation of the first procurement attempt did not go away. They have continued and continue to define the current procurement process.

So to be able to overcome groundwater problems TBMs were equipped for extensive pre-excavation drilling and grouting facilities as shown in the Figure 5.17. They were delivered with four fix-mounted hydraulic drill rigs and a fifth portable rotary drill for mounting on the segment erector. Between them, the rigs could drill a total 26 holes; 15 at 1.5° through the bulkhead and cutterhead directly into the face, and 11 at 8° through equally-spaced glands in the shield skin. State-of-the-art Häny grout mixing and pumping stations were installed on the TBM backups along with silos for holding bulk pre-excavation grout materials.

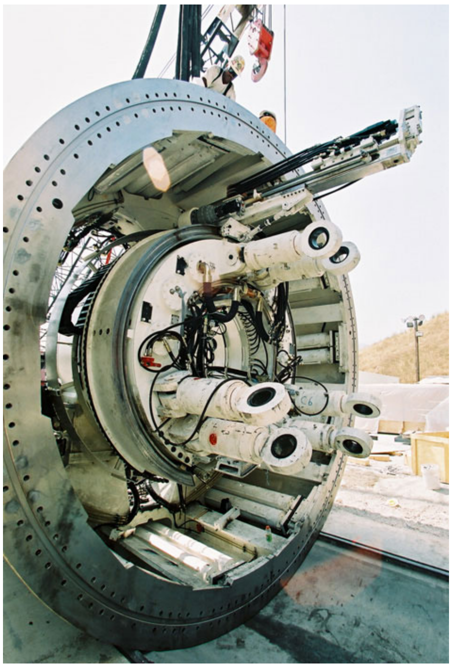


Figure 5.17 : Drive unit with its forward grout/probe drill in assembly [18].

All types of ground were causing delays. Squeezing and unstable ground required hand-mining to free the shield; raveling ground required special backfilling and grouting to stabilize the face; weak rock slowed drilling cycles due to hole collapse and ineffective sealing of grout packers; hard rock increased cutter wear and slowed penetration; water inflows and high water pressures washed-out fines from the excavated material and overwhelmed the muck handling systems; and weak, unstable or leaking grout at the plug zone ahead of a new grouting cycle caused delay to drill and grout extra holes or allow grout to set.

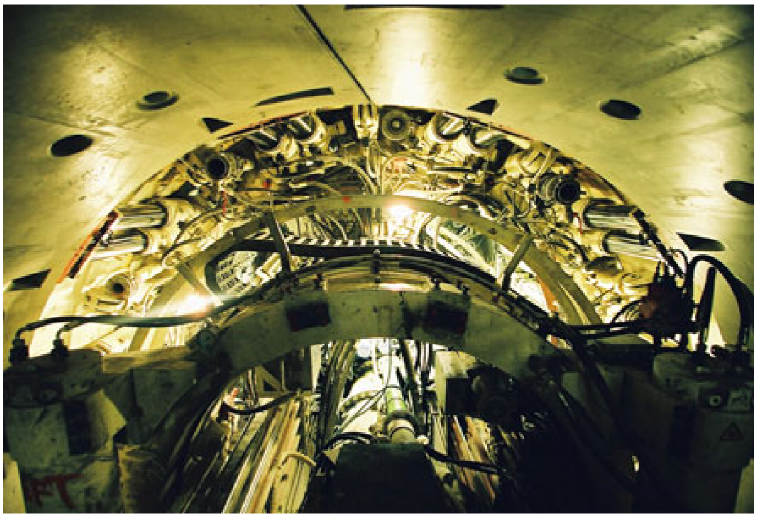


Figure 5.18 : Inside the shield of hi-tech TBM [18].

While the TBMs were designed according to contract specifications, they were not entirely able to cope well with the zones of 'variable and highly altered' ground or with the consequences of high water inflows or excessive volumes of fines produced. The TBM's and their systems needed to be adapted to conditions as actually encountered. These modifications are shown in detail on table 5.2.

Table 5.2 : Original specifications and upgrades of the two 5.8m Herrenknecht Hard Rock Machines.

Design criteria	Original specification	Modification
Tunnel length (m)	7211	7067
Total installed power	3,600kW (4,825hp)	
Static pressure capacity	10 bar (147psi)	
Operating pressure	3 bar (44psi)	
Disk cutters	39 x 432mm (17in)	
Max load/cutter	250kN (56,000lb)	
Cutterhead power	2,000kW (2,800hp)	
Cutterhead operational torque	2,000kNm (1.5m-ft-lb)	3,520kNm (2.6m-ft-lb)
Cutterhead exceptional torque	2,323kNm (1.7m-ft-lb)	4,400kNm (3.3m-ft-lb)
Maximum cutterhead speed	9.75 rev/min	7.5 rev/min
Thrust force	29,270kN (3,300 ton)	58,540kN (6,600 ton)
Screw power	160kW (215hp)	

Perhaps most significantly, the mechanical systems (see Table 5.2) , management, and the materials used for the vital water control pre-grouting regimes were modified.

A further 19 drill ports at 4° angle were added to the bulkhead to increase to 34 the number of probe and grout holes that could be drilled directly into the face using the TBM drilling equipment as shown in the Figure 5.19. In September, it was said that the 34 x 1.5° and 4° angled drill ports in the bulkhead and directly into the face are used more frequently and that the two drills on the rear ring beam for drilling 11 x 8° angled holes through the shield are used now only rarely. Collapse of grout holes has been a constant source of delay and frustration as has achieving adequate penetration of grout materials into soft, unstable ground where it is needed most.

Water inflows in excess of 12.6lt/sec (200 gpm) from a single 30m (100ft) long probe hole and drilling back pressures of up to 250psi (17 bar) have been recorded. In specific locations, drain holes have been required through the segmental lining to relieve excessive ground water pressures.

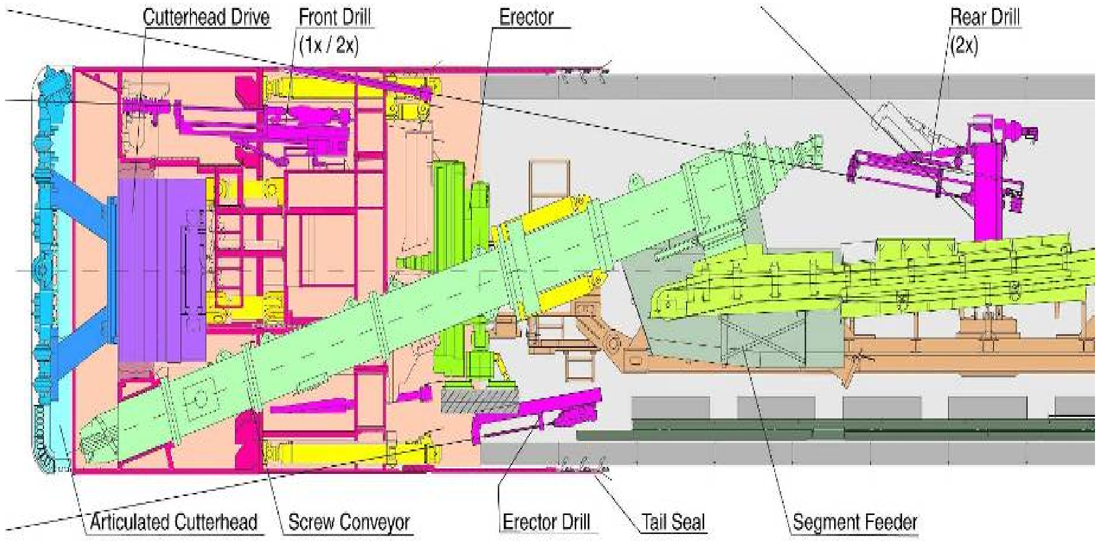


Figure 5.19 : TBM Design for Pre-injection.

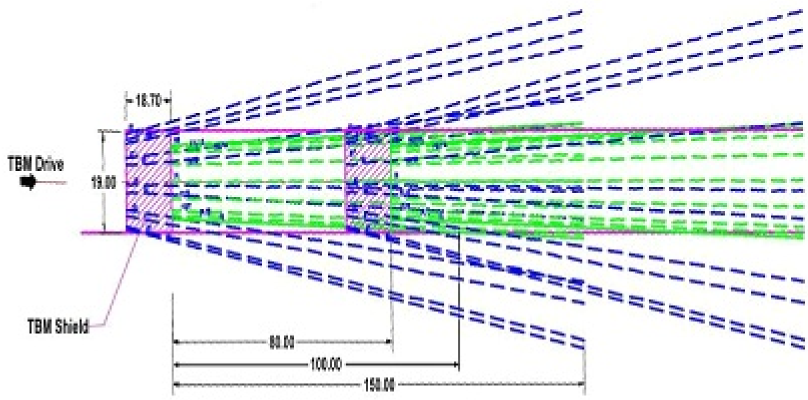


Figure 5.20 : Shield and Rib-Holes layout Arrowhead Tunnels.

With the assistance of material experts different grout mixes have been developed and used for different purposes. These have included micro-fine and Type II and III cement based grouts, polyurethane and chemical grouts and colloidal silica grout from BASF, which has been successful in reducing water inflows in conditions that have resisted micro-fine cement grouts, even in dilution.

Those grouts were injected using the pre-injection layout as shown in the Figure 5.21. Respectively Figure 5.22 and 5.23 show injection layout in good rock and water bearing with fault zones.

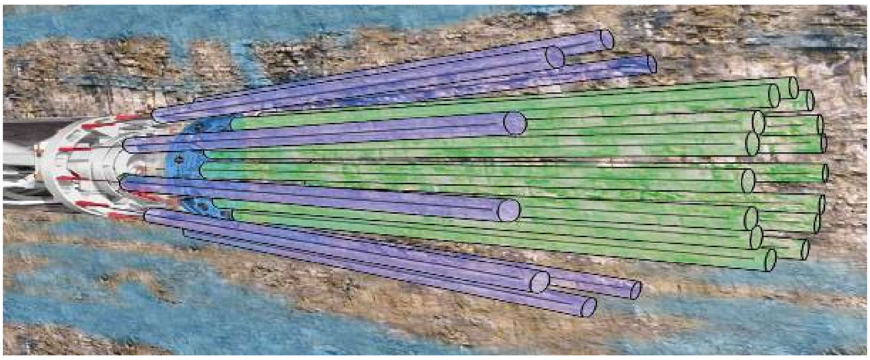


Figure 5.21 : Pre-injection layout.

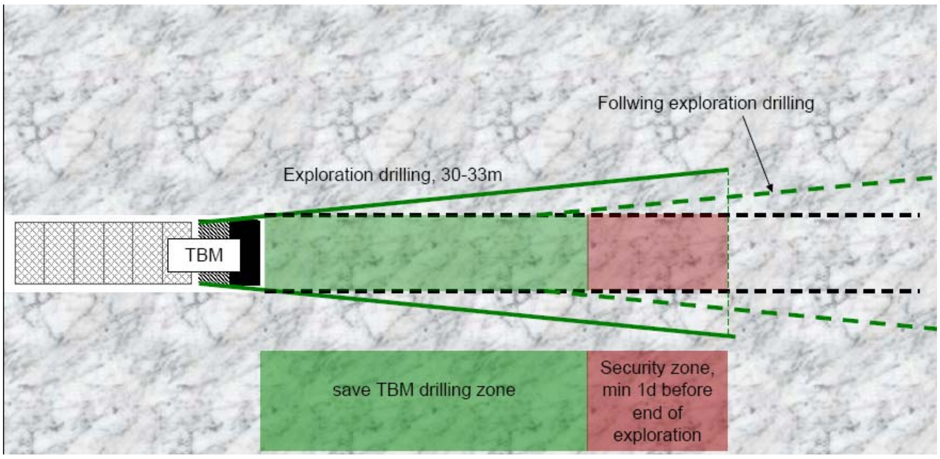


Figure 5.22 : Layout of exploratory drilling scheme ahead of the TBM- Situation (a) only good rock contiditions detected.

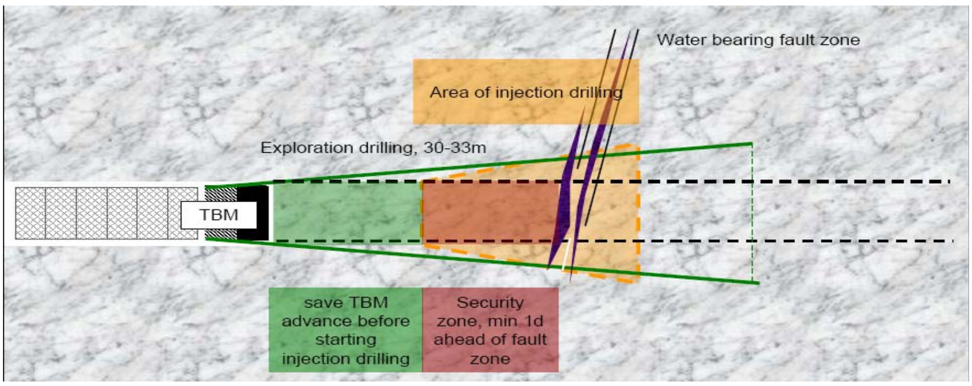


Figure 5.23 : Layout of exploratory drilling scheme ahead of the TBM- Situation (b) water bearing fault zones detected.

To reduce the water inflow in the arrowhead west tunnel mainly micro cement injection was used (Figure 5.24) on the other hand colloidal silica was mainly the product used for east tunnel (Figure 5.25).

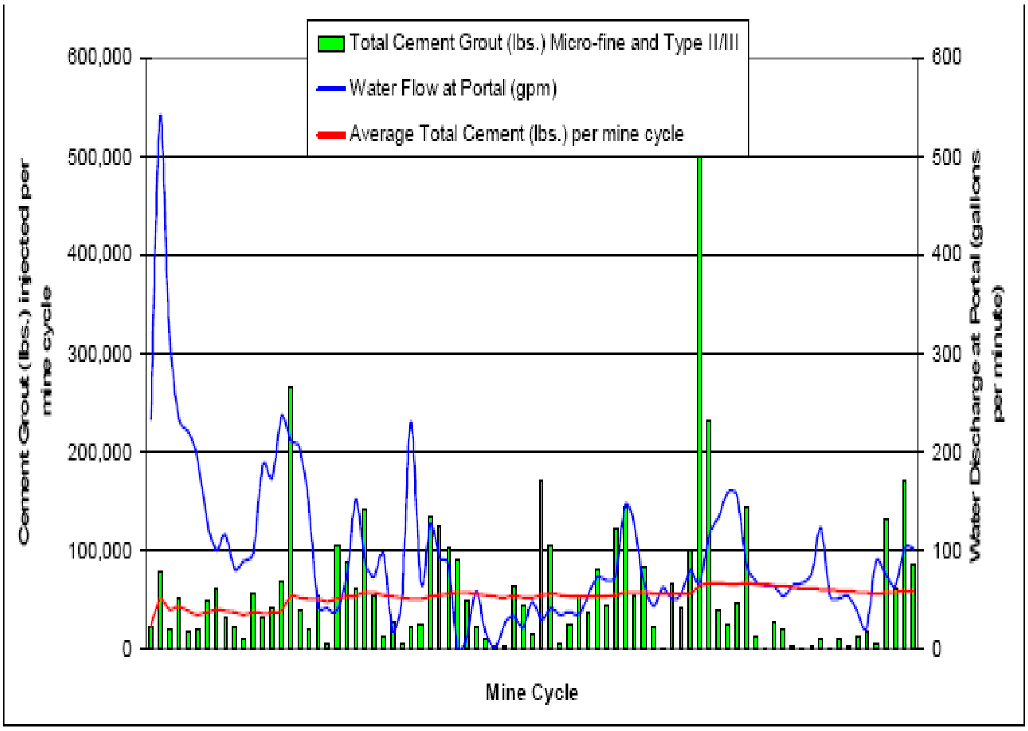


Figure 5.24 : Arrowhead West Tunnel cement injection data.

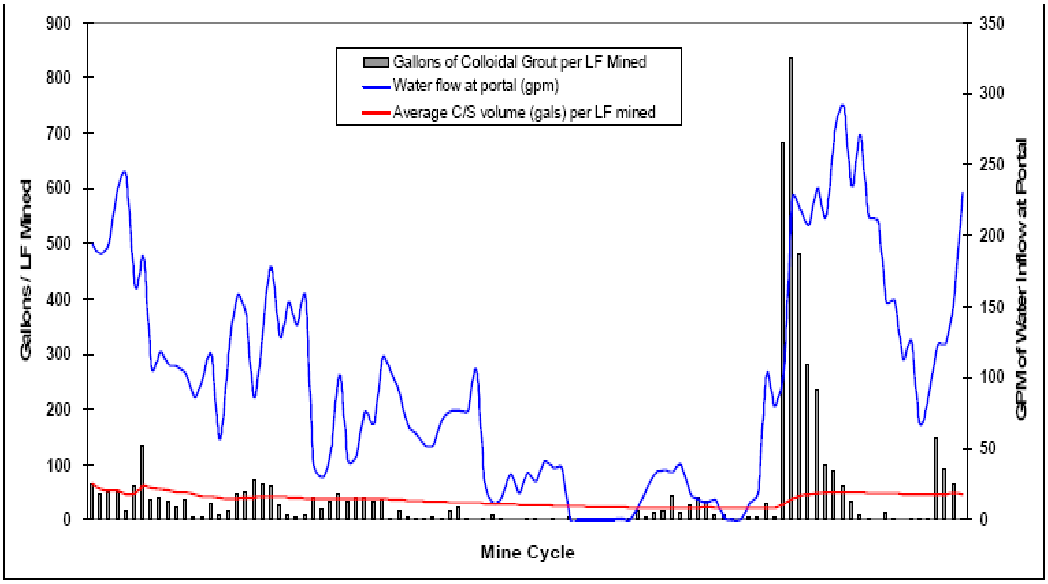


Figure 5.25 : Arrowhead East Tunnel colloidal silica injection data.

Table 5.3 shows the best advance rates in east and west Arrowhead tunnels until October 2007 when using pre-injection grouting [18].

Table 5.3 : Best Advance rates from Arrowhead Tunnel until October 2007.

Progress	East Tunnel	West Tunnel
Tunnel total length	6,840m(22,443ft)	6,062m (19,890ft)
Tunnel (TBM drives)	6,765m(22,195ft)	6,023m (19,760ft)
Best month to date	361m (1,185ft)	250m (820ft)
Best week to date	108m(355ft)	67m (220ft)
Best day to date	29m (95ft)	24m (80ft)
Average month advance	118m(387ft)	90m (297ft)
Average week advance	30m (97ft)	22.5m (74ft)
Average day advance	5.9m (19.4ft)	4.5m (14.8ft)

6. TWO DIFFERENT TBM PROJECTS FROM TURKEY WHERE GROUTING HAS BEEN APPLIED

6.1 Bagbasi Dam and Mavi (Blue) Tunnel Project, Konya, Turkey

Almost 67% area of Konya is the agriculture area and is total 2 million 560 thousand 196 hectares. The amount irrigable areas existed in the province is 1 652 762 hectares and still 377 426 hectare land is being irrigated. This area consists of the 14% of the total agriculture land. 308.073 Ha land has been irrigating with Konya Plains Irrigating Projects up till today and the amount of the land to be irrigated is intended to be increased to 617.923 ha area at the end of the completion of Blue Tunnel Project. The projects to develop the water resources of the plains at the covered basin characteristics which take place in the Konya and Karaman provinces have been called Konya Plain Projects (KPP) since 1985. KPP project includes meeting drinking, potable and industrial water needs and hydroelectric energy production besides irrigating the Konya and Karaman Plains primarily. KPP project is total 12 containing 9 major water project, 1 drinking water project, 1 Goksu basin energy project and 1 project for many individual surface and underground water irrigations.

The biggest project out of the projects under the scope of KPP is Konya – Çumra Project and is the second biggest irrigation project following the GAP in terms of irrigable area. Konya – Çumra Project is consisted of III Stages and Stage III project constitutes the biggest and the most important parts of it.

KPP Project will carry over annual 414,13 million m³ water of Upper Goksu basin which flows to Mediterranean via Blue Tunnel which will be the second biggest irrigation tunnel of Turkey following the Sanliurfa tunnels with its 17 km length to Konya closed basin through the dams of Bagbasi, Afsar and Bozkir to be built.

Furthermore, water recycled from refined waste water and irrigation with the ground and underground water to be taken through Apa-Hotamis conduction canals and Hotamis storage will both raise the underground water of Konya plain and support the irrigable water of approximately 200.000 hectares cultivate areas which are the final irrigable areas. Moreover, annual 147,5 million kwh energy production will be supplied at the 50,6 MW established power with 3 units of HES. The Blue Tunnel constitutes the most important pace of KPP. Blue Tunnel project which will make approximately one third of Konya plain irrigable will change the plant pattern of Konya and provide production variation by opening many arid areas which cannot be irrigated.

Figure 6.1 : Upper Göksu River Basin

6.1.1 General geological outlines of the project area

The region where the Mavi Tunnel will be excavated is located in the Middle Taurides nappe-structured belt (Southern Turkey), about 80 Km South of Konya City.

According to the existing and well developed geological studies related to the Mavi Tunnel project, the geological asset of the project area includes three different structural units:

• Geyik Mountain Unit;

• Bolkar Mountain Unit;

• Bozkir Unit.

Bolkar Mountain Unit and Bozkir Unit are allochthonous nappes overthrusted, with a general South vergence and with more than kilometrical estimated horizontal displacements, over the underlying Geyik Mountain autochthonous Unit.

The foreseen tunnel alignment, directed as a rough rectilinear trace from SSE to NNW, passes through all these three main units, whose most important characteristics are described as it follows.

Geyik Mountain Unit (lower Paleozoic - lower Tertiary): it is the lowermost structural unit in the region and is considered almost autochthonous. This unit is formed by two sub-units called Hadim Unit and Yildizli Mountain Unit. These sub units are separated, in the region, by a low angle fault named "Korualan Fault".

Only Hadim Unit is directly interested by the tunnel alignment, so Yildizli Unit will not be described.

Hadim Unit is constituted by five main lythological formations, which are (in stratigraphic order):

• Caltepe Formation (Middle Cambrian);

• Seydisehir Formation (Ordovician);

• Polat Limestone (Middle Jurassic - Lower Cretaceous);

• Belkuyu Formation (Upper Creaceous - Paleocene);

• Avdan Formation (Lower Eocene - Upper Paleocene).

A huge regional stratigraphical “hiatus”, marked by an angular unconformity and corresponding to the Late Paleozoic-Early Mesozoic interval, separates Seydisehir Paleozoic Formation from Polat Mesozoic Limestone.

Only the last three formations (Polat, Belkuyu and Avdan) will be interested by the tunnel alignment.

Bolkar Mountain Unit (Late Devonian - Late Creaceous): it is an allochthonous nappe that, in the project area, has horizontally overthrusted, since Eocene time, the Geyik Mountain Unit formations. Bolkar Unit is represented by three main formations (in stratigraphic order):

• Taşkent Formation (Upper Permian);

• Ekinlik Formation (Upper Triassic);

• Sinat Mountain Limestone (Lower Jurassic – Lower Cretaceous).

Sinat Mountain Limestone lays uncomformably over Ekinlik Formation. Only the last two formations (Ekinlik and Sinat) will be directly interested by the tunnel alignment.

Bozkir Unit (Triassic - Cretaceous): it is the structurally higher allochthonous nappe in the project area, overlying tectonically Bolkar Mountain Unit and parts of Aladağ Unit (outside of the project area). This structural unit, very disomogeneous from a lithological point of view, seems to represent a “mélange” made of shelf and basin sedimentary rocks, basic undersea volcanites, tuffs, diabasis, serpentines and variable sized chaotic blocks.

The Formations of this unit in the project area are (in stratigraphic order):

• Dedemli Formation (Triassic);

• Hill Mahmut Limestone (Triassic - Cretaceous).

None of these formations will be encountered along the tunnel alignment in depth.

The following Table 6.1 and the geological sketch map reported hereafter (Figure 6.2) summarize, in extreme synthesis, the regional geological asset of the project area and the nature of the contacts among the different units.

Table 6.1 : Geological table of the project area.

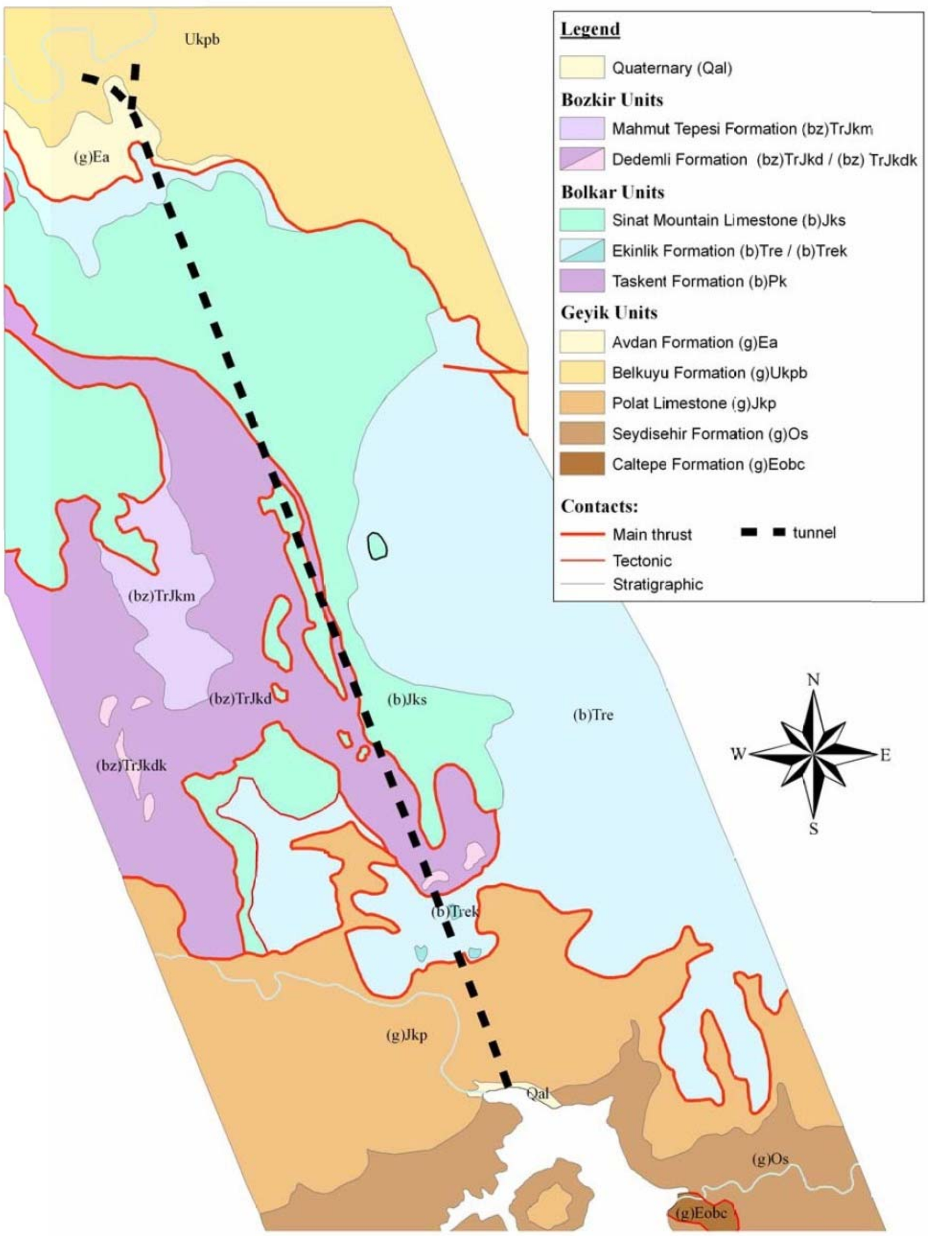


Figure 6.2 : Geological sketch map of the project area.

6.1.2 Geological and hydrogeological conditions along the Blue Tunnel alignment

On the basis of the geological studies carried out for the Mavi Tunnel Project (see Reference Documents), the SSE-NNW tunnel alignment will cross, running at the base topographic level of about 1100 m a.s.l., most of the main structural units present in the studied area and briefly described in the previous sections of this report.

From the existing geological section of the tunnel (Figure 6.3), provided at the original scale 1:25.000/1:5.000 and schematically reproduced in Fig. 2, it can be deduced that, starting from the inlet site located in the Bagbasi reservoir (Pk. 0+000 Km), the tunnel will be excavated for about 3,6 Km in the Polat Limestone Formation, that is part of the basal Geyik autochthonous Unit.

The Jurassic-Lower Cretaceous Polat Formation is made up of uniformly bedded dolomites and dolomitic limestones showing local open karstic solution channels. The borehole TSK-11, executed in this part of the alignment during the past studies, reveals that the groundwater level is here located next to the tunnel level. The presence at short distance of the very steep Goksu river valley, representing the most important drainage base level of this area, seems to justify this local depression of the groundwater level.

After this first part, the tunnel will then cross the main North dipping thrust fault, separating the autochthonous Geyik Units from the overlying Bolkar allochthonous nappe.

Beyond this thrust fault zone, the tunnel will continue, for about 5,6 Km (from PK 3+625 to PK 9+220), in the upper triassic Ekinlik Formation, made up of an heterogeneous association of irregularly bedded limestones, dolomites, sandstones, volcanites and very low permeability shales.

In this part of the tunnel, two boreholes (TSK-16 and TSK-12) have been previously executed, attesting a groundwater level of about 200÷300 m higher than the tunnel level. This formation is expected to contain limited quantities of groundwater, except where limestone facies with possible dissolution voids content locally prevail.

After this section the tunnel will penetrate the overlying Jurassic- Cretaceous Sinat Formation, made up of massive neritic limestones, locally jointed and fractured, quite intensively interested by karstic structures in depth.

In this part of about 3,9 Km length, the Sinat Formation seems to be structured in a wide synform, determined by an apparent local inflection of the base thrust surface in depth. This particular formation asset seems to configure, in this tunnel section, a local trap for the groundwater, originating, in combination with the karstic nature of the Sinat limestones, expected critical hydrogeological conditions. In fact, the design has here foreseen possible high water inflows during excavation works, according to the possibility of intercepting karstic solution channels.

Proceeding towards NNW the tunnel will cross, once again, the Ekinlik Formation and the underlying main thrust fault surface, here locally South dipping, for a total length of about 1,1 Km.

After the thrust fault zone, the tunnel will then proceed through the uppermost formations of the Geyik autochthonous Unit. In detail, the tunnel will develop, for a short length, in the Avdan Formation (from about Pk 14+230 to Pk 14+495), and, until the tunnel end, in the underlying Belkuyu Formation.

Avdan Formation represents the uppermost autochthonous formation in the studied area and it is mainly made up of arenaceous deposits and shale layers organized in facies of flysch. During the excavation of the tunnel, water may occur only as leakage in sandstone layers, resulting in a generally low rock mass permeability.

The underlying Belkuyu autochthonous Formation is made of locally karstic bituminous limestones and dolomites, showing from medium to thick bedded structure. The permability of this formation is much more higher than the previous Avdan Formation, but, due to the proximity of the Mavi Strait, leakage is expected at the tunnel level only during rainy seasons. From a structural point of view, the previous studies carried out in the project area pointed out that the main structural features characterizing the area are linked to the presence of low angle South vergent thrust faults, separating the main structural units.

However, at a minor scale of observation, the same studies state that, in general, the geological formations of the Geyik autochthonous Unit roughly show a uniform bedded structure and appear to have undergone relative low tectonic disturbs.

On the contrary, the allochthonous formations constituting the overlying Bolkar and Bozkir nappes seem to be characterized by a more complicated internal structures, determined by the pervasive presence of folds and faults of various generations and attitudes.

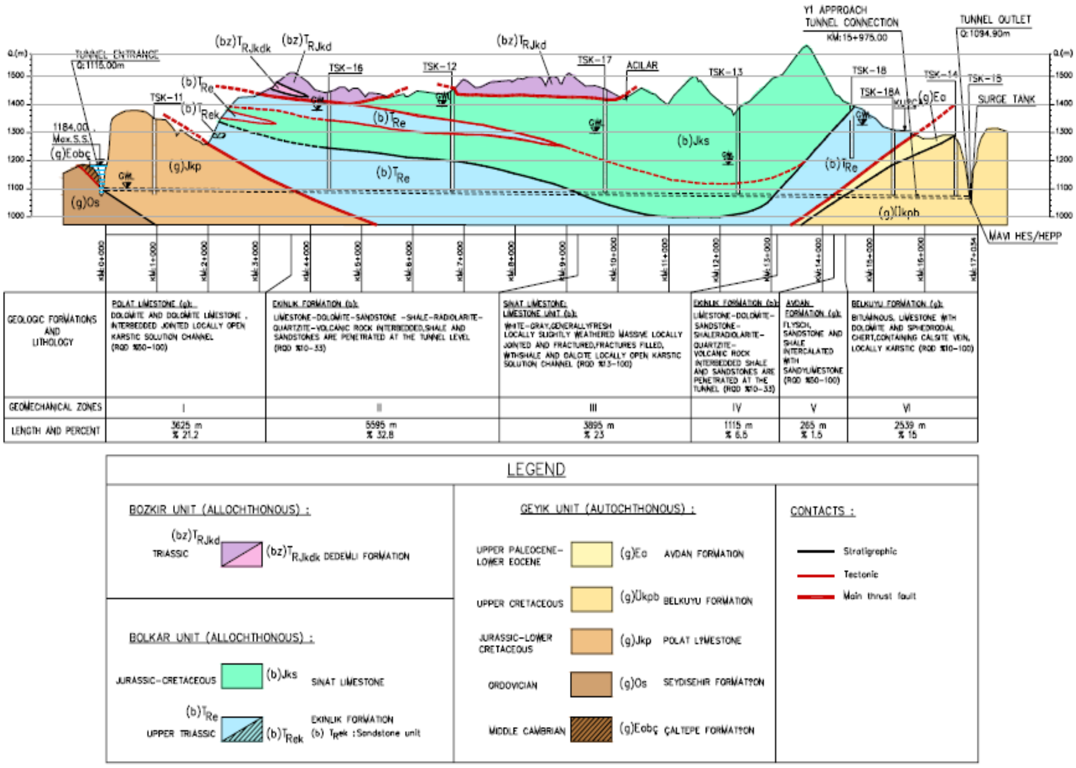


Figure 6.3 : Geological section of Mavi (Blue) Tunnel

6.1.3 Geomechanical classification of rock masses

In the geological profile as shown in the figure 6.3 a well defined partitioning of the tunnel route in six main “zones” (from I to VI), considered roughly homogeneous from a geomechanical point of view.

This partitioning has been mainly conceived on the basis of lithological characteristics of the geological formations present at the tunnel depth and its limits (boundary of zones) indeed coincide with the formation contacts.

The geomechanical classification of the rock mass adopted in the previous studies is based on the Q Index, as defined in the Q-System Geomechanical Classification originally proposed by Barton et Al. (1974).

Q-System is a quantitative classification system of rock masses specifically proposed for tunnel rock support design and is based on a numerical assessment of the rock mass quality.

By definition, the numerical value of the Q index varies, on a logarithmic scale, theoretically from 0.001 (exceptionally poor rock mass) to a maximum of 1,000 (exceptionally good rock mass) and is defined by six parameters and by the following equation:

$$Q = (RQD / Jn) \times (Jr / Ja) \times (Jw / SRF) \tag{6.1}$$

where:

- RQD is the Rock Quality Designation;

- Jn is the joint set number;

- Jr is the joint roughness number;

- Ja is the joint alteration number;

- Jw is the joint water reduction factor;

- SRF is the stress reduction factor.

For each zone recognized along the tunnel section, the above mentioned geological profile also reports a well determined range of expected values of Q index, among which a minimum, a medium and a maximum value have been chosen with a conservative design criterion, as summarized in the Table 6.2 below.

Table 6.2 : Expected values of Q index along the tunnel alignment.

Zones	**Chainage**	**Lythological Formation**	**Min**	**medium**	**max**
I	0 - 3+625	Polat	0.1	1	10
II	3+625 - 9+220	Ekinlik	0.025	0.1	1
III	9+220 - 13+115	Sinat	0.4	1	10
IV	13+115 - 14+230	Ekinlik	0.025	0.1	1
V	14+230 - 14+495	Avdan	0.025	0.1	4
VI	14+495 - 16+620	Belkuyu	0.1	1	10

6.1.4 Permeability of rock masses

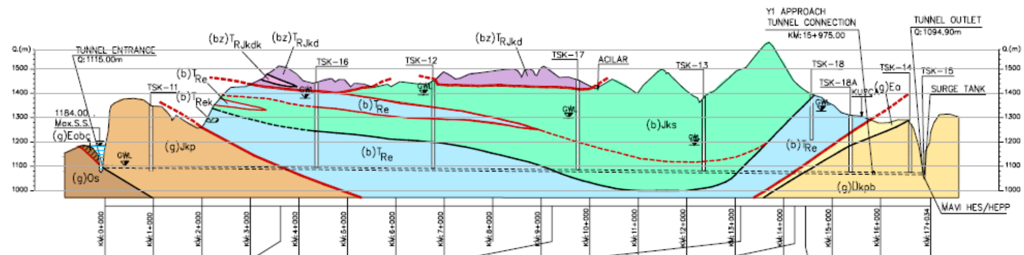
The permeability of the rock masses present along the tunnel alignment has been qualitatively deduced, for every homogeneous zones, on the basis of the lithological composition of the rocks and of the ground water conditions predicted by the previous studies.

In particular, as shown in the following Figure 6.4, in Zones I, III and VI, where mainly karstic limestones and fractured dolomitic limestones occur, the permeability of the rock masses can be assumed to be high and very high.

This evaluation agrees with the ground water conditions predicted by the existing studies, that have estimated, during tunnel excavation works, ground water leakages during rainy seasons in Zones I and VI and very high inflows conditions in Zone III.

In Zones II, IV and V, where Ekinlik complex lithological formation and flysch type sediments occur, the average permeability of the rock masses can be considered to be very variable but, in general, lower than in the other sections of the tunnel.

In these zones, in fact, only locally wet and moist conditions are expected during tunnel excavation.



GEOMECHANICAL ZONES	I	II	III	IV	V	VI
LITHOLOGY	POLAT LIMESTONES (g): DOLOMITES, DOLOMITIC LIMESTONES AND KARSTIC LIMESTONES	EKINLIK FORMATION (b): LIMESTONES, DOLOMITES, SANDSTONES, SHALES, RADIOLARITES, QUARTZITES AND VOLCANIC ROCKS	SINAT LIMESTONES (b): WHITE-GRAY KARSTIC LIMESTONES	EKINLIK FORMATION (b): LIMESTONES, DOLOMITES, SANDSTONES, SHALES, RADIOLARITES, QUARTZITES AND VOLCANIC ROCKS	AVDAN FORMATION (g): SANDSTONES AND SHALES IN FACIES OF FLYSCH	BELKUYU FORMATION (g): BLACK-DARK GREY BITUMINOUS LOCALLY KARSTIC LIMESTONES
ROCK MASS PERMEABILITY	HIGH	LOW	VERY HIGH	LOW		HIGH
EXPECTED GROUNDWATER CONDITIONS	LEAKAGE IN RAINY SEASONS	LOCALLY MOIST AND WET	VERY HIGH INFLOWS	LOCALLY MOIST AND WET		LEAKAGE IN RAINY SEASONS

Figure 6.4 : Estimated permeability conditions of the rock masses along the tunnel alignment.

6.1.5 Double shield TBM

The TBM shown in the figure 6.8 is a completely shielded Tunnel Boring Machine.

The excavation is performed by a flat rotating cutterhead equipped with 17’’ disc cutters, which are mounted from the rear of the cutterhead face. Rotary power is supplied by six electric motors (through associated gear reducers and pinions) placed behind the main bearing, on which the cutterhead is mounted.

The main bearing is placed inside the shield immediately behind the cutterhead, the front shield. As the cutterhead rotates, a set of oleodynamic cylinders furnishes the forward thrust that, together with the rotary power, effectively disgregates the rock. If the ten main thrust cylinders are used for this purpose, during their extension two gripper shoes are anchored to the walls of the tunnel being excavated through hydraulically actuated pads situated at both ends of two gripper cylinders. The part of the TBM from which these anchoring shoes extend, and where the rear parts of the main thrust cylinders are fixed, is called the gripper shield.

Between the front and gripper shields there are two concentric shields which overlap to protect the main thrust cylinders (Figure 6.5) at all stages of the boring cycle: the outer telescopic shield moves together with the front shield, and the inner telescopic shield is fixed to the gripper shield through a set of articulation cylinders. Using the articulation cylinders the telescopic shields can be closed inside one another even when the main thrust cylinders are extended, for various purposes (facilitate steering, maintenance, visually check rock condition, etc).

Figure 6.5 : Thrust cylinders of Mavi (Blue) Tunnel Double Shield TBM.

Two torque cylinders are also placed between the front and gripper shields to contrast the reaction to the rotary power, which would tend to twist the telescopic shields in the opposite verse of the cutterhead. When the main thrust cylinders have reached full extension, two stabilizer cylinders (with relative shoes) at the 2 and 10 o'clock positions can be extended out of the front shield to anchor it to the rock walls, so that by retracting the main thrust cylinders the rest of the TBM and back-up can be pulled forward (Figure 6.6). After this phase, called regripping, the stabilizers are retracted and the gripper cylinders can be extended again to start a new boring cycle. The stabilizer cylinders can also be extended during the excavation itself, in low-pressure mode, to act as dampers to reduce possible excessive vibrations.

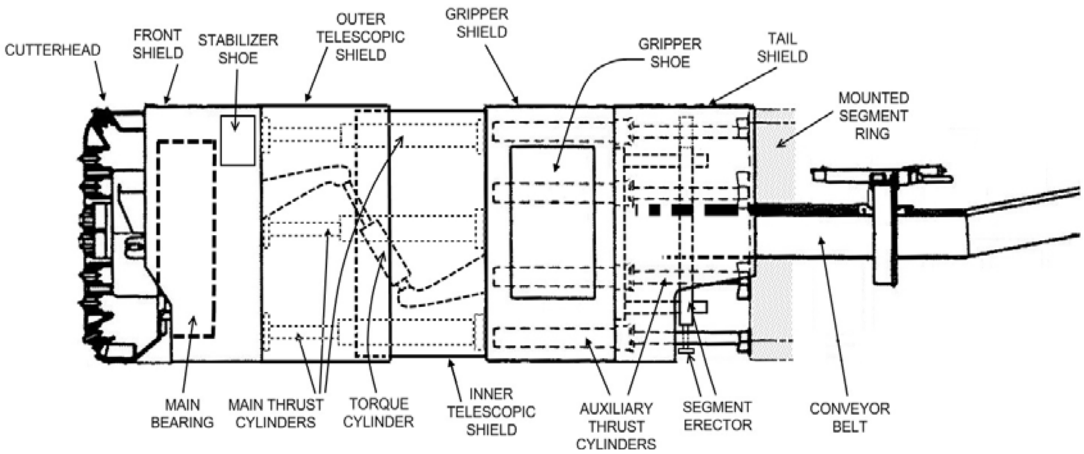


Figure 6.6 : Schematic general drawing of Double Shield TBM.

If the rock is too soft or caved in to anchor effectively the grippers, another way to provide forward thrust is using the 14 auxiliary thrust cylinders which extend behind the gripper shield and push directly on the last tunnel segments placed. These cylinders are covered by the tail shield, inside which operates also the segment erector, which places, as the machine advances, the concrete pieces that constitute the tunnel walls.

Four hexagonal segments as shown in the figure 6.7 make up one tunnel ring. Until the rock conditions permit once again the use of the grippers (which is the standard way of operating), the auxiliary thrust cylinders get retracted after their full extension, and then other tunnel rings are constructed to push on. The auxiliary thrust cylinders are also used to keep in place the segments as they're being mounted by the erector.

Figure 6.7 : Hexagonal Tunnel Segments of Mavi (Blue) Tunnel.

A belt conveyor starting inside the front shield is used to transport rock cuttings (muck) from the cutter chamber to the rear of the machine; successively that muck is transferred into other two conveyors on the back-up which deposit it into cars used for removal from the tunnel. The conveyor is fixed to the main bearing support, but it can be retracted via a dedicated cylinder to facilitate maintenance in the telescopic shields area.

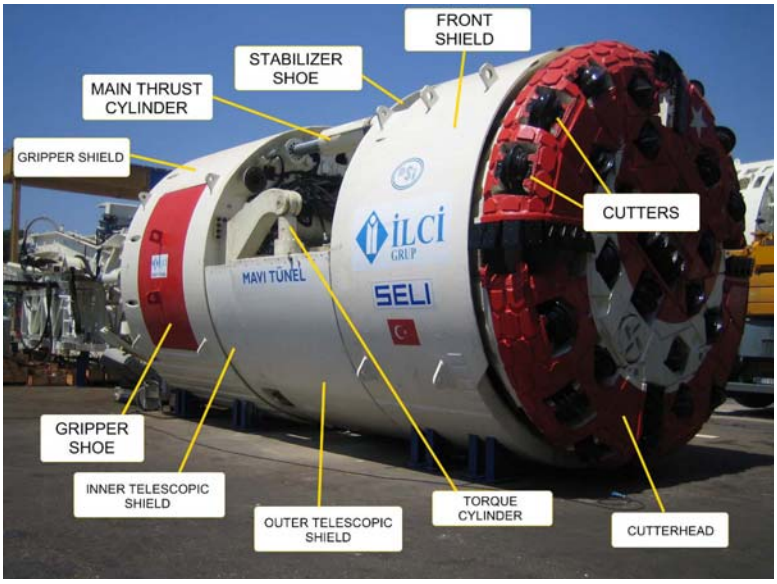


Figure 6.8 : Mavi (Blue) Tunnel Double Shield TBM.

Technical specification for Seli Double Shield Mavi (Blue) Tunnel hard rock machine is given in table 6.3.

Table 6.3 : Technical Specification of Mavi (Blue) Tunnel Double Shield Machine

Machine	Double Shield 0488120
Type	Telescopic double shielded TBM
Boring diameter	4880 mm
Minimum curve radius	400 m
Weight	Approx.390 tons (570 overall)
Length	11.2m (Shield); 165 m (TBM)
Cutterhead:	Flat with 17" disc cutters, plates
Maximum recommended thrust	8544 kN (32x 267 kN)
Rotation speed	0 to 10,9 RPM (cont. variable)
Drive power	6x315 kW
Torque from 0 to 7,5 rpm	2367 kN.m
Torque at 10.9 rpm	1620 kN.m
Unlocking torque	4200 kN.m
Main thrust	
Thrust force	25538 kN
Number of jacks	10
Maximum hydraulic pressure	350 bar
Auxiliary thrust	
Thrust force	27374 kN (exceptional : 40000kN)
Number of jacks	14
Maximum hydraulic pressure	345 bar (Max.520 bar)
Other characteristics	
Grippers anchoring force	13600 Kn
Probe drill length	80 m

6.1.6 Performance of Mavi (Blue) Tunnel Double Shield TBM

Figures 6.9;6.10 and 6.11 show the performance between December 2008 and November 2009 of Mavi Tunnel DS TBM

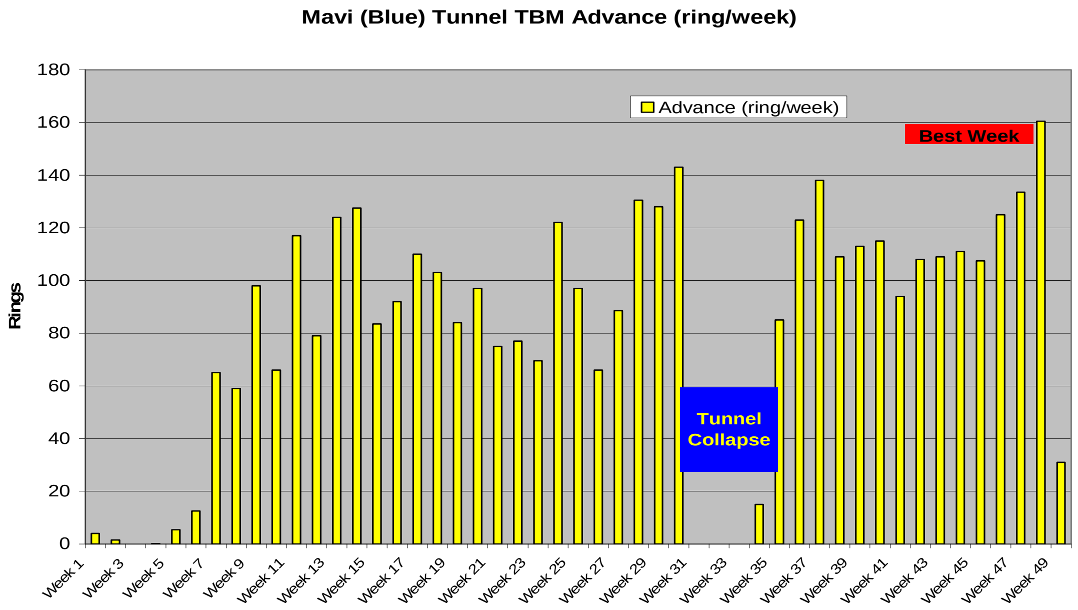


Figure 6.9 : Mavi (Blue) TBM Weekly Advance (December 2008 -November 2009).

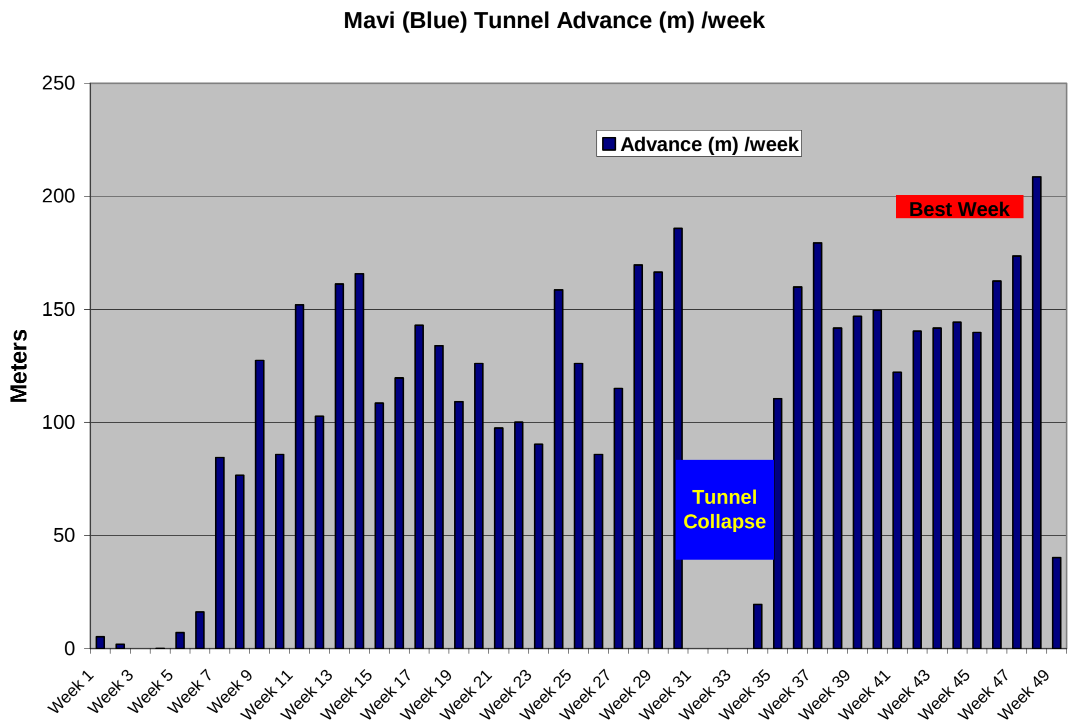


Figure 6.10 : Mavi (Blue) Tunnel TBM Advance (December 2008 -November 2009)

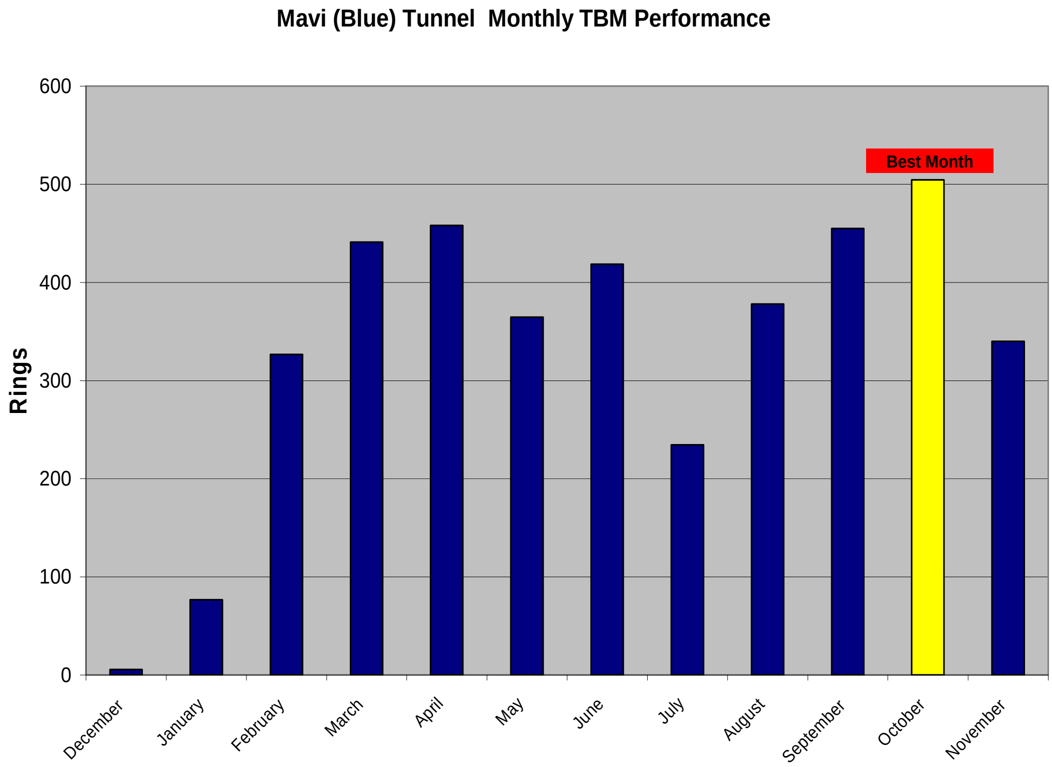


Figure 6.11 : Blue Tunnel Monthly Performance (December 2008-November 2009).

6.1.7 Mavi (Blue) Tunnel collapse and post grouting injection

Mavi (Blue) Tunnel is being excavated with a double shielded, 4.88 m diameter hardrock TBM. The hexagonal tunnel segments with peagravel annulus fill is being installed concurrently with the excavation. After approximately 3.022 m of excavation the TBM encountered a weak sandstone with poor stability between Polat Limestone and Ekinlik formation which was excepted on around 3500-3600th meter of the tunnel. (See figure 6.3 Geological section of Mavi (Blue) Tunnel). Expected Q index value for Ekinlik formation is between 0.025 and 1 as shown on the table 6.2. When attempting to excavate through this ground, several collapses occurred over the TBM cutterhead and shield. The collapses propagated up to 8-10 m height over the TBM.

The voids which formed above the cutterhead represented an imminent danger for a continued deterioration of the stability of the rock mass above the TBM. A temporary measure to fill and stabilize the voids was therefore needed.

This task could be broken down in the following elements:

- Filling of the voids over the TBM by means of injection
- Possibility for step-wise application enabling complete filling of the void from different working positions on the TBM
- Immediate application for resolving of this urgent and difficult situation.
- Avoiding of large volumes of un-reacted or unhardened grout.

Figure 6.12 : Excavated material just before tunnel collapse.

Chemical company providing technical support to the contractor suggested solving the problem by injecting polyurethane foam into the voids. But before that contractor tried unsuccesfully to solve the problem by hand mining as shown in the figure 6.13. Expanding polyurethane suggested by the chemical comapany would create a rapid and thorough filling of the voids, as well as consolidation of the rock debris which was resting on top of the shield and cutterhead. The rapid reaction of the foam would cause it to remain in place where intended, and not flow into the TBM.

Figure 6.13 : Hand mining operation after tunnel collapse in Mavi (Blue) Tunnel.

To be able to achieve the best result both one-component polyurethane and two component polyurea products were used as post injection. A two component high pressure pump was used for both products. PU injection has been executed to fill the voids and PUR resin injection was aiming mainly to stabilize the ground ahead of the cutterhead as shown in the figure 6.14.

Figure 6.14 : PU and PUR post grouting injection in Mavi (Blue) Tunnel.

In order to obtain the highest possible expansion factor of the polyurethane, water had to be added since the ground was completely dry. For the two component polyurethane water was added to component A (1% by volume), hence creating an instant foam when the two components were mixed. For the one component polyurethane, water was sprayed into the void through perforated plastic pipes. Hence, wet conditions were created and the polyurethane foamed (figure 6.15) instantly.

Figure 6.15 : PU injected as post grouting behind the segments in Mavi (Blue) Tunnel- Expansion reaction is ongoing.

In such cases speed is essential - Equipment and product were both on stock in Istanbul and were delivered at site 24 hours after the site management made their decision to inject.

Application was performed by the Ilci tunnel staff with support and supervision from personnel from chemicals supplier.

Figure 6.16 : Reacted PU coming through the openings.

Approximately 10 tonnes of polyurethane and polyurea was injected over the TBM. This gave a total foam volume of approx 150-180 m^3 (Figure 6.17). Hence, a significant void filling was achieved. The TBM was moved forwards and the segments could be placed without the debris caving into the tunnel.

Figure 6.17 : Foamed PU in front of the cutterhead.

6.1.8 Effects of Tunnel Collapse to General Performance of the TBM

Table 6.4 shows mavi (blue) tunnels excavation performance between December 2008 and Noveber 2009. It is very obvious that tunnel collapse decreased the general performance.

Table 6.4 : Mavi Tunnel Performance Data's (December 2008-November 2009)

Total Excavation (rings)	4002,5
Total Excavation Time (weeks)	49
Average weekly advance including time lost during tunnel collapse (rings)	81,7
Average weekly advance excluding time lost during tunnel collapse (rings)	88,9
Performance loss because of tunnel collapse	8,2%
Best Weekly Performance (rings)	160,5
Best Daily Performance (rings)	31
Best Monthly Performance (rings)	504,5

The machine set up gives the contractor the opportunity of doing probe drillling in case of necessity and urgency unfortunately it had not been used before the tunnel collapse.

6.2 Melen Water Supply Project; Bosphorus Crossing Tunnel , İstanbul, Turkey

6.2.1 General information about Melen Project

The Melen Project is expected to provide Istanbul with fresh water. The 150 kilometre water-supply pipeline will require 25 kilometres of tunnels to be engineered. Mosmetrostroy staff has been deployed at the section that lies under the Bosporus. A 3,400-metre long section is being engineered using the shield tunnelling technique. The work is being done using a six-metre Herrenknecht TBM, which was acquired specially for the project and is fitted with several boring units. The site geology had not been studied in sufficient detail; underground flooding is a hazard. This necessitates exploration drilling to a depth of 40 to 100 metres. If necessary, water will be pumped out and soil stabilized.

The tunnel is nearly straight, with a gradient of 7.45%. Its maximum depth is about 140 metres. The length of the tunnel is 3450 m and the geology consists sand, silt, gravel, limestone and quartzite. Excavation started on February 2008 and Construction of the tunnel has been completed in April 2009 succesfully [19].

Figure 6.18 : General layout of Melen Project.

After commissioning of the Bosphorus Tunnel (project Melen) in the European part of Turkey in Istanbul total volume of 37,5 m^3/s or 3,24 million m^3/day water will be supplied to the city.

Geological studies for the construction of the Bosphorus tunnel system Big Melen began in 1993-1995.(Figure 6.20) The first tender for the construction of the tunnel, announced in 1997, won a consortium of Turkish and German companies, but because of the high geological risks and the lack of research, the tender was canceled.

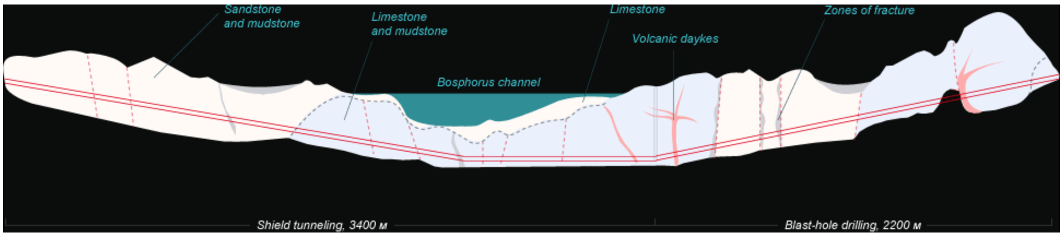


Figure 6.19 : Geology of Melen TBM Tunnel.

From 1997 to 2002 further research (exploratory drilling) were carried out, and then the alignment of the tunnel has been changed. The new tender, which was opened in 2002, won by the Russian-Turkish consortium Mosmetrostroy-STFA-Alke [18].

Table 6.5 : Technical Specification of Melen (Bosphorus Crossing) Tunnel HK Hard Rock Machine.

Machine	Herrenknecht S-391
Type	Hard Rock-EPB
Boring diameter	6150 mm
Minimum curve radius	600 m
Weight	Approx. 730 tons (with back-up)
Length	11.2m (TBM); 142 m (TBM+back-up)
Maximum torque	3519 kNm
Drive Power	2000 kW

Figure 6.20 : Herrenknecht S-391 Hard Rock TBM (Melen Tunnel) [19].

6.2.2 Exploratory (probe) drilling ahead of the Melen (Bosphorus Crossing) TBM

The main objectives of the exploratory drilling programme were the following:

- detect eventual zones ahead of the advancing TBM which contain permeable rock or weak rock which can represent a risk for uncontrolled water ingress or collapse at the tunnel face (risk of chimneying)
- provide sufficient information in order to decide at which location to stop the TBM and carry out an injection campaign
- decide if it's possible continue excavation without carrying out injections

The figures number 6.21 and 6.22 shows the layout of the exploratory drilling

programme. Figure 6.21 shows the case when there is normal advance without water in the exploratory drilling holes and the figure 6.22 shows the case when water with amounts over action criteria is encountered during exploratory drilling.

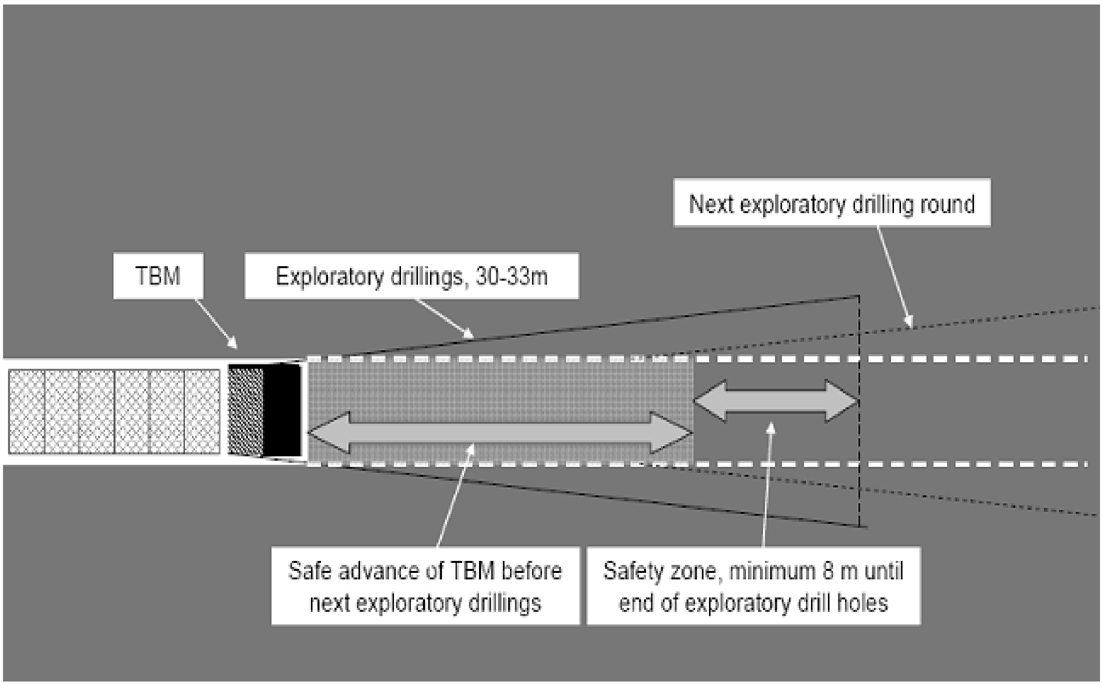


Figure 6.21 : Layout of exploratory drillling scheme ahead of the TBM (No water).

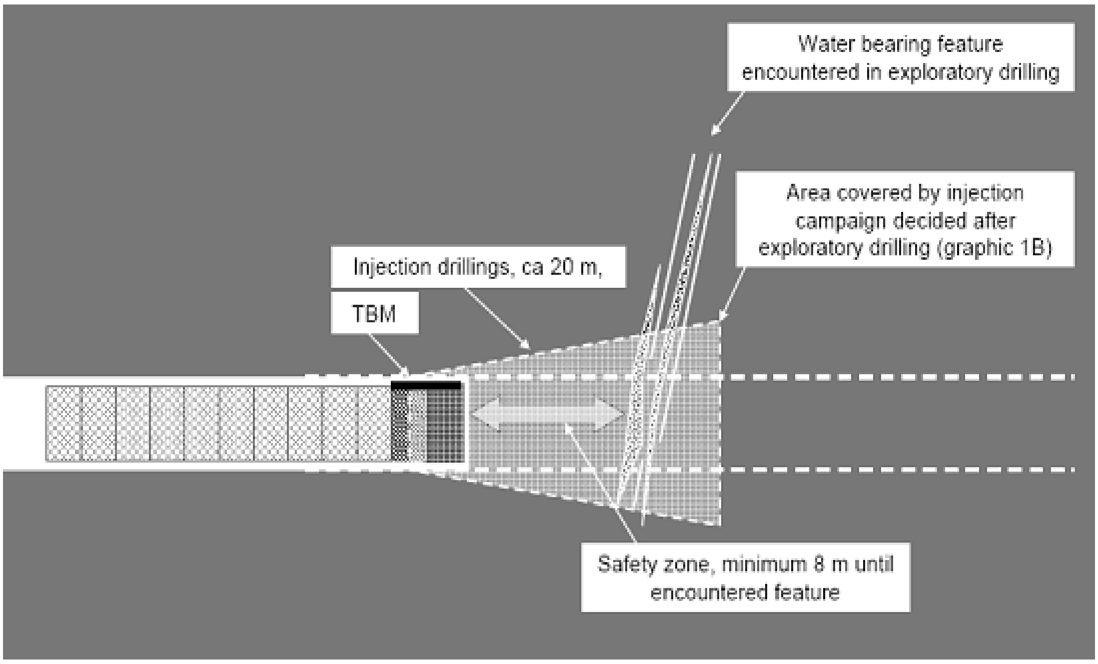


Figure 6.22 : Layout of exploratory drillling scheme ahead of the TBM (Water encountered).

The exploratory drillings should be performed as a mandatory, systematic and continuous measure for the entire tunnel length which is lower than sea level. The exploratory drillings will utilize percussive drilling (Figure 6.23) equipment which is installed just behind the shield of the TBM.

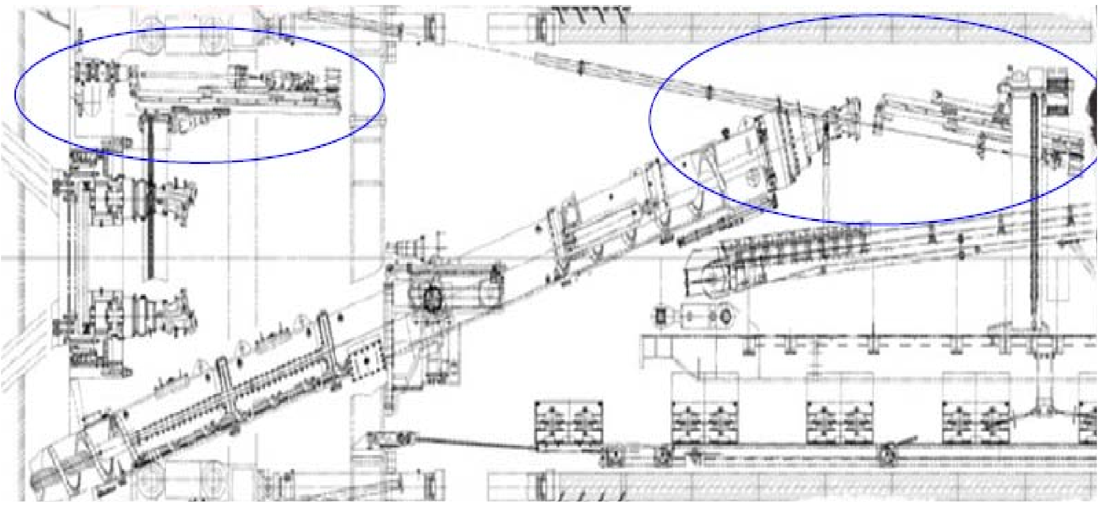


Figure 6.23 : Drillling equipment of Melen Tunnel Herrenknecht Hard Rock TBM.

With this equipment drillling holes with lengths ca 30 m ahead of the TBM was possible. It was decided to have a safety zone which corresponds to 8-10 m overlap between each exploratory drilling round in example. So for that reason probe drill to 33 m length ahead of the TBM has been executed, and then (if no water is encountered) excavate ca 18 m within a “safe” zone (bearing in mind that the drillholes will be collared ca 6 m behind the actual tunnel face).

It has been establish two degrees of “alertness” or levels for exploratory drilling, which again corresponds to the number of exploratory holes to be drilled.

Level 1: Two exploratory holes, located at “11 o’clock” and “1 o’clock”. Level 1 should be performed when good rock conditions are expected and the TBM is not located physically under the sea . Drilling length 33m.

Level 2: Five exploratory holes, Located at 9, 11, 1, 3 and 6 o’clock. Level 2 should be performed for the entire part of the tunnel which is physically located below the sea.

During exploratory drilling the following has been recorded:

- water ingress (measured in l/s or l/min) through the drillholes for every three meter drilling length (corresponding to the length of each drill rod section)
- eventual irregularity in encountered ground, e.g. abrupt change in drilling rate, difficulties during drilling, collapse or blocking of drillholes
- With 14 bar potentially maximum static water pressure, and small diameter holes (maximum 52mm drilling diameter) there shouldn’t be a need to have a preventer system as a standard measure during the drillings.
- For that purpose a system with expandable packers available with extension rods, which will be inserted into the holes after drilling has been chosen. All exploratory holes which have more than 3 l/min, were grouted before continuing excavation.

Figure 6.24 shows a decision flowchart for the exploratory drilling and injection works.

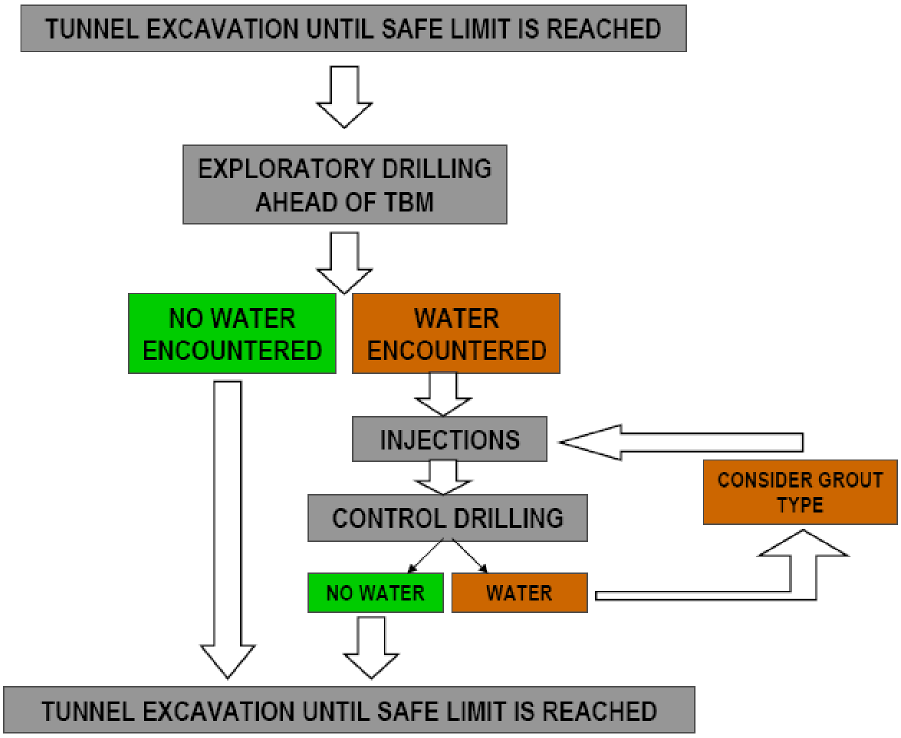


Figure 6.24 : Principal decision flowchart for exploratory drilling and injections.

6.2.3 Injections ahead of the TBM for water ingress reduction and improvement of poor rock

Injections ahead of the TBM has been decided when one of the following situations occurs:

- water is encountered in one or more exploratory holes, exceeding 10 l/min in one single hole,
- poor ground is detected (drilling difficulties, collapsing drillholes) in combination with water. (detailed criteria to be discussed)

When the action criteria for injection is exceeded, excavation of the tunnel should continue until a safety zone of minimum 8m to the encountered feature is reached. Then the drilling of injection holes could commence.

A very important issue was the termination criteria for the injection works, once they have started. The normal termination criteria when grouting a hole is pressure, e.g. 40 bars over static water pressure for cementitous systems, and 25 bars over static water pressure for liquid colloidal silica.

In the case the grout enters without increase of injection pressure, a maximum amount (kg/m drillhole length was pre-defined)

These criteria was evaluated and adjusted after an initial period of a few weeks when experiences with the actual achieved injection result are available at site.

6.2.4 Layout of injection drilling fan

The layout of the injection programme is shown on the figure 6.25 in detail.

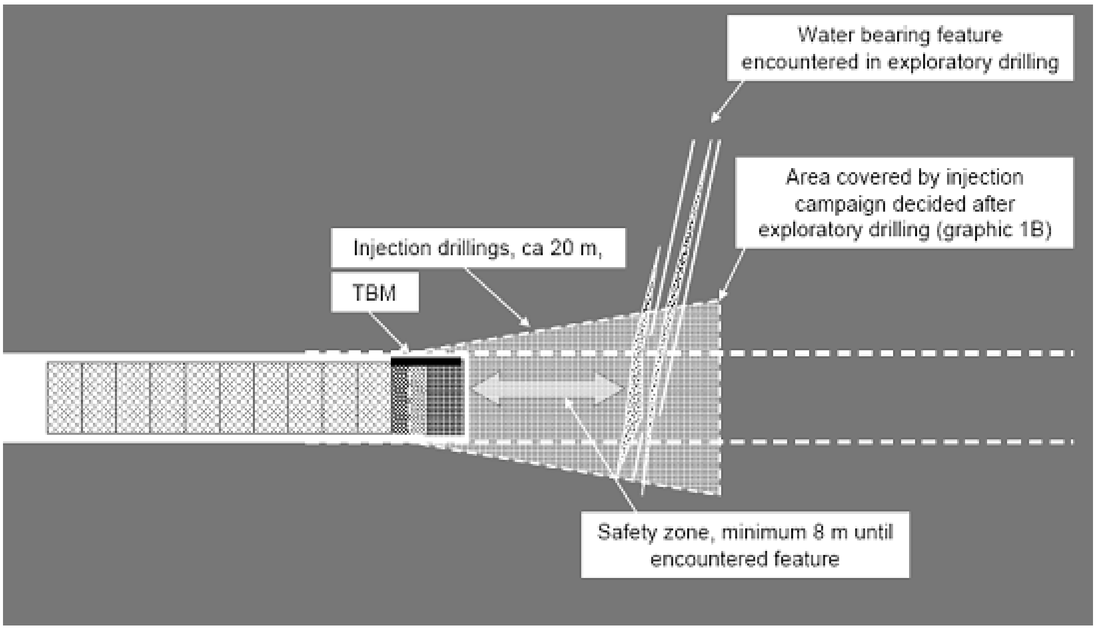


Figure 6.25 : Layout of the injection ahead of the TBM.

The length of the injection holes were not longer than 20m. If too long injection holes are drilled, one could easily looses control of the injection. The consequence is often that the effort spent has reduced effect.

It has been planned to start to drill a “limited” fan , e.g. 8 holes, carry out injection, then carry out control, in order not to inject more than necessary. If the control drilling reveals an unsufficient result, one can drill more holes and continue injection in the area which still has water seepage.

The injection works always started at the bottom (invert), and proceed upwards in the tunnel profile.

The control of the achieved injection result, and decision as to how to proceed was following the procedure for exploratory drilling (Figure 6.26).

Figure 6.26 : Herrenknecht TBM capable of executing injection drillings.

6.2.5 Pre injection grout mix designs

The grout mixes were aiming to achieve the following result:

- sufficient penetration into the seeping joints to reduce water ingress
- avoid too large spreading of the grout away from the tunnel if large joint openings
- reduce the time consumption as much as possible during the grouting campaigns

For keeping it simple the number of grout mixes were kept to a minimum. There was a simple and principal criteria for the selection of which grout mix to use. This was creating clear working methods with a minimum source of confusion and waste of resources. Table 6.6 shows main grout mix designs for Melen (Bosphorus Crossing) Tunnel.

Table 6.6 : Pre-injection grouting mix designs for Melen TBM Tunnel.

Grout Mix Design Number	Mix Design	Main Property	Main area of use
1	Portland cement, w/c ratio 0.8 + Dispersing agent 1,5%	Moderate to slow setting grout with moderate penetrability	When no rapid setting or high penetrability is required (e.g. grouting just before weekend, or during TBM maintenance breaks)
2	Microcement, w/c ratio 1.0 + Dispersing agent 1,5%	Rapid setting grout with high penetrability in fine joints	Injection fan grouting, control of injection result after 2 hours is desire (cement is set after 2 hours)
3	Mineral grout, liquid colloidal silica	Rapid setting low viscous grout with extremely good penetrability properties	Ground with difficult penetrability, extremely fine joints, or clay filled joints, weathered rock with clay. After the use of grout type 2, and Required result is not achieved

Grout mix number 1:

When time is not a critical issue during grouting, like the filling of leaking exploratory holes or control holes, a slow setting grout based on normal Portland cement may be used.

Grout mix number 2:

The main issue with this grout is the combination of technical performance (penetrability) and the minimum use of time. Special microcement used was designed especially for tunnel use, combining the penetrability of a microcement with rapid setting, also at low temperatures, as well as very low bleeding (giving a stable grout during injection).

Hence, in spite of a slightly higher material unit cost (compared to normal Portland cement), the total cost-effectiveness for the contractor will be significant. The use of this grout allows the contractor to perform immediate control drillings (two hours after injection) in order to verify the injection result, and hence determine if further injection works are necessary without delay caused by waiting for curing of the grout.

On the other hand except Microcements used in Melen Tunnel two finer grades were also available. (Rheocem 800 and Rheocem 900). But the contractor has chosen the most cost effective (the cheapest of the three) in this case.

Grout mix number 3:

This grouting system consists of mineral grout with extremely good penetrability properties. The use of this grout requires an initial injection round with grout mix type 2, in order to homogenize the ground. This grout would be favourable in cases when weakness zones with fine jointing and/or significant presence of clay (e.g. caused by deep weathering) is an issue. Under such circumstances repeated use of grout type 2 will not give sufficient results. One big advantage with this grout was that one can use the same equipment as shown in the Figure 6.27 as for grout 1 and 2.

Figure 6.27 : Injection pump used for pre-injection grouting in Melen Tunnel.

6.2.6 Quality plan for exploratory drilling and injections

The quality plan for the exploratory drilling and injection works was aiming at:

- assure that the correct decisions are made on a shift foreman level with respect to injections and safe advance of the tunnel with the minimum possible use of time
- record and report all measured/observed features during exploratory drilling and control drilling after injections
- record and report the details of the executed injection works

The quality plan was containing the following crucial documents:

- working instruction for exploratory drilling: shift foreman
- working instruction for grouting works, specificly emphasizing the termination
- criteria for each injection hole : shift foreman
- report form for exploratory drilling and control drilling after injections : shift foreman
- report form for injection works : shift foreman

6.2.7 Result of pre-injection grouting

Thanks to the very good planning and preparation from site personnel from Mosmetrostroy and support given by the chemical supplier Melen Tunnel TBM excavation started on February 2008 has been completed in April 2009 succesfully. (Breaktrough Figure 6.28 from Melen TBM).

Contractor did not faced any major water problems but execute regularly exploratory drillings and in very few cases injection drillings as described above.

Contractor injected OPC and microcements in very few cases to reduce the amount of water entering the tunnel. Furthermore he executed post injection with PU products behind the segments to give the chance to the backfill grout to set as well.

Generally eventual zones ahead of the advancing TBM which contain permeable rock or weak rock which could represent a risk for uncontrolled water ingress or collapse at the tunnel face (risk of chimneying) has not been detected.

In most of the cases water encountered in one or more exploratory holes, did not exceed 10 l/min (limit decided) in one single hole, so it was possible to continue TBM excavation without carrying out injections.

Figure 6.28 : Breakthrough Ceremony of Melen (Bosphorus Crossing) Tunnel.

7. CONCLUSION AND RECOMMENDATIONS FOR FUTURE APPLICATIONS

Dr. Nick Barton presented a paper with the title “The theory behind high pressure grouting” , printed in Tunnels & Tunneling International in September and October 2004 (parts 1 and 2) [4].

The paper clarifies on a theoretical basis the reasons why high pressure (50 to 100 bar) does not cause «damage» and why it also substantially improves injection results.

It is strongly recommended to study this paper carefully if further substantiation of the high-pressure approach is needed. Some of the most important aspects of the paper can be summarized as follows:

The use of 10 bar water pressure testing will under most conditions have negligible effect on the existing joint apertures.

This is one reason why such testing normally shows no correlation with grout takes and frequently is not executed at all as a routine test in tunneling.

The physical joint apertures E are larger than the theoretical hydraulic aperture e. This is easy to understand when considering that natural joints (E) have roughness and rock / rock contacts across the joint plane, but they still convey the same water flow as the smooth and completely separated walls at a distance e apart.

The physical joint apertures may open up between 10 and 50 micron close to the borehole, when 50 to 100 bar injection pressure is used.

This could represent an increase of E by 20 to 100 % and is of course important for creating grout permeation by avoiding joint entrance blockages by filtration. In the case of using an ultra fine cement with d95 = 10 micron, the joint opening would have to be > 40 micron for the grout to enter (according to the formula used by Barton). Permeation could then be achieved on joints with original apertures between 20 and 33 micron, due to the widening by injection pressure.

A very positive side-effect of high pressure grouting is discussed by Barton. The average ground stability or ground quality as expressed by the Q-value is substantial improved and this will influence cost and time of construction in a very positive way.

It must be emphasized already at this stage that post-grouting alone cannot achieve any of these results, unless extreme cost is accepted.

Because of this, post-grouting should only be considered a supplementary method to pre-injection.

High ground water static head, high ground water yield, excavation on a decline and other possible problem enhancing features, require that the following set of measures should be applied to reduce the problems:

Probe drilling ahead of the face on a routine basis must be executed.

The amount of pre-grouting must be balanced against the cost of pump and pipe installation and operation of the system. Pre-grouting and dewatering by pumping must both be used (these measures should not be viewed as exclusive alternatives, but must be utilized in combination).

Post grouting is difficult and time consuming and may become impossible.

Pre grouting, on the other hand, is simple and efficient, provided that a tight face area is maintained. A 5 m buffer zone is recommended in sound rock. In weak and poor rock more is required. High static head requires care and special measures. Do not allow high-pressure water too close to the face, particularly in poor ground.

Using the consumption of cement as the main pay item is equal to inviting problems. Even at high consumption of cement, the cost of cement is only about 5 % of the total Contractor's cost. It is obvious that such a relatively small and highly variable cost factor should not be the main basis for payment.

Depending on conditions, the added cost of pumping, probe drilling and pre grouting may be in the range of 50 to 100 % of standard excavation cost.

Finally: Keep the face area watertight and never blast or excavate the next round if in doubt.

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CURRICULUM VITA

Candidate's full name: Suat Seven

Place and date of birth: Istanbul 18/02/1974

Permanent Address: Bahcelere Giden Yol Sok. 15/8 Bostancı /Istanbul.

Universities and
Colleges attended: Yıldız Technical University