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Grouting Theory and Practice

E. Nonveiller

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PREFACE

Grouting started nearly 200 years ago as the ingenious idea of a French engineer, and implemented with extremely rudimental means, it was steadily improved and its use spread to become a standard method in civil engineering construction. Modern grouting techniques were definitely developed around the twenties of this century. It was treated for a long time as an art which eluded scientific investigation and improvement. Its performance was for some time more or less a privilege and a well protected secret of a select few specialized companies in France, Switzerland and Italy, which on the other hand contributed much to the development and improvement of grouting methods, means and equipment.

Rock and soil - the subjects of grouting - are extremely erratic formations; their physical properties vary in a wide range from place to place. Broad practical experience is necessary in order to extend a solution to handle problems such as the percolation of ground water and stored water through the foundations of small or large dams, the improvement of soil properties in cases of the deficient bearing capacity of foundations, and many other practical applications of grouting techniques in civil engineering.

Various specialities ought to be engaged when collecting the basic data necessary for design solutions of grouting problems and when implementing solutions to produce a grouted structure. Geodesy provides the morphological data of the location, geology, tectonics, hydrogeology and soil-and rock-mechanics the basic physical characteristics of the environment, they measure the relevant parameters which define them. Knowledge and experience are then essential to digest and collate all the resulting information and to prepare a design report according to which the work can be carried out on site. Skill and experience are very important when the project is being implemented on site.

Grouting has played an important role in the development of the hydroelectric potential of the Dinaric Karst in Yugoslavia after World War II, a region in which the geologic conditions are complicated and very permeable karstified limestone formations prevail. It was a professional challenge at the time to come up with the best solution upon which the success or failure of important projects for the development of hydroelectric power depended. The author had the privilege of becoming involved in these projects, especially in aspects of foundation, permeability exploration and grouting. Later he became acquainted with a number of grouting problems on many different sites. The material for this book is based on experience from work on grouting in the field, laboratory and office collected and put forth with the aim of presenting a survey of the problems and methods studied, and solutions applied in the author's own activity and in many other papers scattered thought Journals and Symposia.

A historical review of the development of grouting and improving of grouting concepts, materials and technology introduces the reader to the subject. The properties of the subject of grouting -

rock and soil - are described and methods of their evaluation are elaborated. The kinds and properties of materials used to prepare injected grouts and the relevant laboratory tests to establish them are then described. A brief chapter is devoted to a description of the essential characteristics of equipment for drilling, preparation of grout compound and grouting. Then individual chapters are devoted to the design and production of grout curtains in rock and soil, prestressing of natural and excavated slopes with grouted rock anchors, prestressing of tunnel lining and there is a chapter on lifting and levelling buildings by compaction grouting of soils. A glossary of terms used in the grouting branch is added for the readers convenience. The Bibliography contains a list of references cited in the text.

Credit for the excellent language editing of the author's text is due to Mrs. Ellen Elias Bursać and Dr.Vlasta Szavits-Nossan. The faultless computer set was typed by Mrs. Marija Štanfel, the illustrations were expertly drawn by Mrs. Emilija Novak. The author thanks them for their valuable contributions to the success of the work.

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E.Nonveiller

Zagreb, 9.6.1988.

1. INTRODUCTION

When presenting a less known and highly specialized branch of civil engineering such as grouting actually is, it is wise to briefly acquaint the reader with its basic characteristics and historical development.

Grouting is a procedure by means of which grout is injected into voids, fissures, crevices or cavities in soil or rock formations in order to improve their properties, specifically to reduce permeability, to increase strength or to lessen the deformability of the formations. In order to achieve the intended purpose bore holes are drilled into the formation, and then grout is injected under pressure until the voids around the injected section are filled to satisfy conditions specified in the design. Grouting has a wide application in modern civil engineering. It is applied:

- to reduce the permeability of formations under the foundations of hydrotechnical structures in order to control seepage and loss of stored water;
- to check uplift on the structure, or to prevent the danger of erosion of soil from the foundation;
- to increase the strength of material below the foundation of heavy structures, and or to reduce the deformability of the material in the foundation, sometime called consolidation grouting;
- to connect distinct structural elements in a homogeneous structure by injecting the seams between them with grout compounds, as in concrete gravity or in arch dams;
- to fix reinforcing cables in precast and prestressed concrete structures;
- to fix rock prestressing anchors;
- to lift and erect leaning structures and buildings;
- to fill voids between rock and tunnel linings, called contact grouting;
- for rehabilitation and reinforcement of old defective masonry on historical buildings;
- to construct underwater concrete by injecting voids of preplaced gravel in the „Prepact„ procedure;
- and many other applications.

The selection of the appropriate grout compound to be injected depends on the effect to be achieved and on the properties of the injected material to be permeated, it ranges from thin to thick suspensions and mortar of cement and fillers, to colloidal and chemical solutions and resins for grouting in voids of finely porous material. Grout is injected under pressure into a section of a bore hole drilled into the material to be injected until it fills a desired volume of material around the hole or until the maximum specified pressure is attained and a specified minimum grout flow is reached; these are called the saturation criteria. From injected watery suspensions, injected water is squeezed out in the process and the compacted mass of the injected compound fills the fissures and voids.

Grouting has nearly 200 years of history, a short period compared to that of the development of civil engineering, which started on the eve of human civilization. The inventor of the injection technique was the French civil engineer Charles Berigny (Glossop, 1960) who entered the Ecole Polytechnique in 1794. In 1802 he was engaged in the management of the harbor of Dieppe in which water had eroded the soil under the shallow foundation of the tide sluices built on gravelly material so that heavy undercurrents endangered the safety of the structures. In order to overcome impossible technical problems, the idea to repair the damages by grouting occurred to him as a real inspiration. First Berigny repaired the wooden sheet piles of the foundations. Then holes were drilled at 1 m spacing through the bottom of the sluice. Plastic clay was injected through the holes by means of what he called a „blow pump„ (pompe à percussion), see Fig. 1.1. It consisted of a wooden cylinder of 8 cm ID which he filled with plastic clay on top of which a wooden piston was placed. The clay was forced into the hole by blows of a heavy hammer onto the piston, and the procedure was repeated until the clay completely filled the voids between the foundation slab and the underground. The same procedure was used to fill voids between the foundation slab and underground with pozzolanic mortar.

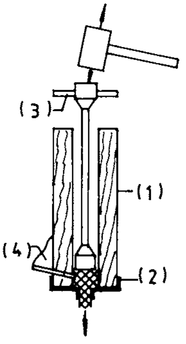


Fig.1.1. Injection „blow pump„ of Berigny, 1-wooden cylinder, 2-fitting for connection with hole, 3-wooden piston, 4-hole and plug to prevent vacuum when piston is pulled out

This first primitive application of grouting was a great success, and the restored sluice was put into service again. It was used once more for the foundation of a new sluice in Dieppe with a wooden raft with foundation wooden piles 12 m below spring tide level, completed in 1809. Berigny injected a suspension of pozzolana in the contact between the foundation and the ground in order to prevent erosion. This was the first application of the injection technique in foundation engineering history.

At that time underwater foundations were very difficult undertakings. Because mechanical pumps for dewatering were not available, they were the weak point of hydrotechnical engineering. After the successful application of grouting in Dieppe, the method was used to prevent erosion of foundations. Instead of clay, pozzolana and hydraulic lime, Portland cement was later used. In 1838 Collin (known for his pioneering studies on the stability of slopes) was the first to apply grouting to fill fissures in the masonry of the Grosbois Dam in France. To this purpose he constructed an improved version of the blow pump with a jack by means of which a watery suspension of cementing substance was injected under constant pressure.

An interesting application of grouting to reinforce the foundation of a bridge at Tours was described by Beaudemoulin (1839). The bridge with 15 arches of 25 m span each was built in 1765. The arch buttresses were founded on wooden rafts, five of them by the newly introduced method of wooden caissons. The history of the construction of this bridge was marked by many accidents involving foundation problems. Four arches collapsed during a flood in 1789, the reconstruction of which lasted until 1802 when the bridge was put into service again.

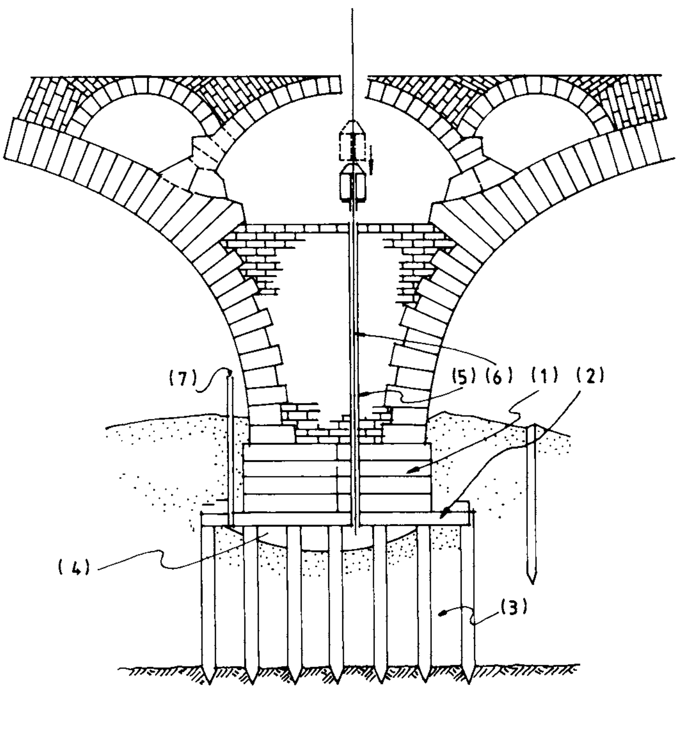


Fig.1.2 Section through pier of bridge at Tours, 1-masonry pier foundation , 2-wooden raft foundation, 3-piles, 4-void bellow foundation, 5-injection bore hole, 6-piston with ram, 7-draining hole

Large voids were discovered by drilling trough the buttresses under the caissons. A slurry of hydraulic mortar was poured into the voids below the foundation through injection holes until they were completely filled. Then a piston on a metal rod was inserted and pushed down by blows of a 150 kg ram, in order to consolidate the injected slurry. Beaudemoulin's observations from this work are of some interest:

- auxiliary holes should be drilled in order to allow water to be drained out from the slurry;
- the thick slurry should not mix with the water through which it is injected, it should not shrink and should harden slowly;
- steady pressure better consolidates the injected slurry than shocks exerted by the blow pump;

which shows that he had already grasped the basic requirements for successful grouting.

In the USA the „blow pump„ was used for the first time in 1845 when W.E.Worthen injected cement mortar under the foundation of a spillway chute and in 1854 to reinforce the masonry of a pier. Until that time grouting was used only as a remedial measure for defective foundations and structures, and not as a construction method.

Grouting started in Great Britain much later. W.R.Kinniple experimented in 1856 with grouting gravel. In his long lasting career he often used injecting clay and cement mortar with a device similar to Berigny's „blow pump„, among others for stopping underseepage under the Damietta and Rosetta Dams at the mouth of the Nile in the Mediterranean in 1886. It is interesting that this is the first instance of producing underwater concrete by injecting cement mortar into preplaced gravel for the construction of several dams and ship locks on the Nile.

P.W.Barlow obtained a patent for the construction of tunnels in 1864 by means of a shield in which injecting cement grout was provided for behind the lining. The method was frequently used for the construction of tunnels under the ground water level in the London Tube and in the Paris Metro. Injecting grout with compressed air from a conical pressure vessel was introduced at that time, the practice of which remained in use for a long time. It was only about 1910 that the first injection pump was constructed; it was a hydraulically driven membrane pump.

Further development and a wide application of grouting was in the construction of mining shafts, where it was used to protect the excavation from groundwater infiltration and seepage because of great difficulties involved in pumping from great depth. At a shaft in Bethune (France) some kind of permeability measurement of the formations was introduced as early as 1904 in order to direct injections. When the flush water loss during exploratory drilling exceeded more than 200 l/min, drilling was interrupted and the hole was injected with cement grout. In some cases differently colored cement was injected in different holes so that the grouting paths could be observed during excavation of the shaft.

Registering pressure gauges were introduced about 1910 from which records the character of grouting was studied. Experience was gradually accumulating with grouts of a different composition, with the development of the injection pressure, the diameter and the spacing of injection holes. At the first scientific congress of the society of engineers graduated at the Ecole de Liege in 1902, Fransois stated that it is better to grout from many holes of small diameter, and that a high injection pressure in the range of 50-200 bar is more efficient than the low pressure of 15-20 bar then in general use.

Grouting for dam foundations was applied only as a remedial measure in case of excessive seepage under the dam foundation, and some success was attained in such instances especially in the USA. Toward the end of the 19th century the study of permeability and fissuring of the foundation rock was started to which purpose some ingenious but primitive contrivances were invented to isolate the respective section of the exploration hole, such as small bags filled with seeds which swell in water placed around the injecting tube, or injection of water with bran which would signify that the fissures are narrow if the flow diminished.

Grouting as a method for reducing permeability was still regarded skeptically in Great Britain in 1911 although Kinder in that year provided foundation grouting as a design measure for a dam foundation. Lapwood (1911) maintained that grouting was a last possibility and that in the design of dam foundations any other possible means ought to be considered to control seepage and uplift, because grouting was based on guesses alone. It was Gourley (1922) who concluded from a detailed study of hydrotechnical projects on which a great volume of grouting was carried out, mainly in the USA, that such an opinion should be revised in order to reduce cost and delay in construction of dams without sacrificing tightness and safety.

Until 1930, grouting was carried out in the USA on 19 large dams. It was a time when dams were mainly constructed on sites with competent rock foundations, at a time when among good sites the best ones could be selected for construction. Today, when almost all good sites are already developed, we have been left with only bad ones, and technology has to be advanced in order to be able to develop them as well.

Systematic improvement of grouting techniques started with the injection of foundation rock at the Hoover Dam on the Colorado River in the USA which was constructed from 1932 to 1936 at Boulder. A grout curtain was injected and consolidation grouting on the dam foundation was carried out in order to compensate for cracking caused by the foundation excavations. Specifications for design and construction of grouting works were then prepared, based on experience gained at Hoover Dam injection works.

Grouting was developed independently in France for dam foundations and it was used for the first time at the Chavanon Dam in 1934. Through systematic studies a relation between development of pressure during injection and a gradual decrease of permeability was established as well as the importance of continuous registration of injection pressure development.

While the practical application of grouting fissured rock was solved, the injection of gravelly granular soils still remained a problem. Although Berigny and Kinniple already attained some practical results in grouting gravels, the effect was uncertain and erratic. Difficulty arises because larger suspended particles block the narrower flow paths between the grains limiting the reach of grouting.

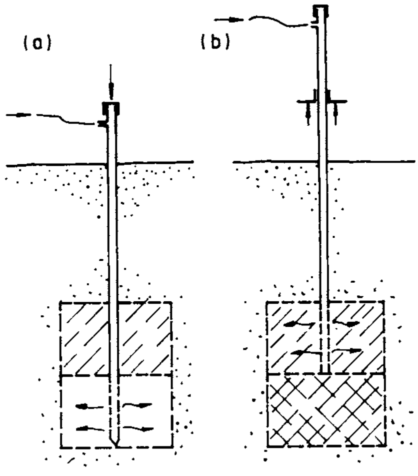


Fig 1.3. Grouting sand beds by the Joosten method, (a) driving the injection pipe, perforated at the bottom, (b) injection of salt and withdrawal of pipe in stages

When the pressure is raised, lenses of grout develop around the hole by hydraulic fracturing, the ground is lifted and the grout compound is uselessly wasted. Blockage of flow paths between the soil grains is avoided when colloidal or real solutions are injected. It was first introduced by Joosten (Denmark). A solution of water glass was first injected trough a driven perforated pipe then a solution of salt which caused the formation of silica gel in the voids where the two solutions mixed. The pipe was gradually withdrawn as illustrated in Fig. 1.3 and a column of grouted soil of about 1.0 m in diameter was injected.

By repeating the procedure at 1.0 m intervals, a required volume of soil can be injected. The permeability of the soil is thus reduced and its mechanical strength is enhanced. Later this two-solution procedure was replaced by injecting only one solution with the required reagent to induce delayed gelling of the solution after it is injected into the ground. Such processes are now in general use.

In order to permeate the required volume of soil uniformly it is necessary to be able to reinject the already grouted ground evenly with different compounds to achieve as uniform results as possible in nonhomogeneous ground. This problem was solved by the invention of the sleeve grouting technique, introduced by Mr. Ivry of France, which will be described in detail in Chapter 4.5. One of the early projects where this method of grouting up to 100 m deep alluvial beds was used on a large scale, was the foundation of the 130 m high Serre Ponçon Dam on the Durance River. Also the 130 m deep sandy and gravelly alluvial formation of the Sadd El Aali Dam on the Nile was successfully grouted with this method.

The papers by Glossop (1960, 1961) bring much detailed information on the history of grouting and its successful application from the end of the last century with interesting citations from original papers published at that time.

The first reported grouting works in Yugoslavia date from 1932 when the Grošnica Dam was built near Kragujevac.It is a 42 m high concrete gravity dam completed in 1938. From scanty data available it is only known that 50 mm diameter holes were drilled from a gallery into the dam foundation and cement suspension was injected; no details about the procedure and the extent of works and the cement consumption are known.

After World War II the industrial development of the country required an intensive development of the hydroelectric potential of the Dinaric Karst along the Adriatic Coast with dams and reservoirs in the very pervious karstified limestone formations. Extensive studies were undertaken with field explorations and laboratory studies which led to the successful construction of the storage reservoir on the Cetina River with the 60 m high Peruča Dam (see Chapter 6.7). This experience enabled the construction of storage reservoirs in the karstified region with a total of more then three billion cubic meter capacity. Up to date more then sixty dam foundations were injected in the country and Yugoslav grouting specialists have gained experience and reputation appreciated in the country and abroad.

2. PROPERTIES OF SOILS AND ROCKS

Rocks are all materials from the surface of the earth. They include water, ice, loose sediment, sedimentary, metamorphic and igneous rocks. In connection with grouting we distinguish soil and rock.

Soils are loose geologically young and recent sediments - cohesionless and cohesive - which consist of agglomerations of solid mineral particles which coalesce under forces which depend on the local stress tensor. Their porosity n (volume of pores over total volume) may be in the range between 0.15 and 0.50.

Soils are granular media consisting of randomly arranged particles of different size and shape which are mutually in contact, leaving between them interconnected voids of varying shape and size. The size of the voids depends on the relative content and size of the smaller grains in the mix and the maximum stress to which the material was exposed since its deposition. The porosity and the size of voids govern the permeability of the medium.

Rocks are solid materials the elemental particles of which are bound together by permanent diagenetic, crystalline and molecular intergranular forces which are independent of the local stress tensor. The porosity is generally low in the range of 0.01-0.10.

Rock is a more or less homogeneous mass with very low porosity. Since the voids are mostly isolated and do not afford a net of interconnected paths through which water can percolate, its permeability is generally negligible. The geologic age of rock ranges from the oldest archaic to youngest sedimentary formations. These were all subjected to tectonic forces causing bending and faulting which repeatedly caused fracturing, jointing and fissuring so that today the masses of rock in foundations are fragmented, fissured and jointed to different degrees forming discontinuous masses of fragments divided by systems of bedding, jointing and fissuring planes. These discontinuities may be tight in some cases, open and void or filled with debris and sediments conveyed into them during later geologic periods. These discontinuities present "voids" in rock through which water can percolate and which define the permeability of the mass.

Within each of these two groups of material with basically different properties regarding grouting there are materials of variable density, strength and deformability. The main difference between these two categories is the character of their porosity and of the way water and grouting material percolate through them under some pressure gradient.

In order to establish the properties of soils and rocks for the design and construction of grouting works, so that the appropriate procedures and materials can be devised in any particular case, comprehensive exploration and investigation is needed. The study of examples of works carried out in similar geologic conditions and materials is a valuable guide for planning of exploratory works and for a better evaluation of their results.

The ***permeability*** of the formations is the basic parameter for the computation of the equipotential field which ensues in the foundation of dams when water is being impounded. It is also basic for the design of the grout curtain needed to prevent loss of water through the foundation and possible negative effects of water seeping through the foundation material. The permeability of rock formations in which water circulates though joints and fissures of variable width and extension, which are open, partly empty or filled with granular or cohesive material, is expressed in Lugeon units (LU). One LU is defined in liters per minute of percolation through a 1 m long section of a test hole under a pressure of 10 bar.

The permeability of granular soils is generally expressed by means of the Darcy coefficient of permeability $\boldsymbol{k}$ in (cm/s).

The kinds and the amount of exploration works for a grouting project must be planned so that the most suitable site can be selected and data are obtained which are needed to design a safe foundation of the structures. Site exploration should encompass:

- geologic mapping, stratigraphic and lithologic data of the foundations, geosynclinal or epicontinental spaces, establishing the geologic structures of the site and the analysis of tectonic processes and units (Herak, 1976);
- geophysical measurements which provide global data on electric resistivity and the propagation velocity of elastic waves in the region;
- excavations, trenches and adits which provide a direct insight into the kind and composition of the materials, the sequence of strata, their lithology;
- drilling exploration holes from which cores are obtained for geologic and lithologic determination of deeper strata and of tectonic relations;
- in situ testing of deformability and permeability of the foundations.

Whenever grouting is envisaged, the site exploration program should also establish data which are needed for the design such as:

- survey and exploration of the system of discontinuities in the foundation as bedding planes, joints and fissures, as well as the materials filling the discontinuities;
- careful determination of the permeability by means of water pressure tests,
- investigation of hydrogeologic conditions and the characteristics of the ground water table;
- exploration of wells, springs, estavels, ponors etc. in the region and their capacity.

Such a complex exploration program should be prepared by a team of experts of various specialities including geologists, hydrogeologists, foundations and grouting experts. Starting from existing data the problems which may arise with the planned construction should be studied and the data needed for their solution established. From these studies the need for the kind and volume of site exploration, investigation and other studies can be planned.

Site investigations should be carried out in a systematic sequence, the results must be studied and analyzed as they become available so that new problems are detected early and the works are amended as needed to obtain complete data at a rational expense.

2.1. Geology of the site

The conditions for establishing a water-tight foundation below a dam structure and other scope of grouting are defined by the geology of the site and its tectonic structure, the permeability and the physical characteristics of the site. The comparison of results and experiences obtained on completed projects on other sites in comparable similar geotectonic conditions will enable to anticipate possible problematic areas and to guide planning of exploration works.

Characteristics and impact of overthrusts, folding, of synclinal and anticlinal features, several types of faulting and seepage paths have been studied on many examples of existing reservoirs and dams. A wealth of data exists in the archives of design organizations, some of it was also published in reports and papers from international congresses and symposia on geology, hydrogeology, large dams, foundations, grouting and on special mining problems.

2.2. Percolation of underground water and permeability of foundations

The Laplace analytical solution of the differential equation of water flow in porous media with defined boundary conditions is the base for the computation of the pressure head of water percolating through a given region in homogeneous, non-homogeneous or anisotropic material. Physical models are based on this solution, such as the hydraulic, electric and viscous fluid analogies as well as numerical or FEM simulations of flow by means of computer solutions.

The model of a porous medium applied to fissured rock presents a very crude approximation of the real conditions. Nevertheless it is attractive for the study of percolation through rock, because the application of simple mathematical models results in a rough simulation of the real spatial distribution of potential head, in the space, of pressure gradients at critical spots and of the volume of seepage flow.

With the assumptions that:

- percolation of water in the void spaces is laminar and Darcy law applies, the velocity of flow is given by

$$v = k\, i, \tag{2.1}$$

v = average flow velocity through a volume, (m/s)

k = coefficient of permeability, (m/s)

i = pressure gradient, $i = dh/dl$,

- water is incompressible,
- flow is stationary,

the Laplace differential equation of a second order is obtained:

$$k_x\, \partial^2h/\partial x^2 + k_y\, \partial^2h/\partial y^2 + k_z\, \partial^2h/\partial z^2 = 0 \qquad (2.2)$$

in which:

$k\,(x,y,z)$ is the permeability coefficient in the direction of coordinates (x,y,z),

h is the potential head in (x,y,z),

(x,y,z) coordinates in three orthogonal directions.

Under consideration of the boundary conditions the solution of the above equation yields two sets of curves:

- the equipotential lines $\Phi = \Phi(x,y,z) = \text{const.}$

(2.3)

- the flow lines $\Psi = \Psi(x,y,z)$

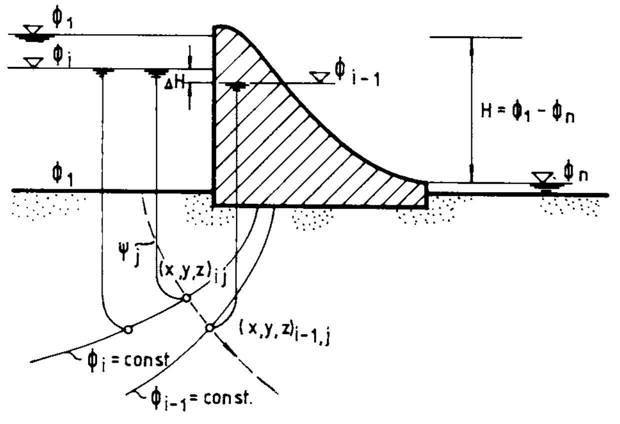


Fig. 2.1 Equipotential and flow lines in foundation below a dam

The solution is presented graphically *by a flow net as shown in Fig. 2.1. The set of equipotentials* Φ_i and flow lines Ψ_i presents the distribution of the function of the potentials and the direction of flow at any given point (xyz) of the domain. In Fig. 2.2 the example of a planar flow net around an impervious sheet pile with impounded water, and in Fig. 2.3. of a flow net in a nonhomogeneous layered foundation of a dam with a grouted curtain is shown (Nonveiller, 1970).

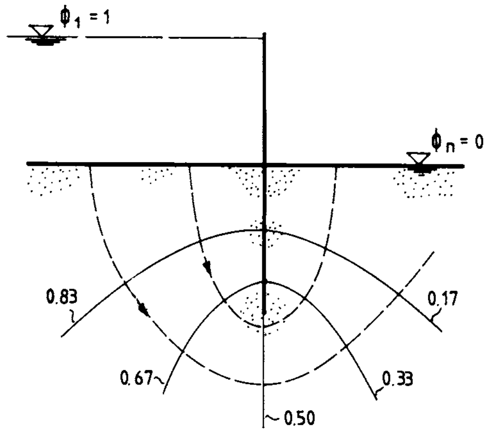


Fig 2.2 Flow net in homogeneous soil around a vertical sheet pile wall

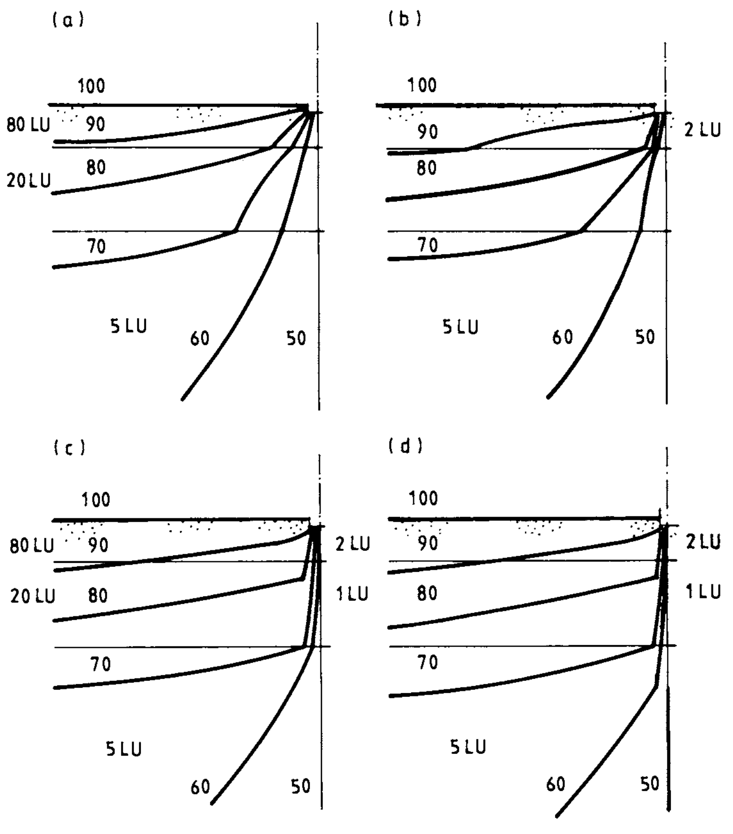


Fig. 2.3 Flow net below a dam on horizontally stratified soil, (a) without grout curtain, (b) with curtain through first layer, (c) with curtain through first and second layer, (d) with curtain 25 m deep into third layer

There are solutions for plane and axi symmetric problems in a homogeneous isotropic material with simple boundary conditions. Such solutions are of little value for practical cases in which the material is generally nonhomogeneous, anisotropic and the boundary conditions are complex. For such cases numerical solutions of Eq. 2.2 are used either with finite differences or with finite elements. There are several programs available for such computations.

Fractured and fissured rock is substantially different from porous media. The permeability of the systems of bedding planes, joints and fissures is many orders of magnitude greater than the permeability of the solid rock matrix between them. It can therefore be supposed without much error that water percolates the domain only through the discontinuities it contains. Wittke and Louis (1968) have developed an analytical procedure for the computation of the potential function Φ in a medium containing one or two sets of parallel equidistant fractures of equal width. The factors governing the flow of water through fissures are:

- direction and position of discontinuities;
- their spacing;
- degree of fracturing of the medium;
- width of fractures;
- roughness and shape of the walls of fractures;
- material filling the fractures;
- permeability of the soil between fractures.

The direction and position of the fractures is determined by the dip angle α and the direction of strike. The spacing of the fractures and the degree of fracturing are determined statistically from measurements of the distance between them and the width of the fractures in different sets of fractures.

The roughness of fractures is defined by the average deviation from the plane as shown in Fig 2.4. It is defined by the following parameters:

roughness $K = (i\,\Sigma K_i)/n$

$$\tag{2.4}$$

relative roughness $\varkappa = K/R_h$

in which:

$R_h = a$, is the hydraulic radius,

2a is the average width of the fissure.

In order to obtain reliable values of these parameters very extensive field work in needed, the reliability of the parameters is proportional to the ratio of the number of measurements to the volume of rock involved.

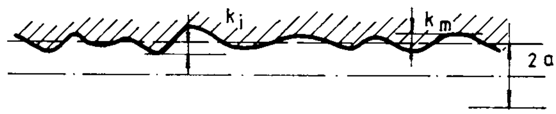


Fig. 2.4 Notations for computation of relative roughness of fissures

The hydraulic gradient of water flowing through a fissure with an average velocity v can be expressed as:

$$i = \lambda v^3 / \nu R_e .2g \tag{2.5}$$

in which the Reynolds number is

$R_e = 2 R_h v / \nu$

ν - kinematic viscosity of water (cm/s)

λ - coefficient of hydraulic resistivity

The resistivity coefficient λ depends on the roughness of the fissures, on the kind of percolation - laminar or turbulent - and on the Reynolds number Re.

The relation between the relative roughness $\varkappa$ and the Reynolds number was established by Wittke and Louis (1968) from an analysis of model tests as shown in Fig. 2.5

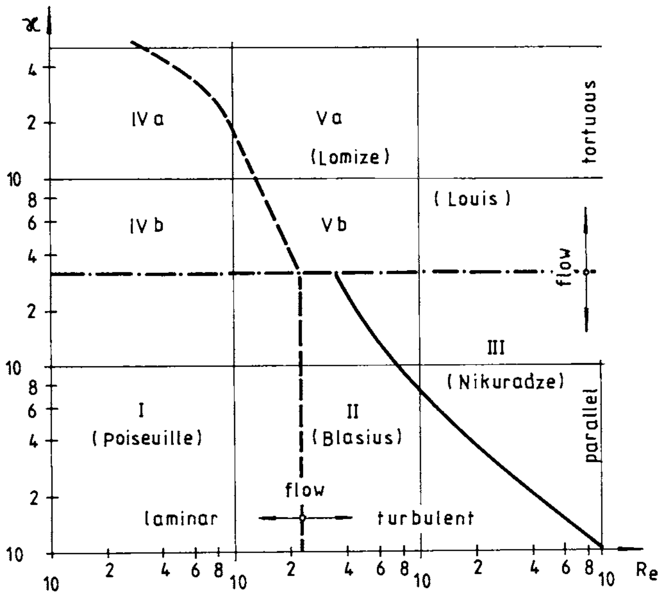


Fig. 2.5 Diagram for computation of coefficient of relative roughness

The authors have developed a numerical and a graphical solution of the potential function Φ for given boundary conditions, satisfying the condition of continuity of flow which requires that at any intersection of fissures the quantity of water flowing in and out of it should be zero ($_i\Sigma\, q_i = 0$). The condition that the sum of head losses in any closed loop should be zero ($_i\Sigma\, l_i\, i_i = 0$) is also satisfied. With these conditions a system of equations is obtained from which the head potential Φ is obtained on all intersections of the fissures. Since the character of flow in the fissures, and the corresponding coefficient λ are not known in advance, an iterative approach is used to obtain the solution. In the first iteration, laminar flow is assumed in all the fissures, which is subsequently adjusted to turbulent in the corresponding fissures. The iteration is continued until the computed flow and the corresponding R_e are in agreement in all the fissures in two successive iterations.

The authors have verified this procedure on the model of a tunnel which drains a fissured domain with one and two systems of fissures. The size of the model was 100x40 cm, and the diameter of the tunnel was 20 cm, as shown in Fig. 2.6. The model was made of a plexy plate, and 2 mm wide fissures were cut into the plate.

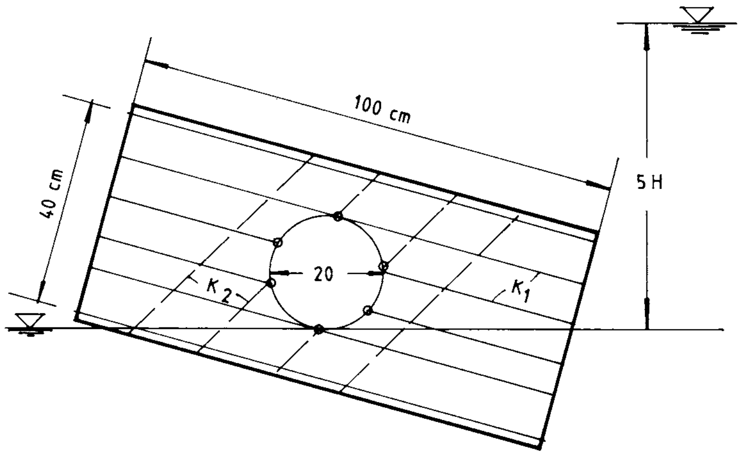


Fig. 2.6 Model of tunnel in fissured rock

The tunnel was simulated by means of six holes drilled on the intersection of its periphery with the fissures. The potential head was measured at 52 locations in the fissures. The results are shown in Fig. 2.7 (a) in the case of a single system of fissures, and in Fig. 2.7 (b) in the case of two systems of fissures. The equipotential lines computed by the authors as explained above (Wittke, Louis, 1968), are also shown in Fig. 2.7 (b). It is evident that in some points the computed values differ significantly from the experimental ones.

Since the application of the sophisticated computation by Wittke and Louis (1968) is based on data and assumptions which can only roughly simulate actual field conditions, it is interesting to compare these results with those obtained assuming the rock to be homogeneously pervious, and using Laplace's theory for the computation of the potential function Φ.

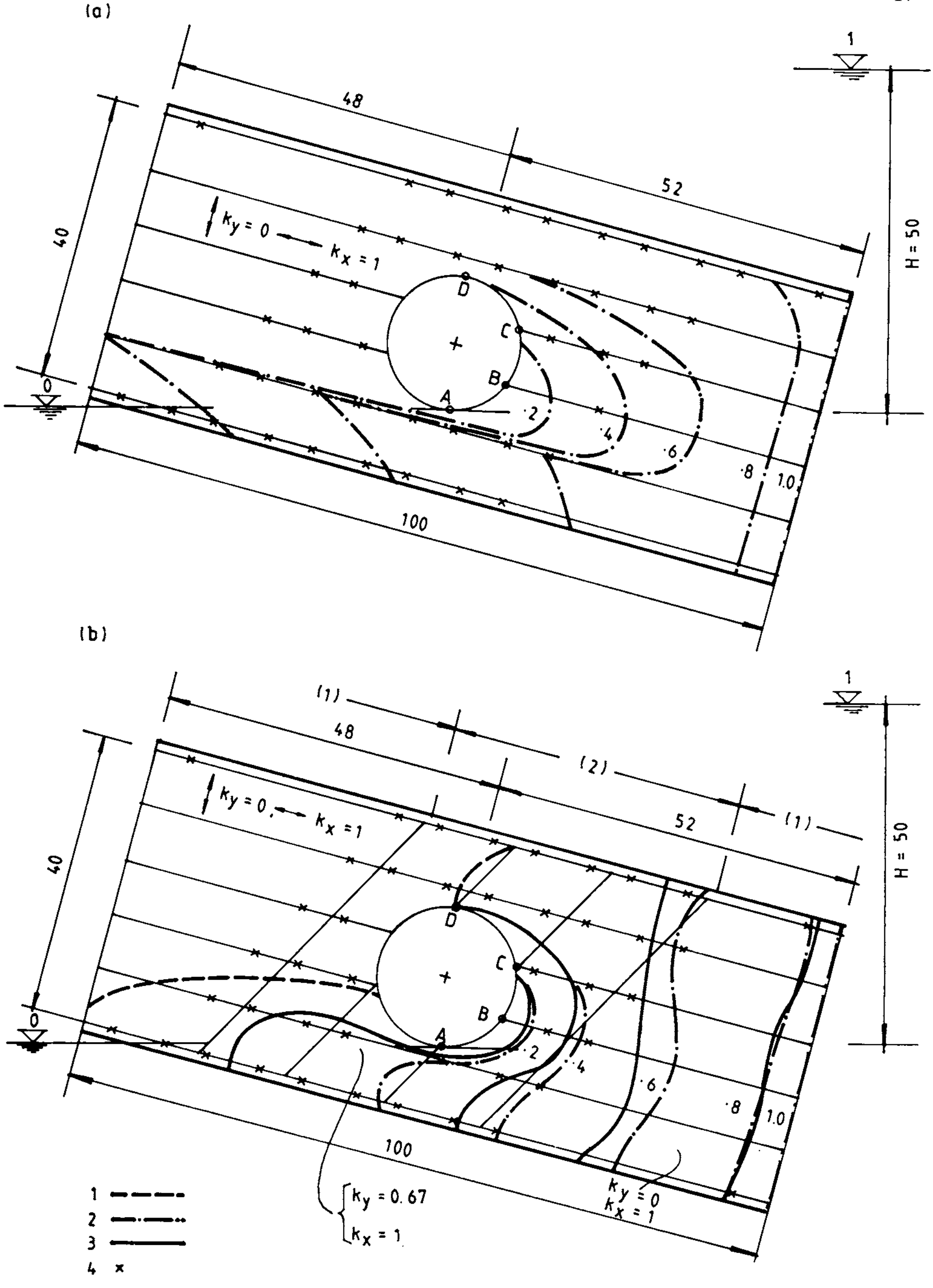


Fig. 2.7 Equipotentials of flow around a tunnel in fissured rock after Wittke at al. (1968): (a) only main system of fissures permeable, (b) main system of fissures permeable only in zone (1), both systems permeable in zone (2), 1 free water surface, 2 equipotentials from the theoretical computatiòn, 3 equipotentials from the model test, 4 points in which potential was measured,A,B,C,D, points where flow was measured (dimensions in cm)

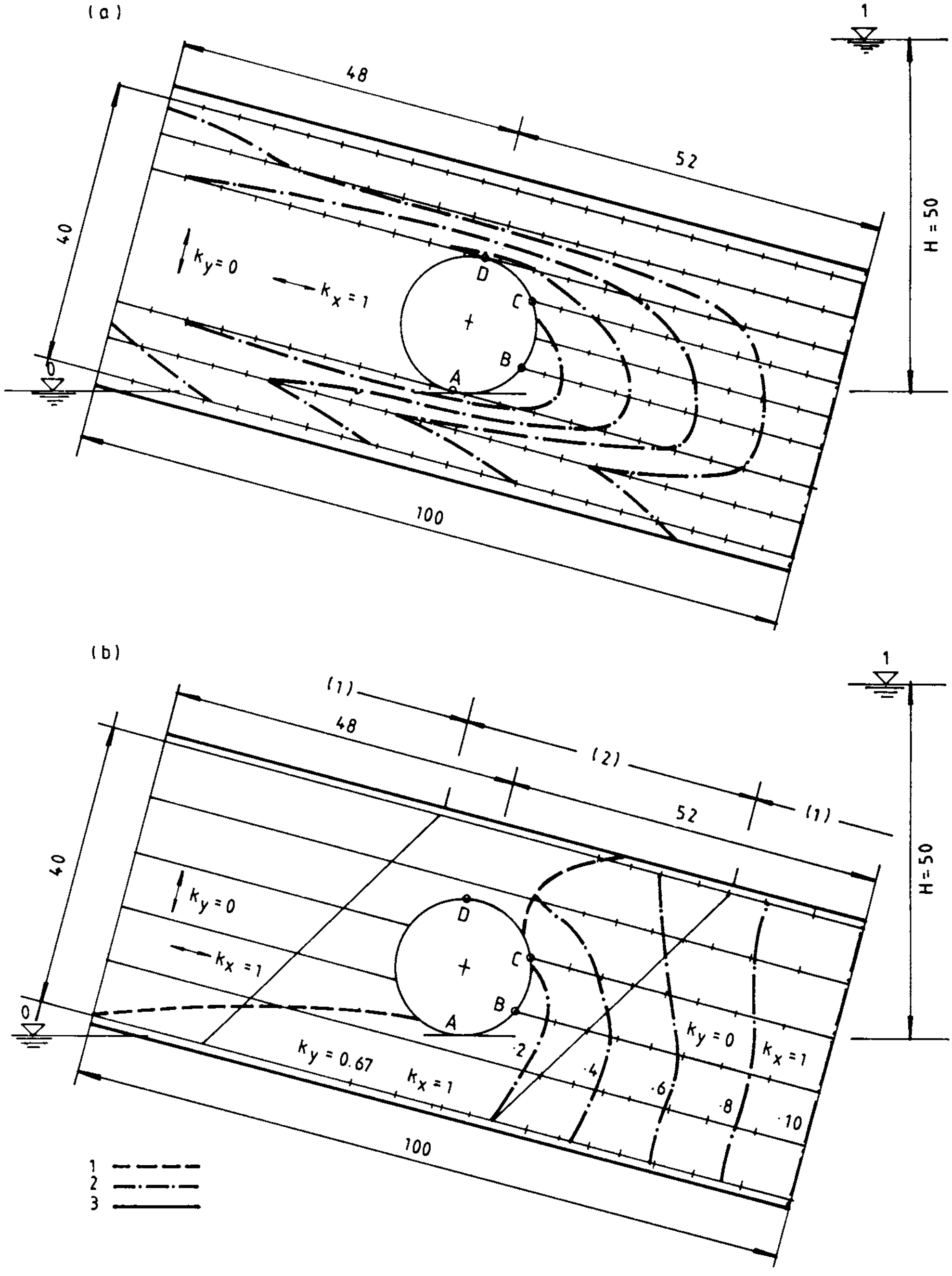


Fig. 2.8 Equipotentials of flow around a tunnel in homogeneous anisotropically permeable ground from FEM computation, (a) region permeable only in x direction, (b) boundary zones, zone 1 permeable in x direction only, zone 2 permeable in x an y direction, 1 free water surface, 2 equipotentials, 3 nodes of FEM net , A,B,C,D points where flow was computed

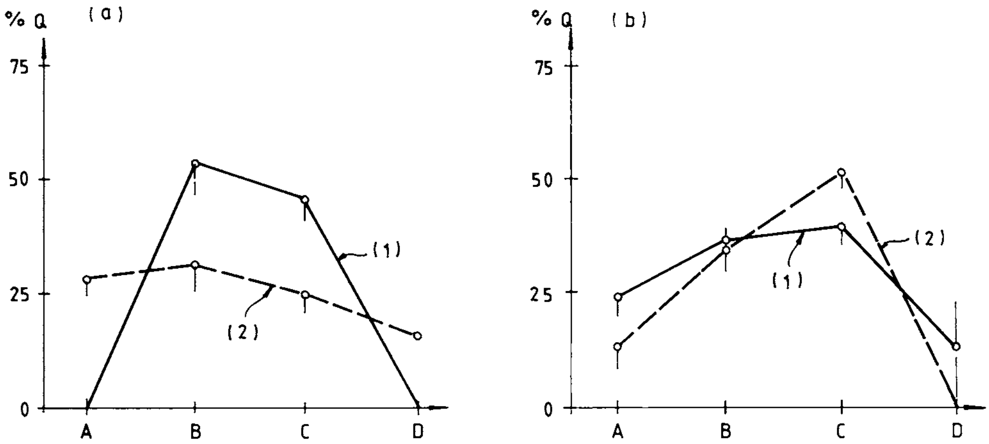


Fig. 2.9 Comparison of relative flow rate at points A,B,C,D from model test and from FEM computation, (a) for examples in Figs. 2.7(a) and 2.8(a), (b)from examples in Figs. 2.7(b) and 2.8(b), 1 measured in the model, 2 from FEM computation

This is obviously a less sophisticated approach, but it requires less parameters: permeability and its anisotropy in the characteristic regions of the domain, which in practical cases can be more readily established.

For this purpose the example shown in Fig.2.7 was analyzed using a FEM computer program for an anisotropically pervious porous medium as shown in Fig. 2.8. The resulting equipotentials are shown in Fig. 2.8 (a) for the case with one system of fissures and in Fig. 2.8 (b) for the case of two systems of fissures, analogous to the cases of Fig. 2.7 (a) and (b). By comparing the respective equipotential lines it is evident that they do not differ one from another as much as it could be expected from two theoretically different approaches.

It is also interesting to compare the relative flow rates at points A, B, C, D, of the tunnel perimeter shown in Fig. 2.9(a) and (b) respectively, which are similar.

Accounting for all the aforementioned facts about the reliability of the input data for such computations, it can be concluded that for the solution of practical problems computations based on Laplace's equation may be applied. The nonhomogeneity of the domain resulting from different geologic and lithological conditions, and from the of the permeability anisotropy of different zones, which can be accurately established from geologic studies and from the results of drilling and permeability measurements in the boreholes, can easily be accounted for in the computation. The reliability of such an approximation of a large domain will obviously be better than that shown in the above example which treats a limited domain with rather complicated boundary conditions.

The existence of wider particularly permeable zones such as crushed fault zones, open channels and caverns in the domain, must be adequately considered when establishing a mathematical model for the study of the behavior of ground water percolation after impounding a reservoir, as well as of the effect of a grouted zone on it. It is very important that such particulars are well defined and described during the exploration stage, their permeability adequately measured so that they may be considered in the model. Such irregularities often act as drains of high capacity which dominate the percolation and which may prove difficult for efficient grouting (see examples in Section 6.6. Sklope, Buško Blato).

2.3. Field measurement of rock permeability

The permeability of rock and soil is measured in the field, except for formations of very fine grained granular sedimentary strata where laboratory tests on small undisturbed samples can be useful.

The percolation of water flow in the measuring section of a bore hole in jointed and fissured rock under an imposed pressure depends on the distribution and interconnection of the systems of bedding planes, joints and fissures which are cut by the measuring section. It depends also on the permeability and stiffness of materials contained in some or in all fissures, and its resistance to erosion. The result is expressed in Lugeon Units as mentioned previously.

In granular materials the circulation of water in the voids which form flow paths is dominantly laminar, the permeability depends on the range of the grain sizes and on the density of the material which governs the size of the voids. In order to establish reliable values of the in situ permeability, the exploration holes drilling must be done very carefully so that the porosity of the material surrounding the hole is disturbed as little as possible. For permeability measurements the Lefranc method is convenient, which will be described later on. The result is expressed as the permeability coefficient k in cm/s.

2.31 . Leugeon's method for permeability measurement

Lugeon (1933) was the first one to introduce the measurement of rock permeability for the geological study of dam foundations. The unit for permeability introduced by him, described previously, has been in general use.

Permeability is measured in the exploration drill holes at intervals of specified length. When the bottom of the hole reaches the specified depth, drilling is stopped and the string of rods is removed from the hole. Another double string of pipes is introduced into the hole at the bottom of which a plugging device called a „packer„ is fixed. The packer is set in the hole at distance L from the bottom as shown in Fig. 2.10. The length L of the tested section is varied according to the degree of fissuring of the beds between 5 m in less fractured and 1 m in highly fractured rock. The inner pipe is connected to a water pressure pump, with a flow meter and a manometer at its head.

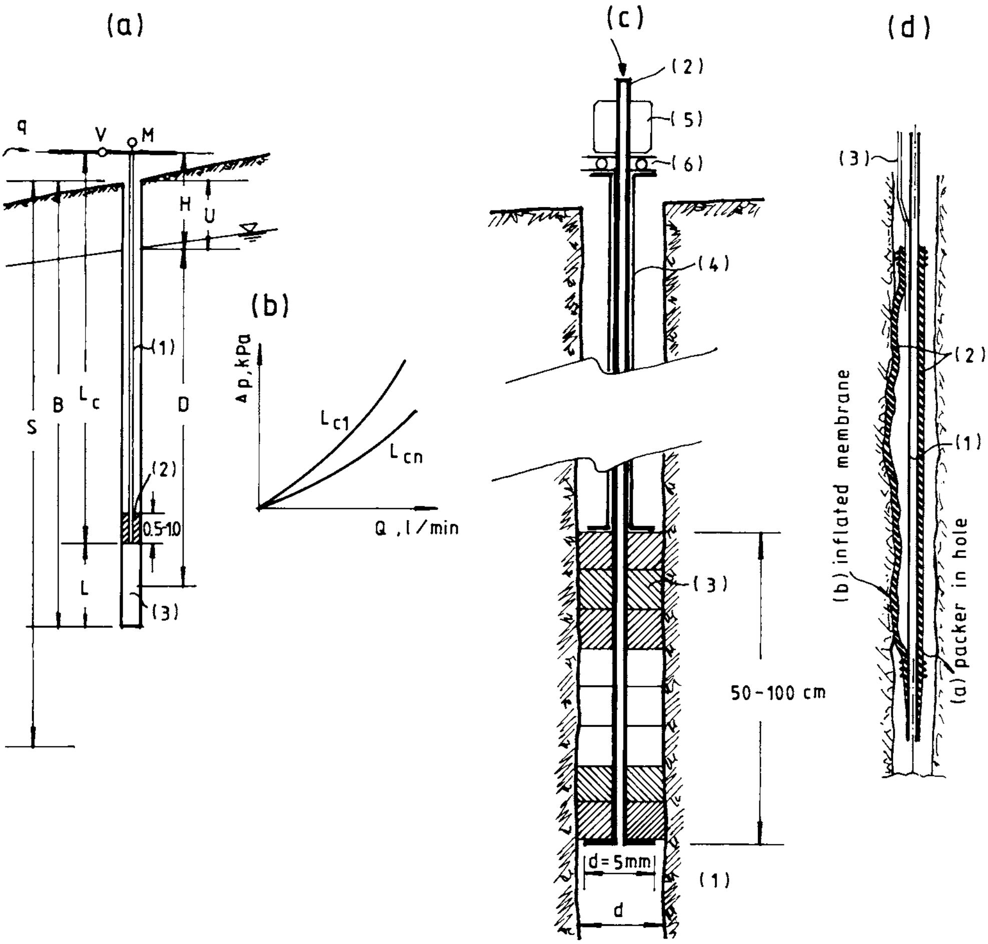


Fig. 2.10 The setup for Lugeon's permeability test, (a) sketch of installation, (q) flow from pump, (V) water meter, (M) manometer, 1 packer pipe, 2 packer, 3 measuring section, (b) diagram of pressure losses in standpipe assembly and packer pipe; (c) sketch of mechanical packer, 1 borehole walls, 2 packer pipe, 3 package of soft rubber rings, 4 external pipe, 5 nut for compressing pipe and rubber rings; (d) inflatable packer, 1 packer pipe, 2 soft rubber tube with ends fixed to packer pipe, 3 compressed air line

The permeability test is carried out in successive stages. First the section is saturated with constant low pressure until the seepage is stabilized at a constant value. Then the flow is measured during a few consecutive flow intervals of 5 min until the difference between two measurements is less than 10% of the measured flow. Then the procedure is repeated with increasing pressure up to p=10 bars and with the same pressure stages in descending order. Houlsby suggested a sequence of pressure stages of p=4,7,10,7,4 bars measured with the manometers at the top of the pipe.

For the evaluation of the test results the actual pressure in the middle of the test section must be computed as shown in Fig. 10(a) from the expression

$$p = (p_M + \Delta H \cdot \gamma_w/10) - \Delta p(q) \quad (2.6)$$

in which:

p = pressure in the middle of test section, bars

p_M = pressure read on manometer, bars

ΔH = difference between the elevation of manometer and the ground water table (or the middle of test section if no GWT exists)

$\Delta p(q)$ = hydraulic pressure losses along the length of pipes and fittings from the manometer to the packers (which is a function of flow q)

Pressure loss Δp (q) is best established for different flows along the length of pipes by direct measurements with the equipment used. It is then shown in charts, Fig. 10(b).

The number *N* of Lugeon units (LU) is established using equation

$$N = 10\, Q / p\, l\, t \quad (2.7)$$

in which:

Q - flow of water through a section of test hole, lit/min

l - length of test section, m

t - time during which Q is measured, min

p - testing pressure, bar.

A centrifugal pump of 200 l/min capacity at 10 bars pressure should be used for pressure testing. In very pervious strata even a larger capacity pump may be needed. The manometers and flow meters used should be regularly checked for accuracy. The elevation of the ground water table should be reliably established by direct measurements before inserting the packer into the hole.

A great inaccuracy of test results may be caused by leakage of water around the packer either by a faulty seal or by water passing from the measuring section into the hole above it through interconnected fissures. Such inaccuracies can best be avoided by carefully setting the packer and by using the appropriate type of packer.

In sound rock with regular and smooth bore hole walls a mechanical packer, shown in Fig. 2.10(c), will be adequate. The set of rings of soft rubber is pressed by means of the nut (5) and the outer pipe (4) and squeezed laterally to produce a good seal with the walls of the hole.

In weak rock the walls of the hole may be irregular and caved in at certain locations. Under such conditions a longer inflatable packer is more reliable. It consists, as shown in Fig. 2.10(d), of an expandable tubular membrane (2) of soft rubber about 8 mm thick, fixed on the pipe. The diameter of the 1 m long membrane should be about 2 mm less than the nominal diameter of the hole.

The space between the membrane and the pipe is connected to the surface by means of a thin tube (3) through which the membrane can be inflated when the packer is in the right place. Since the length of inflatable packers is greater than that of the mechanical type it can be used conveniently even in hard highly fractured rock.

A more complex interpretation of the Lugeon test results based on the hydrodynamic potential field around the test section of the bore hole.It was published in the Earth Manual of the US Bureau of Reclamation. The position of the measuring section of the bore hole relative to the ground water table is considered. As shown in Fig. 2.11(a) the measuring section can be located in zones I or II above the ground water table in unsaturated rock, or in the saturated zone III below it. The location of the measuring section in zones I and II resp. is established from the diagram (a) with the parameters:

$$\chi = 100\ P/U_r \quad \text{and} \quad \Delta = U_v/L \tag{2.8}$$

which define the position of the point (χ , Δ) in the field divided by the line B in zones I and II. The number N of Lugeon units, and the permeability coefficient k are computed for the three zones from the equations:

zone I

$$N = 120\ Q/c_u\ r\ P \qquad k=16.8\ 10^{-4}\ Q/c_u rP \tag{2.9}$$

zone II

$$N=240\ Q/(c_s+4)\ r\ (U_u+P\text{-}L) \qquad k=33.6\ 10^{-4} Q/(c_s+4)\ r\ (U_u+P\text{-}L)$$

zone III

$$N=120\ Q/(c_s+4)\ r\ P \qquad k=16.8\ 10^{-4}\ Q/(c_s+4)\ r\ P$$

The units in these equations are:

Q (l/min) ; L (m) ; P (m) ; U_u (m).

Coefficients c_u and c_s are obtained from the curves in Fig. 2.11(c) and (d) respectively with parameters P/r and L/P for c_u and L/r for c_s.

This procedure does not yield significantly different values of N than equation 2.7 in zone III when length L $\geq$ 3 m. In zones I and II, however, the value of N is overestimated by more than 60% if L = 1 m, and underestimated by more than 50% if the section is above the ground water level. Such deviations from the correct value of N might be important when deciding about the need for, and the volume of grouting.

For the evaluation of permeability for seepage below a dam foundation the hydrodynamic potential theory is generally applied considering all relevant results of the geotechnical exploration in which the permeability coefficient k figures. Its values can be determined from equations (2.9).

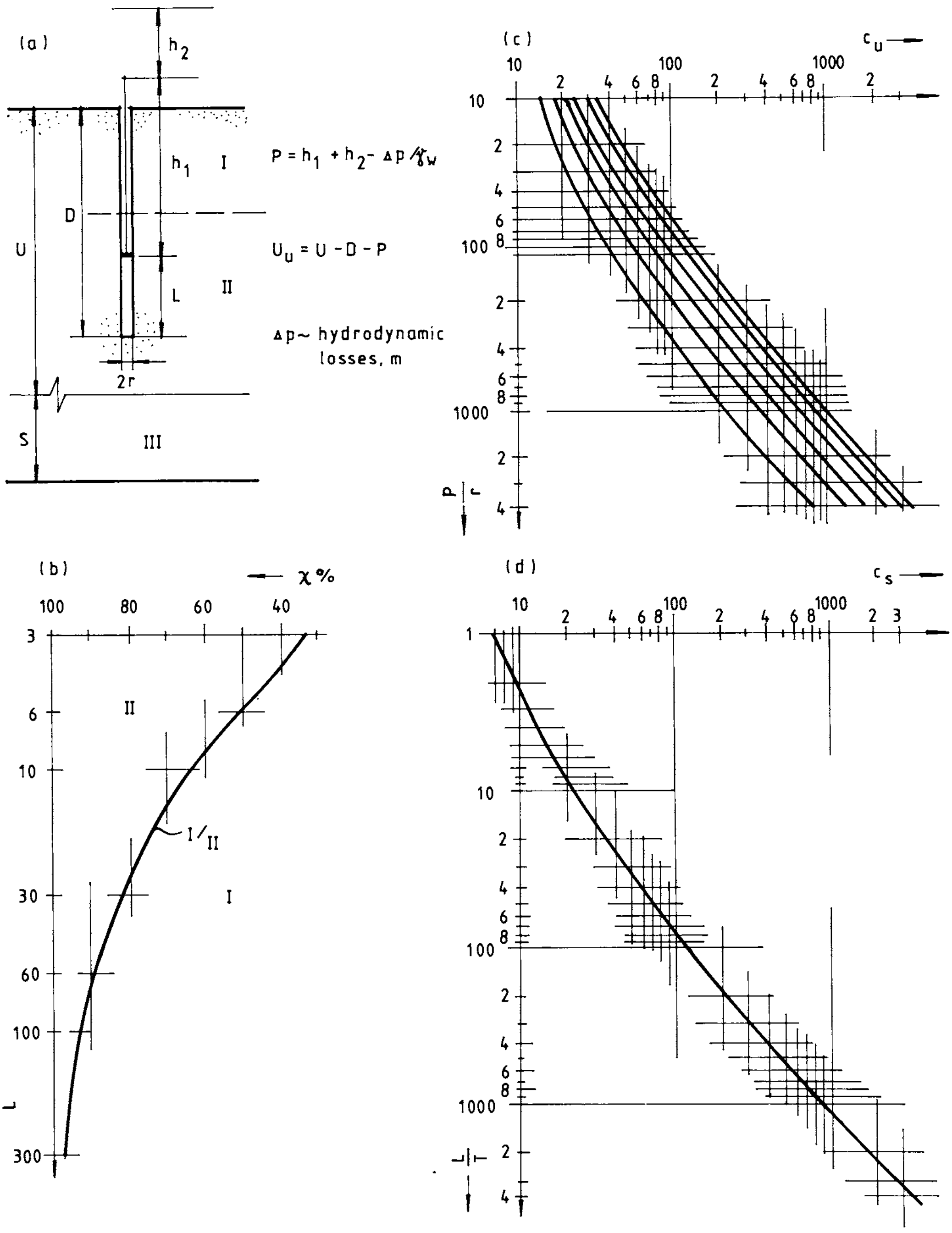


Fig.2.11 Diagrams for permeability evaluation from Lugeon tests, (a) sketch of installation, (b) graph for computation of parameter χ (c) diagram for computation of coefficient c_u, (d) coefficient c_s

A simple relationship holds between N and k when the length of the measuring section is L = 5 m:

$$k = 1.5\ 10^{-5}\ N,\ \text{cm/s},\ \text{when}\ r = 4.6\ \text{cm}, \tag{2.10}$$

$$k = 1.3\ 10^{-5}\ N,\ \text{cm/s},\ \text{when}\ r = 7.6\ \text{cm}.$$

Testing pressure p applied in water pressure tests induces stress changes around the tested section which influence the measured permeability. These changes should be properly accounted for in order to achieve realistic results.

The results of water pressure tests are presented in diagrams with pressure p and flow q as coordinates. Some characteristic examples are discussed based on Fig. 2.12.

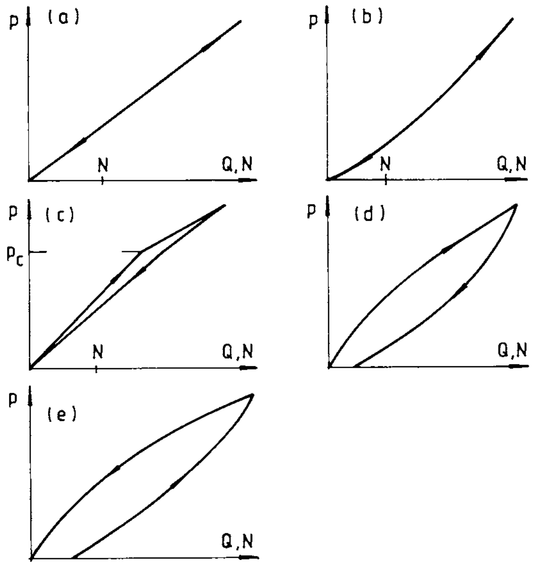


Fig. 2.12 Different types of p/q diagrams from water pressure tests, (a) at laminar flow, (b) at turbulent flow, (c) at hydraulic fracturing, (d) at erosion of fracture fill, (e) at plugging of fissures

Fig. 2.12(a) presents the case in which the flow in the fissures around the section is laminar, the material in them is not eroded, and the relationship between p and q is linear. The value of N in this case is constant for all pressures p applied during the test. In the case of Fig. 2.12(b) where the flow in the fissures is probably turbulent, the flow q decreases gradually with the applied pressure. In the case shown in Fig. 2.12(c) the formation was hydraulically jacked (claquage) at the pressure p_c - existing fissures are widened and new ones opened where the minor principal stress becomes negative, inducing a sudden increase in permeability . In most hydrotechnical structures the impounded water can not produce hydraulic jacking in the foundation, thus lower permeability corresponding to the testing pressure $p<p_c$ should be considered in the computation of percolation.

In Fig. 2.12(d) a case is shown where the material filling the fissures is being eroded during the test, the permeability increases exponentially with increasing pressure, but it remains high when the pressure is reduced.

The case where the material filling the fissures is being displaced during the pressure test, but deposited at some distance from the test section, is shown in Fig. 2.12(e). In such a case the permeability of the formation may be reduced after the test.

The permeability measured in the water pressure test and expressed in LU is not constant. It depends, among other factors, on the applied pressure, on the strength and on elastic properties of the rock. Its value must be analyzed and selected differently in two groups of problems:

- the analysis of water percolation through the foundation of dams;
- the analysis of grouting,

as will be explained later on.

2.32 The Lefranc test

This test is devised for measuring the permeability of granular media in sections of the exploration hole. It is carried out in two ways:

- through the bottom section of a cased hole;
- through the length L of an uncased section of the hole.

In materials of higher permeability, constant pressure head h_t is maintained and flow rate q (l/s, cm^3/s) is measured. The permeability coefficient k is computed from the applied pressure head h, the shape and size of the tested section and from the flow q. It is assumed that the material around the test section is homogeneous and that the percolation is laminar. The pressure gradient is evaluated from the hydrodynamic field around the hole and the corresponding boundary conditions. If the material is less pervious a falling or rising pressure head test can be carried out.

The hole should be prepared for the test with great care. Drilling operations should be carried out so that minimum disturbance occurs in the surrounding material. Drilling manoeuvres should be slow, especially pulling out the drilling equipment (rods and sampler), in order to avoid any hydraulic failure at the bottom of the hole which would loosen the surrounding material and increase its permeability. Only slowly circulating clean water should be used for drilling, which prevents the loosening of the material on the walls and the bottom of the hole.

The test results depend to a high degree on the satisfactory sealing of the casing on the top of the tested section. Circulation of water from this section to the surface along the casing should be prevented. Various shapes of test sections are shown in Fig. 2.13. In practice, tests are carried out through the bottom of the hole, as in sketch (a) or through a cylindrical section as in sketch (d). In the first case, percolation is dominantly vertical, in the second one dominantly horizontal, therefore the resulting k value corresponds to the vertical permeability in the first, and the horizontal permeability in the second case. Thus the presence of permeability anisotropy can be evaluated. A few details of the operation should be mentioned.

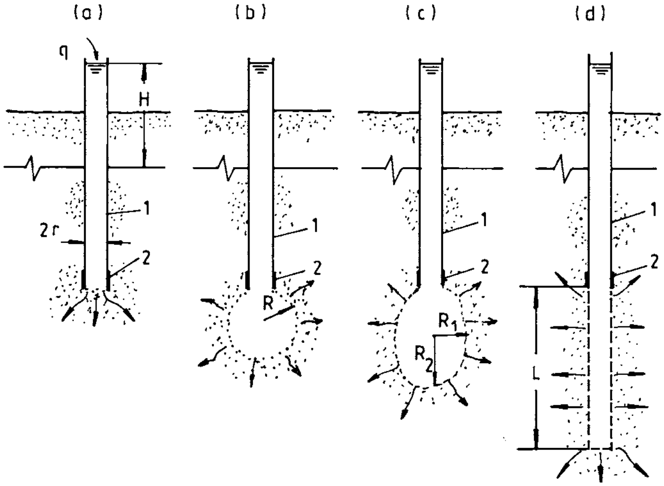


Fig. 2.13 Different shapes of test sections in Lefranc test, (a) test through the bottom of the hole,(b) spherical section, (c) ellipsoidal section, (d) cylindrical section, 1 casing, 2 clay plug for sealing the end of casing

Drilling and preparation of test section

Careful drilling should be carried out with equipment which causes the least disturbance. During the drilling the casing should continuously follow the drilling bit, and the level of flushing water should always be more than 1-2 m above the ground water table. When the bottom of the hole reaches the desired depth, a plug must be achieved which will prevent the percolation of testing water to the surface along the casing, as shown in Fig. 2.14. The casing is carefully raised by about 50 cm. Then small balls of plastic clay (preferably bentonite) are filled into the hole in 2-3 layers and compacted

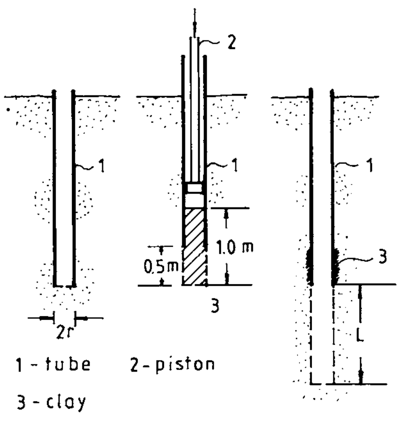


Fig. 2.14 Three phases of placing clay plug at the end of casing

with a rod until a plug 1.0 m high is achieved. Then the casing is pushed down and the clay is drilled through, the bottom is carefully cleaned and the hole is ready for testing through the bottom. Before testing the stabilized ground water level in the hole is measured.

After the water pressure test is completed, the hole is carefully deepened by about 1.0 m, and the test is carried out again through the cylindrical section. If the walls of the uncased material collapse, drilling may be carried out through a perforated casing.

Measurement of permeability

The most reliable permeability test is carried out in three steps, through the bottom and through the cylindrical section:

1. the hole is filled with water up to the top and this water level is maintained by the measured constant flow q which is checked a few times until it becomes constant;

2. the flow is stopped and the rate of level drop dh/dt, and the head h above the ground water level are measured at a few time intervals;

3. then the water level in the hole is lowered below the ground water level, the rate of level rise dh/dt is measured as above; the negative head should be within safe limits to prevent hydraulic failure around the test section.

Three values of the permeability coefficient k are thus obtained in both the test through the bottom and the test through the cylindrical section of the hole. The difference between the values of k from the tests through the bottom and through the cylindrical section, reflect the degree of permeability anisotropy of the material. The differences in the values of k in the three steps of the test should generally be minor. Larger differences would indicate either that the seal of the casing bottom is defective or that hydraulic failure has occurred. In the first case k from the first stage would be larger than the one from the second stage and it would decrease with the dropping head in the second and the third stage. In the second case, k from the third stage would be significantly larger than in the second stage. In such instances values of k are not reliable, the closest value would be the one from stage 2 at low head difference h.

Computation of the permeability coefficient

The computation of k is based on the theoretical potential field of around the test section in a homogeneous porous medium and on the Darcy law

$$v = i\,k \tag{2.11}$$

from which the flow

$$q = v\,A \tag{2.12}$$

is obtained, where A is the area of the test section.

The pressure gradient is expressed as

$$i = c/h \tag{2.13}$$

c being a constant of the flow net which depends on the shape of the test section. Then the permeability coefficient from the constant head test can be written as:

$$k = c\, q/h\, A. \qquad (2.14)$$

In the case of the constant head test through the bottom of the hole the solution yields:

$$k = q/0{,}55\, r\, h, \qquad (2.15)$$

from a dropping head test:

$$k = (1{,}31\, r/\Delta t) \log h_o/h_t. \qquad (2.16)$$

The solution for a cylindrical constant head test, section length L, radius r is i by:

$$k = (0{,}37\, q/L\, h) \log L/r, \qquad (2.17)$$

and for a variable head test:

$$k = (2{,}64\, r^2/L\, \Delta t)(\log L/r) \log h_o/h_t. \qquad (2.18)$$

Sketches and dimensions are shown in Fig. 2.15.

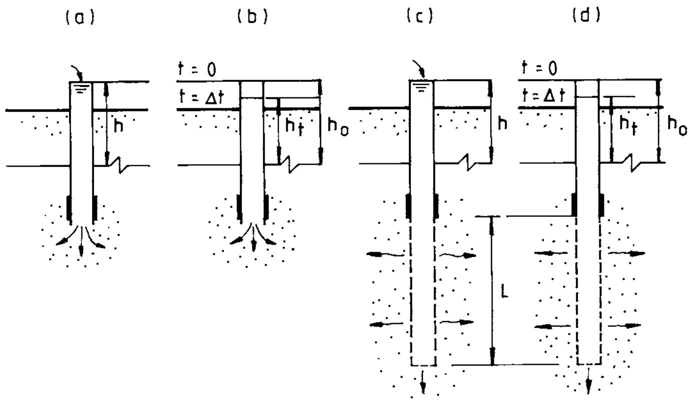


Fig. 2.15 Notations for the Lefranc test equations 2.16- 2.18, (a) measurement through the bottom at constant pressure, (b) same at variable pressure, (c) cylindrical section at constant pressure, (d) same at variable pressure

2.33 Use of the permeability results for percolation and grouting analyses

Lugeon has defined the rock permeability from equation 2.7 as the number of units which is obtained from the measurement at 10 bar pressure. He proposed criteria on which the need for grouting of the foundation of water impounding structures is as follows:

- for dams lower than 30 m, the foundation should be grouted to a depth where N $\geq$ 3 LU, and for higher dams N $\geq$ 1 LU.

In order to make decisions for the need and the extent of grouting under high dams which are built on geologically unfavorable conditions, such a simple rule is not satisfactory. In such cases the consequences of impounding must be evaluated on:

- the uplift on the foundation plane of the structure;
- the impact of the flow of water through the foundation on its stability and the possibility of erosion of filled fissures;
- the total flow of water lost through the foundation rock and its economic importance. The permeability coefficient k is needed for such studies .

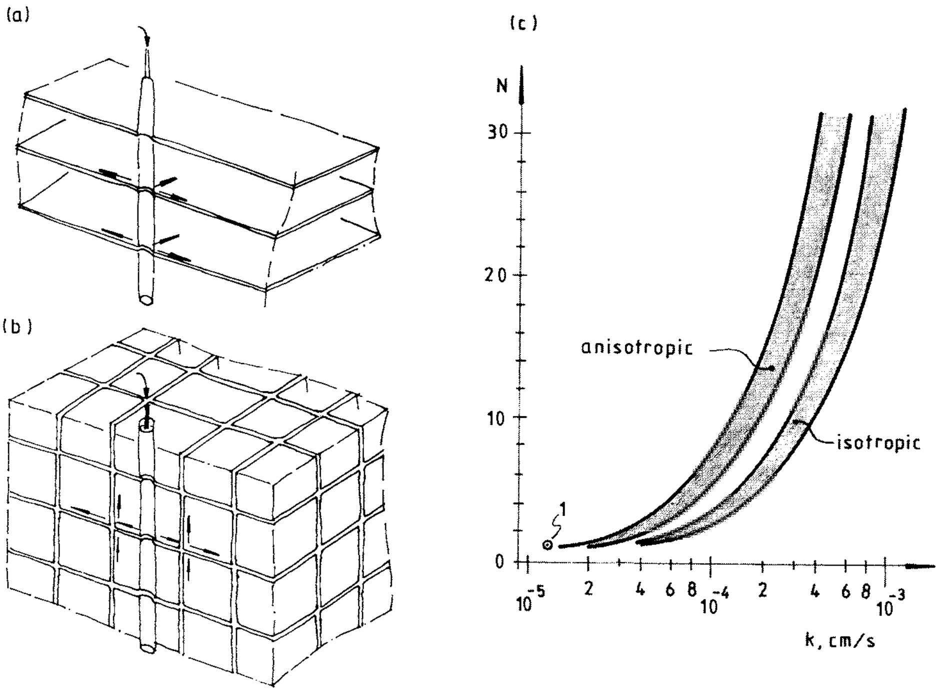


Fig. 2.16 Relationship between Lugeon values and the permeability coefficient k for fissured media, (a) model with extreme permeability anisotropy, (b) model with isotropic permeability, (c) ratio between N and k, 1 from Eq. (2. 10), N Lugeon Units.

Rissler (1977, 1980) has computed k applying the solution of Wittke and Louis (1968), as described briefly in section 2.2, to the test section which is intersected by joints and fissures. The results for a water pressure test in a hole of 7.6 cm diameter are shown in Fig. 2.16 for the measured flow q at a test pressure of 5 bar. Two extreme cases are shown:

(a) one system of fissures perpendicular to the axis of the test section, width δ,

(b) three systems of mutually perpendicular fissures of the same width as in (a).

Case (a) presents an extreme case of anisotropy with k_z, $k_y = 0$, $k_x = k$, case (b) is an isotropic medium, k_z, $k_y = k_z = k$. The relationship $Q : k$ shown by the curves holds for laminar flow through the fissures, the range of the outer boundary of the region is between $10 < R < 100$ m around the

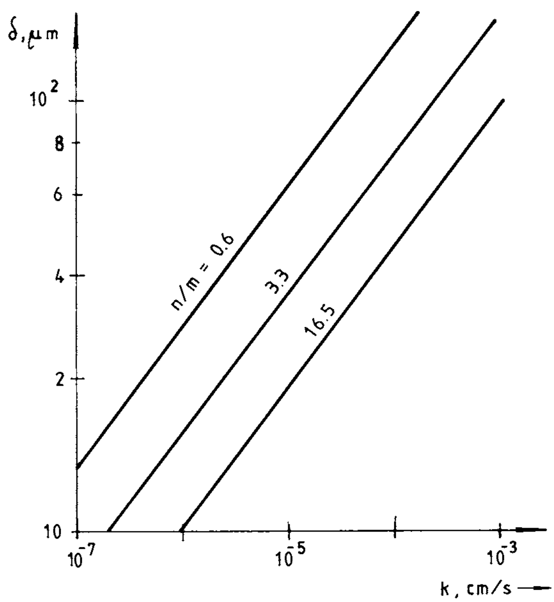


Fig. 2.17 Relationship between the width δ and the number n of fissures per meter, and the permeability coefficient k (Vaughan, 1963)

hole. In the diagram the point which corresponds k from equation 2.10 is shown, which was computed for a homogeneously porous medium. It is seen that the data from equation 2.10 should be doubled to meet the results obtained by the more accurate approach by Rissler. It is interesting to note that the much simpler computation of k assuming a homogeneously porous medium does compare well with the more sophisticated computation for a fissured medium, and that there is a good correlation between the values of N and k.

By equating the flow q through a porous medium with the one through a fissured medium with one system of equidistant fissures at a spacing of 1/n, Vaughan (1963) has computed the width of the fissures which would give the same k as:

$$\delta = (2k/n\,\gamma_w)^{1/3} \tag{2.19}$$

Fig. 2.17 shows the relationship between k, n and δ.

Water percolating through rock causes changes of the stress state/ which are not the same around the test section of a bore hole, as in the foundation rock below a dam, as shown in Fig. 2.18. In the

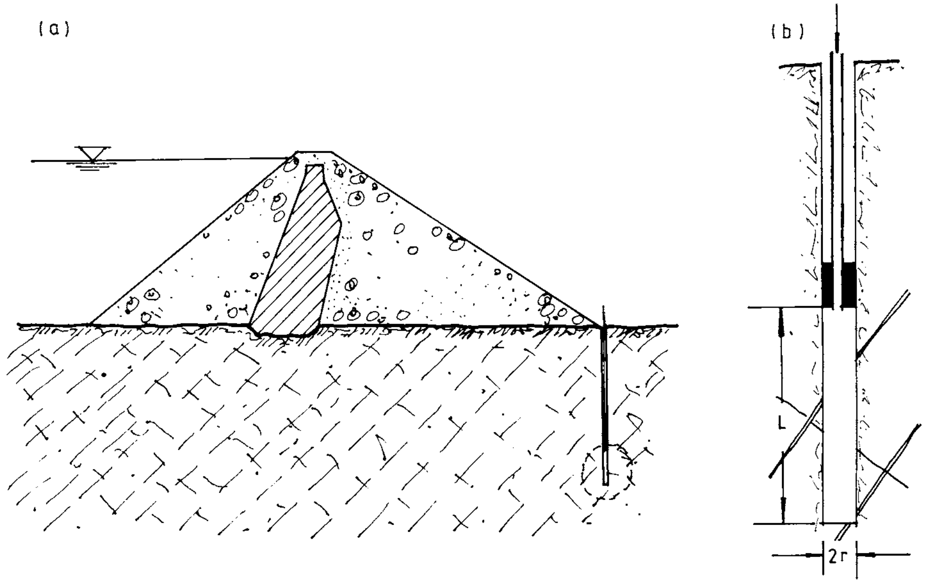


Fig. 2.18 Permeability of fissured rock, (a) in the domain of dam foundation, (b) local conditions around the permeability test section

test section, water pressure acting on the perimeter causes tensile stresses which decrease the ambient stresses, dilate the rock, and increase its permeability.

The opposite is the case in the foundation of a dam, where the weight of the structure increases the stresses and closes the fissures, thus reducing to some degree the permeability. It can safely be assumed that the actual permeability of rock in the domain below the structure is lower than the one corresponding to the tangent at the origin of the (p/q) line of any single measurement, as shown in Fig. 2.12.

The results of water pressure tests described provide the basis for completing a geotechnical model for the computation of the hydrodynamic field in the dam foundation. The result of such computation is only an approximation of the real conditions, the more so the less accurate the input data on geology, stratification and permeability measurements are.

A misinterpretation of permeability tests can lead to an erroneous conclusion as in the example of the investigation for the foundation of the Derbendi Khan Dam (Iraq) in 1959. The dam foundation consists of schisty and marly rock, and the exploration results have shown that the rock permeability was less than 10 LU in only 10% of the tested drill hole sections, while in 50% of them it was more then 55 LU (Fig. 2.19). It was assumed that the rock is very permeable. A detailed scrutiny of the permeability test records has shown that the formation was being hydraulically fractured at test pressures well below p = 10 bar, on which the evaluation of k was based. The critical pressure at which the foundation was fractured is shown in Fig. 2.20. When permeability coefficient k was rechecked at test pressures lower than p_c, it was found that the actual in situ rock permeability was significantly lower. The ground curtain was satisfactorily grouted and no problems arose with seepage after impounding the reservoir.

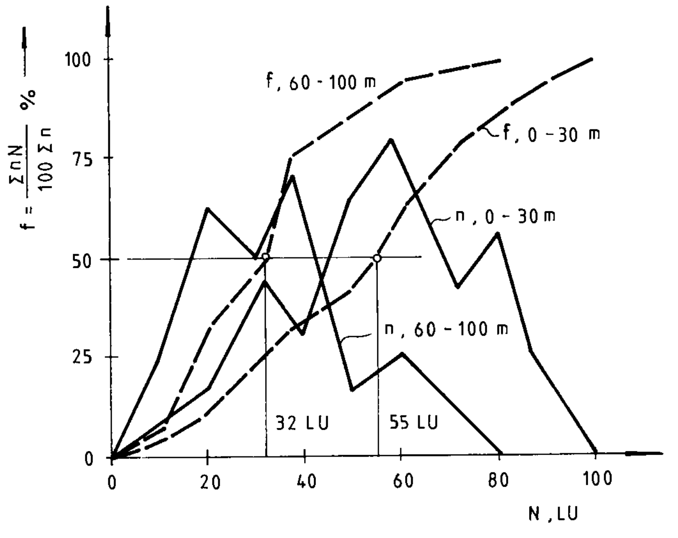


Fig 2. 19 Distribution of measured permeability in sections of exploration holes at Derbendi Khan Dam at depths 0-30 m and 60-100 m

Different criteria are needed for the evaluation of rock properties related to grouting which is aimed at plugging as many fissures as possible in a desired radius around the grouted section. At higher pressures grout penetrates the fissures farther, and the penetration is also improved by hydraulic fracturing when new fissures are opened due to exceeding the critical pressure.

Thus, for the evaluation of rock grouting characteristics the permeability N (or k) should be computed for the maximum pressure achieved in the water pressure test. In the example shown in Fig. 2.12(c) and (d) permeability ought to be computed for the curtain design and seepage analysis from the tangent at the origin of (p,q) line, but for the evaluation of grouting conditions and grout consumption it should be computed the from flow q at the highest pressure achieved in the test. In grouting practice such details are often overlooked.

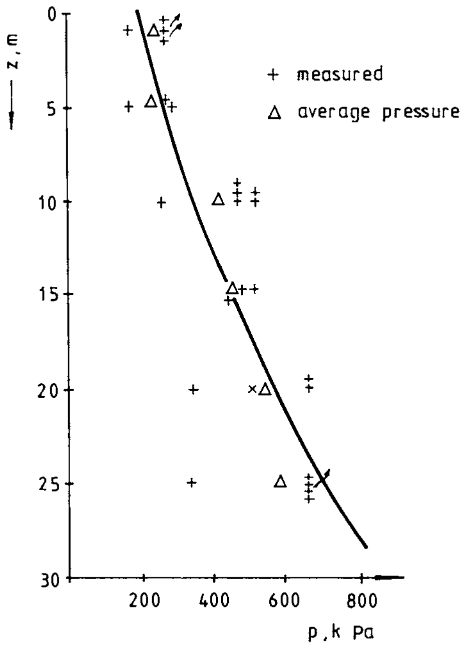


Fig. 2. 20 Critical pressure p_c at which hydraulic jacking occurred in exploration holes at Derbendi Khan Dam foundation

It can be concluded that:

- rock permeability as obtained from Lugeon's test at a pressure of 10 bar (equation 2.7) is not to be used for the analysis of seepage through the foundations of dams; the value obtained from the slope at the origin of the (p,q) line must be used instead
- for the estimate of grouting conditions and grout consumption, the permeability should be used from the (p,q) line at the envisaged grouting pressure.

In cases when the capacity of the pump is not sufficient to achieve the specified testing pressure, the test result is often described as "permeability very large", permeability "not defined" or "permeability indefinite". In such cases p and q values at lower pressures may be used to evaluate the value of N. If low pressures can not be achieved even at full capacity of the pump, the test may be carried out so that the packer tube is filled with the available flow q, and the water level drop in the tube is measured. The permeability is then evaluated as described in the Lefranc test.

3. GROUTING SUSPENSIONS, SOLUTIONS AND RESINS

Various materials are used for grouting, depending on the purpose of grouting and the properties of the grouted rock or soil. They may range from plastic mortars, thick or liquid suspensions of cement and other compounds and additives in water, solutions of chemicals, resins, artificial foams, to hot bitumen and bitumen emulsions.

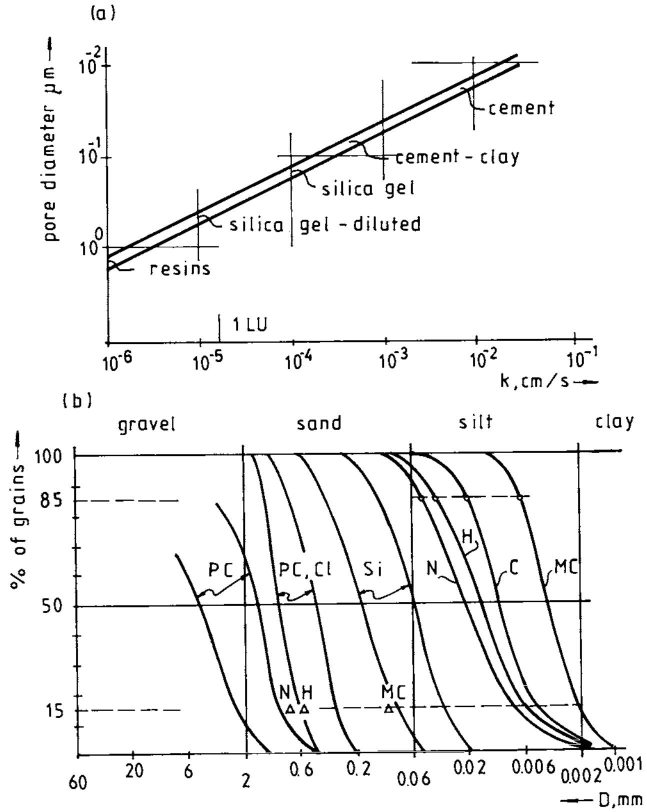


Fig. 3.1 Permeability and grain size distribution of injectable soil, (a) permeability, void size and limits of groutability, (b) grain size distribution and groutability with: PC-Portland cement, PC,Cl-cement/clay, Si silicates; cement grain size curves: N normal, H high early strength, C colloidal, MC ultrafine

Suspensions are injected into fissured rock and into granular media with high porosity and large voids. Mortars which are composed of fine to coarse sand, cement and some plastifiers,are used to plug large fissures and cavities. Solutions are injected into fine grained porous soils into which suspensions can not penetrate, and a gel or stiff material is formed in the voids. A recent advance is in using several kinds of resins the viscosity of which is comparable to viscosity of water, and which are more penetrable than the solutions. The size of fissures and voids and the permeability coefficient of materials which can be grouted within this range of materials is shown in Fig. 3.1(a). In Fig. 3.1(b) the range of the grain size curves which can be grouted with different compounds is shown.

3.1. Suspensions

The ingredients for the preparation of mortars and grouting suspensions are: cement (C), bentonite (B), clay (CL), sand and fillers (S), additives (A) for stability, and water (W).

In practice the description of grouting suspension is not standardized. In some instances the mixing ratio of the ingredients by volume is given, and Anglo-Saxon, metric units and lately also the SI system of units are adopted. In other instances the mixing ratio by weight or by mass units is given. In some cases sacks of cement and liters of water are used, introducing more confusion, because a sack of cement contains about 45 kg (94 lb) in Anglo-Saxon units, whereas it contains 50 kg in Europe and many other countries, and 40 kg in Australia . More confusion is introduced regarding the specific grout consumption which may be given as sacs/ft, kg/m and lit/m. In many papers and publications the real meaning of the units is not even well defined. In order to avoid any ambiguity in this book, mixing ratios by weight and specific grout consumption in kg/m of the dry ingredients will be used. The data presented in other systems of units can be converted by means of diagrams prepared by Houlsby (1977).

The composition of a suspension is defined by the proportion of the dry weight of ingredients in the unit mass, and by the ratio of water added to the dry ingredients. A grouting suspension, for instance consisting of 1 unit of cement, 3.75 units of clay, 0.20 units of bentonite and 0.05 units of additives by weight, and 2 units of water on 1 unit of ingredients by weight can be described as:

$$0.2\ C:0.75\ CL:0.04\ B:0.01\ A:2.0W \tag{3.1}$$

3.11. The penetration of suspensions into fissures and voids

A suspension injected under pressure will circulate in the fissures and voids like a viscous fluid until some of the larger suspended particles are blocked where the fissures or voids get narrower than the size of the grains.

The ability of cement grains to penetrate through the voids of homogeneously porous media can be evaluated from filter tests introduced by Sherard et al. (1984). These tests have shown that grains of diameter 9 $\mathbf{d}_{85}$ in the suspension are safely retained on a filter layer, the grain size of which is $\mathbf{D}_{15}$, and a filter of $\mathbf{D}_{15}=20d_{85}$ will safely pass them through. The permeability coefficient of the filter is in the range of $k=0.35\ D_{15}$ (D_{15} in mm). The range of grain size distribution of several kinds of cement used for grouting are shown in Fig. 3.1(b) (after Shomodi and Ohmori, 1982). From Sherard's relationship it is seen that the normal Portland cement $\boldsymbol{N}$ can be injected into gravelly sand with $D_{15}=0.84$ mm, the high early strength cement $\boldsymbol{H}$ into coarse sand with

D_{15}=0.67mm, the colloidal fine cement ***C*** into medium sand with D_{15}=0.38 mm, and the ultra fine cement ***MC*** into fine sand with D_{15}=0.12 mm. The corresponding permeability coefficient from Sherard's relationship as well as the specific surface of the cements and the respective d_{85} and D_{15} are shown in Table 3.1.

TABLE 3.1

Limits of penetration of cement into granular soils

Cement	Specific surface cm/g	k cm/s	d_{85} mm	D_{15} of soil mm
N	3.170	$2,3.10^{-1}$	0,047	0,87
H	4.320	$1,3.10^{-1}$	0,033	0,67
C	6.270	$3,2.10^{-2}$	0,019	0,38
MC	8.150	$3,5.10^{-3}$	0,006	0,12

A homogeneously porous material can be presented as a cluster of capillaries of diameter ***k*** through which a laminar flow of water is established. From Darcy's law the velocity is v=k i and from Hagen Poiseuille's law it is $v = d^2\, i\, \gamma/32\, \mu$, which, when equated, yields the diameter of the capillaries which would correspond to the permeability:

$$d_k = C \sqrt{k} \quad (3.2)$$

in which ***C*** is a constant.

From this relationship, the diameter of capillaries corresponding to a given permeability ***k*** through which a suspension with maximum grain size d_s can flow is obtained, because $d_k >$ max d_s must hold. The constant ***C*** is given by Kollbrunner (1948) as C_1=0.10, after Cambefort it is in the range $0.06<C_2<0.08$. Taking a cluster of capillaries with diameter d_k as a model of the porous soil, which corresponds to the soil permeability ***k*** from equation 3.2, the maximum diameter of grains d_s can be estimated as shown in Fig. 3.1(a). This is in any case a rough approximation, useful as a general guide. The diagram in Fig. 3.1(a) shows that suspensions of normal cement can be injected into porous soil with $k > 10^{-2}$ cm/s (some authors assume the limit to be $k = 10^{-1}$ cm/s), clayey suspensions can be injected up to the limit of $k = 10^{-4}$ cm/s, and less permeable soil only with solutions or with resins of low viscosity. The lower limit for permeability is in any case $k \geq 10^{-6}$ cm/s, because below this value the velocity of percolation is so small that only a small volume of soil around the grouting stage can be injected within a reasonable period of time.

The statistically relevant permeability of large volumes of fissured rock can be approximated by Darcy's coefficient ***k*** computed from a large number of Lugeon tests as was shown in Chapter 2. In such materials the fissures are much wider than the capillaries corresponding to the same permeability. The width δ of the fissures is 6-30 times larger than the corresponding diameter d_k of the capillaries. Thus a fissured rock of the same permeability k as a homogeneously porous soil can be injected with suspensions of much larger particle size $max\ d_s$, as seen in Fig. 3.1(a). Cement suspensions can be injected into fissured rock of a permeability not less than k $\geq 1.5\ 10^{-5}$ cm/s (N $\approx$ 1LU) while granular media with k $< 10^{-2}$ cm/s can not be injected.

The velocity of flow of the suspension in the fissures or voids of the material under injection pressure varies from point to point. It depends on the pressure gradient, on the water ratio and the viscosity of the suspension, as well as on the dimensions, the roughness and tortuosity of the fissures or void paths.

The flow of the injected suspension stops when the available pressure gradient is not sufficient to overcome the resistance to flow. Then the injected zone becomes saturated.

The grouting suspensions used for grouting fall into three main categories: *unstable, stable* and *thixotropic* suspensions. With *unstable* suspensions the fissures or voids are filled in two ways:

- by settling where the flow velocity becomes low, as shown in Fig. 3.2(a); in the domain (1) the velocity is high and all particles smaller than d_p are carried through, at (2) the velocity is low so that larger grains settle and gradually fill the space, the flow path is narrowed until some larger grains d_s block it;
- by filtering water from the settled particles in the blocked paths as a consequence of the seepage force and the pressure gradient at (1); the subsequent rise of the grout pressure along a blocked fissure (or flow path in a granular material) is sketched in Fig. 3.2(b); the rising pressure gradient compacts the settled particles and consolidates the injected material.

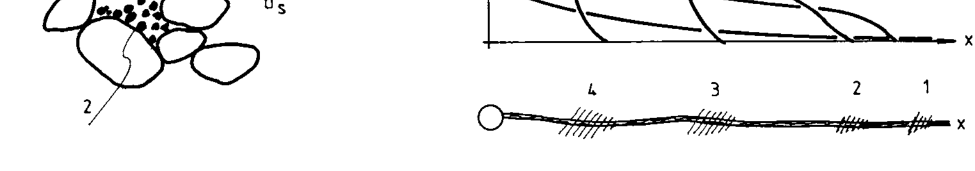


Fig. 3.2 Process of filling voids and fissures, (a) sedimentation of suspended particles, 1 high velocity, particles d_s are carried through voids, 2 block passage d_p, 3 low velocity, particles settle and fill voids, (b) subsequent blocking of the fissure, x path along the fissure, p subsequent injection pressure distribution along the fissure, 1-4 subsequently blocked narrow sections of the fissure and distribution of pressure along the fissure

In both cases the final strength of the material in the grout filling the fissures depends on the grouting pressure, and the pressure gradient at various locations, especially during the saturation phase.

The necessary condition for such a development of the grouting process is a convenient ratio d_p:d_s of the dimensions of the fissures or voids d_p, and the maximum size of suspended grains d_s: if the ratio is high, blocking of the passages will be difficult to achieve, the injected suspension will not be compressed and it will remain soft after setting, the grout will be wasted and the effect negligible. If the ratio is low or less than one, most of the fissures and flow paths will be blocked already at the walls of the injected hole, or not far inside the fissures,and grouting will be inefficient.

With *stable* suspensions, larger grains do not settle during the injecting phase, they travel through the fissures or voids until the flow resistance due to their viscosity overcomes the pressure gradient farther from the grouted hole. Narrow flow paths may also be blocked as described above. With increasing grout pressure, water is filtered out of the suspension in adjacent finer fissures and the injected suspension is consolidated so that it gains initial strength, which does not depend on the setting of the cement.

Thixotropic suspensions develop strength by forming soft gels when at rest. When injected into a formation of high permeability this property enables the blocking of the penetration at a distance of the hole where the viscosity and the developed strength prevent further penetration into the cavities. In this case the final strength of the injected material depends on its composition, and no consolidation by seepage pressure can be expected.

3.2. Materials for suspensions

3.21. Portland cement

The main ingredient for the preparation of grout suspensions is Portland cement, generally available on the market in various types known as Portland, High Furnace or Metallurgical, Pozzolanic, Sulfate Resistant cement, which are also ranged by the strength they achieve and the heat developed during setting. Various Standards exist in different countries which define their properties and characteristics as the chemical composition, the physical properties and the minimum strength.

The chemical composition must comply with a standardized range of contents of SO_3, MgO, $3CaOA_2O_3$, of added inerts, fly ash and pozzolanic material.

The standardized physical cement properties are:

- the fineness defined as content of grains d > 0.09 mm and the specific surface cm^2/g after Blaine;
- the unit mass, g/cm^3;
- the time of the start and the end of setting;
- volume and linear strain after setting;
- the strength after 3, 7 and 28 days.

These properties are established by standardized laboratory tests.

The most interesting property for the selection of cement for grouting is its fineness, which should be as high as possible when granular soil or fissured rock with narrow fissures is to be grouted.

The other properties merely define the cement as a standardized product which as such would be suitable for grouting purposes. The setting time as determined by the Vicat test deserves some comment. In this test the strength of a standard cement paste developed with time is established as the resistance to penetration of standardized small pistons through the sample, and when no marks are left on the sample surface. The start and the end of setting defined in this way are not absolute values, since the hydration process depends on the composition of the sample which is left at rest during the test. Where grouting is concerned the cement suspensions used are not of standard composition and they are constantly in motion until they are injected in the fissures where the cement comes to rest and can start setting and gaining strength.

Some swelling after setting would only be convenient since it would increase the pressure in the injected fissures after the cement has set.

Linear shrinkage, as established from the standardized test, does not apply to injected material since it is continuously in a wet surrounding in the fissures.

Commercially available Portland cement is mostly fine grained (Blaine value about 3,400 cm^2/g) and it can be used for grouting without any restriction. When high turbulence mixers are used for the preparation of suspensions the fineness of the cement is increased, especially if a thick basic mix

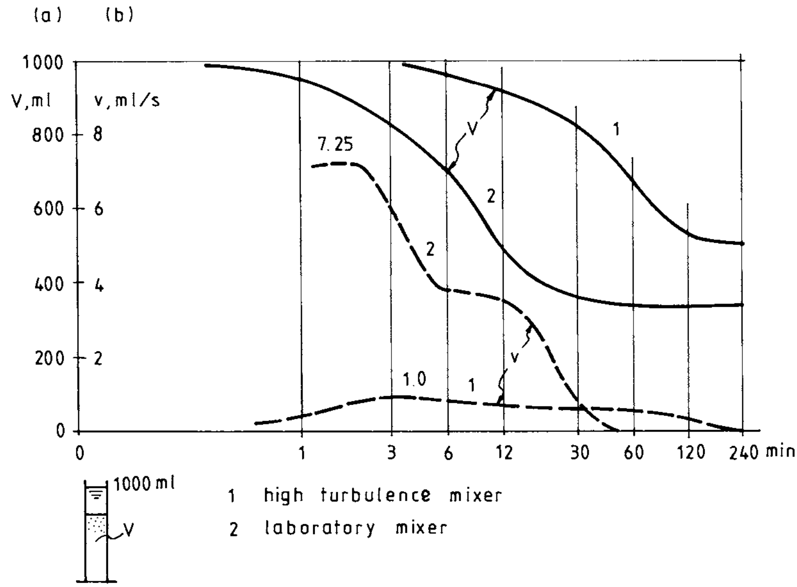


Fig. 3.3 Influence of the mixing method on sedimentation of cement suspensions, 1 high turbulence mixer, 2 laboratory mixer, *V* volume of sediment, *v* sedimentation velocity

is prepared (0.8C:1W) which is then diluted as required. The effect of such a procedure on the sedimentation time and volume, which depends on the grain size of the cement, is shown in Fig. 3.3. The sedimentation of the sample of a suspension of 1C:3W, prepared in a high turbulence mixer, is much lower than the sedimentation of a sample of the same cement prepared only by stirring the suspension in the graduated test jar in the laboratory. As the rate of spherical grains settling in water is proportional to the square of the grains diameter (Stokes law), it can be calculated that the average diameter of cement particles mixed in a high turbulence mixer is $(1:7.25)^{0.5} = 0.37$ of the average diameter of grains in the suspension prepared in the laboratory. Experience has shown that cement containing less than 10% grains larger than 0.09 mm and a specific surface of 2,700 cm^2/g or more, can be used to grout fine fissured rock, the permeability of which is more than 1 LU, provided the suspension is mixed in a high turbulence mixer. The result may be improved when the largest grains are separated from the suspension, leaving the mixer by means of a hydrocyclone.

3.22 Clay

Clay is added as a fine grain filler to reduce the cement consumption, but it also improves the stability and the viscosity of the suspension. Fine fissured rock or granular soil with low content of silt and fine sand should be used. The name clay denotes soil grains smaller than 2 μm, and also a specific group of minerals . These are flaky crystalline particles consisting of one or more groups of alumosilicates with ions of magnesia or iron in place of some or all of aluminum ions in the crystal lattice also containing some alkaline ions. Neither are all particles of clay minerals are not smaller than 2 μm, nor are all soil particles smaller than 2 μm clay particles. For a better understanding of the properties of clay in relation to grout suspensions, it is useful to recapitulate some facts about the structure of clay particles.

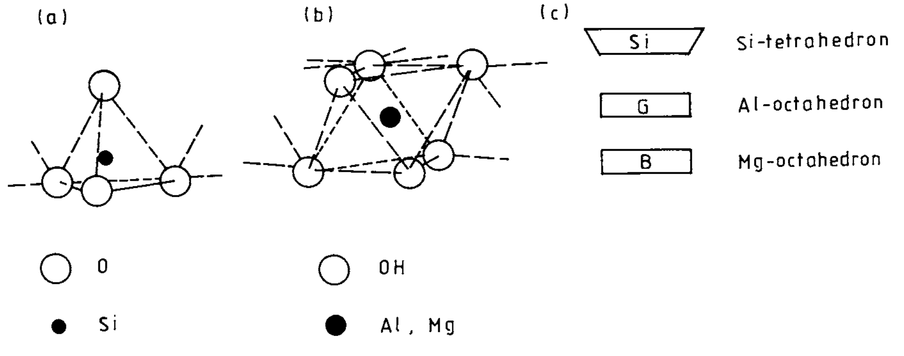


Fig. 3.4 Building blocks of clay minerals, (a) Silica tetrahedron, (b) Aluminum and magnesium octahedrons, (c) schematic presentation of crystals

The building blocks of clay minerals are:

- silica tetrahedrons assembled in sheets on a hexagonal grid in which every three of four oxygen atoms are assembled around a silicon atom (Fig. 3.4(a)).
- aluminum or magnesium octahedrons coordinated in sheets with a common oxygen atom or hydroxile group around the aluminum or magnesium atoms (Fig. 3.4(b)).

In Fig. 3.4(c) these building blocks are shown schematically as units of trapezoidal section for the silica sheets, the narrower side presenting the plane of the top of the tetrahedrons with oxygen atoms, the rectangular ones the octahedral elements of aluminum or magnesium. With such blocks different clay minerals can be presented, as shown in Fig. 3.5.

Kaolinite and montmorillonite clays are mainly used for grouting.

Kaolinite contains two *Si* blocks and one **G** block, as shown in Fig. 3.5.(a), with common oxygen atoms or hydroxile groups in the plane of the tops of the *Si* - tetrahedrons and *Al* -octahedrons.

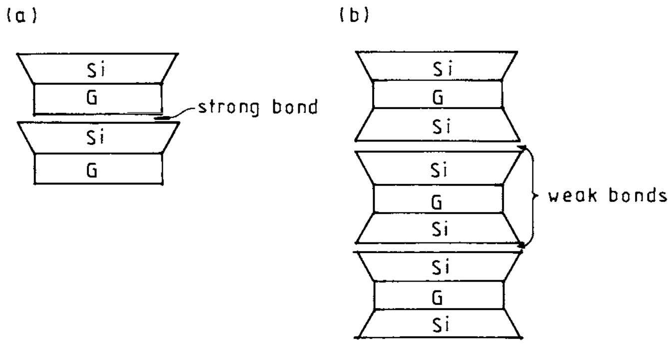


Fig. 3.5 Schematic presentation of clay minerals, (a) kaolinite, (b) montmorillonite

The kaolinite minerals consist of basic sheets bound together by Van der Waals forces and hydrogen bounds which are strong enough to prevent much swelling in the presence of water. This is the most common group of clay minerals, they occur in more or less regular platelets of hexagonal shape with the dimension of 0.1-4 μm in the plane, and their thickness is between 0.05 and 2 μm. The specific surface is in the range of 10-20 m^2/g, and the unit mass around 2.60-2.68 g/cm^3. The ion exchange capacity is 3-15 meq/100 g. It is widely used as a filler in cement/clay suspensions.

Montmorillonite consists of three blocks in the arrangement *Si*:***G***:*Si* with common oxygen atoms and hydroxile groups on both adjacent *Al*-planes and tops of the *Si*-tetrahedrons and build large planes (Fig. 3.5(b)). The bonds on these planes result from Van der Waals forces and cations which may be present to equilibrate missing ions in the structure. These are weak bonds and they are easily broken by splitting or by absorption of water molecules, which produces a high swelling potential on wetting. This property is very important for grouting. The particles of montmorillonite have the dimension of 1-2 μm in the plane, their thickness ranges from 10^{-3} μm to $2 \cdot 10^{-2}$ μm, and the specific surface is 50-120 m^2/g when dry and 700-840 m^2/g after swelling and dispersing in water.The unit mass is 2.35-2.70 g/cm^3, and the ion exchange capacity 80-105 meq/100 g. Montmorillonite is used for the preparation of stable cement suspensions, thixotropic suspensions, for plasticizing thick mortar used for grouting large fissures and cavities, and for the preparation of drilling muds.

Bentonite is a montmorillonite clay containing small quantities of inert mineral grains (quartz, calcite, feldspar, etc.). In natural deposits it is found as calcite bentonite containing calcium ions adsorbed in the crystal lattice. Atterberg's limits of such bentonite are around 30/100%. When the *Ca*-ions are replaced by *Na*-ions the Attergerg's limits increase to about 50/400 %. In Yugoslavia there are vast deposits of bentonite (Petrovo Polje in Crna Gora, Novo Mesto in Slovenia). Activated with sodium it is available on the market pulverized packed in paper sacks of 50 kg. Other clay minerals like illite, biedellite, halloisite etc. are rare and not important in grouting although clays containing such minerals can be used if available at the site.

The mineral composition of clays is determined by means of mineralogic and petrographic analyses, X-ray diffraction and differential thermal analyses. For identification purposes and the evaluation of the geotechnical properties of clays, the grain size analysis and Atterberg's limits are deter-

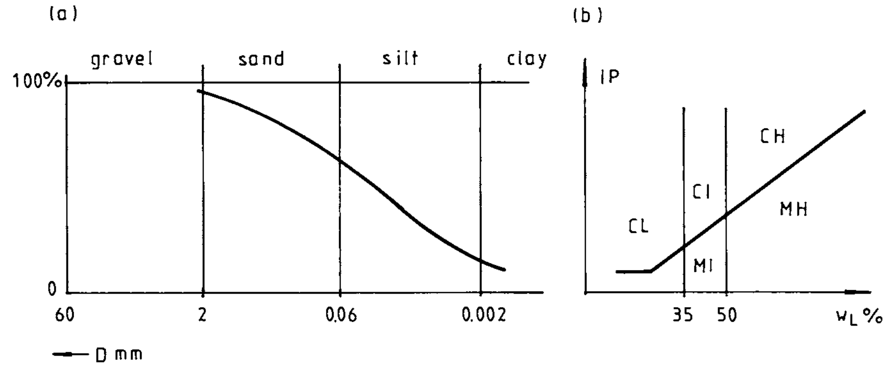


Fig. 3.6 Grain size distribution curve (a), plasticity chart (b)

mined in the laboratory in accordance with the relevant standards. The results are presented as shown in Fig. 3.6. and they are used to establish the classification group of the material. The activity of the material can be established from the grain size curve and the clay mineral content of the material (Skempton, 1953) as:

$$A = IP/C \tag{3.3}$$

in which IP is the plasticity index in % and C = percent of grains smaller than 2 μm. Skempton has determined that the clay mineral activity A is a useful parameter for distinguishing the mineralogical composition of the clay fraction. The activity A of the basic clay minerals is shown in Table 3.2.

TABLE 3.2

Activity of typical clay minerals (after Skempton, 1953)

Clay mineral	Activity A
Kaolinite	0.33
Illite	0.90
Ca montmorillonite	1.50
Na montmorillonite	7.20

These parameters enable to easily identify samples of clay suitable for grouting suspensions and to select samples for more detailed laboratory testing.

Clay can be processed for the preparation of grouting suspensions in its natural moist state or it can be dried and pulverized before used. If a convenient natural supply of suitable clay exists at the site, the first option might prove to be more convenient and rational.

3.23. Sand and fillers

Sand is added to stable grout suspensions when a system of large fissures has to be injected. The grain size distribution and the maximum grain size are chosen to match the size of the fissures and to suit the available grouting pumps, the lines and the fittings. The range of maximum and minimum grain size should strictly meet the specified range.

When larger fissures must be injected, especially if they are percolated by water, several kinds of fillers may be added to stable or even to thixotropic cement suspensions. These may be selected among saw dust, wood shavings, strips of cellophane, polyvinyl or polyester. Chips of artificial sponge added to the grout were successful in plugging wide percolated fissures in the back of some springs of 2 m^3/s capacity (Hlebar et al, 1980).

3.24. Plastifying agents

Although the forces acting on the planes between the sheets of clay minerals are relatively weak, there are unbalanced electric charges on the sections across the sheets, which mutually attract and may bind adjacent single crystals into large agglomerations (flakes). A suspension containing such agglomerations of clay particles has properties which are different from those in a suspension containing only dispersed individual particles, it is less stable and may be more viscous and thus not be suitable for grouting. In order to prevent flocculation in a clay suspension, small quantities of ions called plastifiers are added to neutralize the unbalanced charges on the edges of single crystals, so that they mutually repel. Clay particles then remain individually suspended, and the average size of suspended particles is much smaller than in a flocculated suspension of the same clay.

The most convenient plastifiers are salts of metals which cause the highest increase of its plasticity limit when added to the clay. Such are kations of light metals in the sequence of lithium (***Li***), sodium (***Na***) and potassium (***K***). Since lithium salts are expensive, mainly sodium carbonate (Na_2CO_3) or bicarbonate ($NaHCO_3$) are used, because potassium salts are less active. The optimal proportion of the plastifier to be added depends on the mineralogic composition of the clay particles. It is defined as the ratio of clay by weight in simple laboratory tests which can be easily carried out at the site. The clay is suspended in distilled water in the ratio of 1CL:3W to 1CL:5W several samples are placed in 500 cm^3 graduated laboratory jars and different quantities of the plastifier is added to each jar ranging say from 0.5 % to 5 %. The suspensions are intensively mixed for 10 min and left to rest for 24 hours when the volume of the sediment is measured. The sample which exhibits the largest volume of sediment contains the optimal proportion of plastifier (Fig. 3.7.).

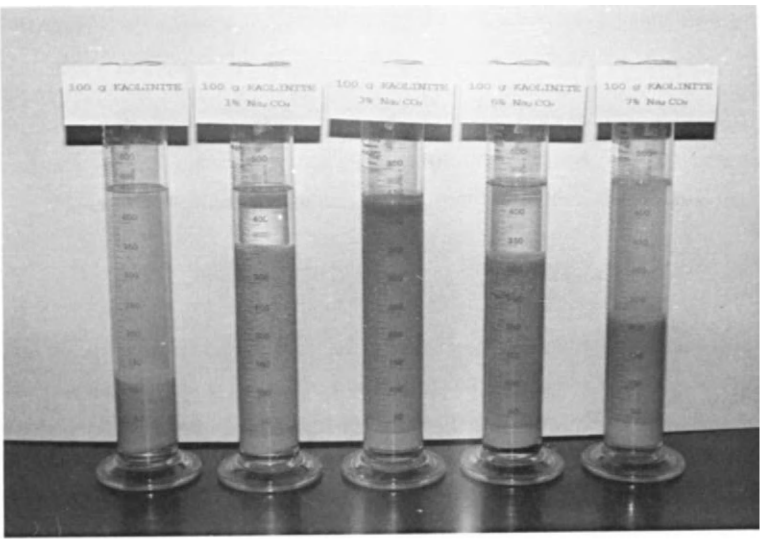


Fig. 3.7 Samples of suspensions with different quantities of sodium carbonate

Several plastifiers for cement suspensions are commercially available. The pumpability of thick cement suspensions and mortars can also be improved by the addition of commercially available air entraining agents. The optimal mixing ratio is also established in the laboratory as described for clay.

When grouting shallow sections of holes, it is sometimes useful to add some accelerators for setting and early strength gain for blocking surface leaks. Besides some commercially available products, potassium chloride ($CaCl_2$) is often used, as well as sodium carbonate. Both reduce the final strength to some extent, more so $CaCl_2$, which also reduces the plasticity of suspensions containing bentonite.

3.3. Properties of grout suspensions

The composition and the properties of grout suspensions should be defined and specified in the design stage of a project to meet the requirements of the job. The properties and the characteristics of the suspensions and of the injected mass are tested in the laboratory. The tests include:

- the components of the suspensions and their mixing ratios;
- the stability of the suspensions;
- the dynamic and the static viscosity;
- thixotropy of selected mixes;
- strength gain in time, strength of the injected grout;
- resistance of injected grout to erosion;
- resistance to chemical corrosion, volume stability of injected grout.

When there is a choice, it is advisable to anticipate the most convenient ingredients from preliminary laboratory tests. Only these ingredients are then tested more comprehensively. The finally selected composition of various suspensions is then a trade off so that the significant properties (such as stability, fluidity, thixotropy etc.) dominate. If there are no special standards for testing suspensions for grouting, those used for testing drilling muds in the petroleum industry may be applied, as well as the standards for soil laboratory testing, where they apply.

3.31. The stability of grout suspensions

Suspended granular particles settle in a fluid at rest under the action of gravity with a velocity which is proportional to the square of the particle diameter D. Coarse particles settle first, followed by finer ones, and the density of the sediment decreases with the size of the particles. Very fine particles are subjected not only to the force of gravity, but also to mutually acting electrochemical forces and to Brownian motion which appears in suspensions of colloidal particles less than 1 μm in diameter, so that the velocity of sedimentation becomes lower than the one corresponding to the Stoke's law.

Carefully composed grout suspensions in which particles do not settle at all within more than 24 hours are called stable suspensions. Some small amount of clear water may appear at the surface, which is not due to particle sedimentation but it is the consequence of syneretic volume shrinkage caused by the action of electrochemical forces and gravity on the whole volume of the suspension.

The stability of grouting suspensions is determined by simple laboratory tests. The specified quantities of ingredients (cement, clay, bentonite, etc.) are mixed and suspended in water (distilled in theoretical studies, from the site for practical studies) to yield suspension samples of desired composition and density. Methods of mixing can also be varied so that the mixing complies with the equipment used at the site. The suspension is then poured into a 1,000 cm^3 laboratory jar and left to rest. At selected intervals of time the volume of clear water segregated on top of the sedimented volume of denser suspension is recorded and the result is presented graphically, as shown in Fig. 3.8.

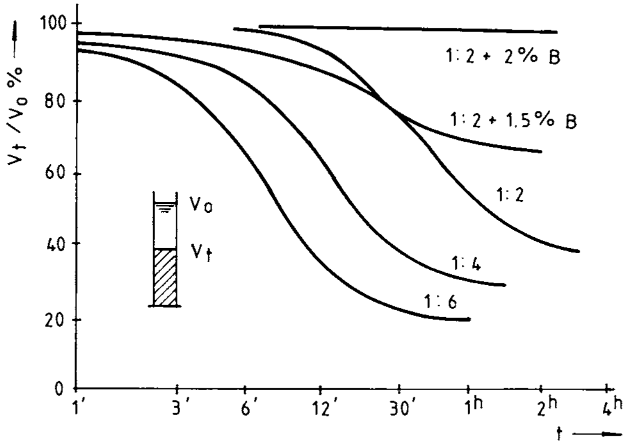


Fig. 3.8 Sedimentation of cement suspensions of different densities, and with bentonite added

In this figure the influence of the initial density of the mix and the influence of added bentonite on the stability of suspensions with the same density is shown.

In Fig. 3.9 the influence of the cement fineness on the sedimentation of its suspension is shown.

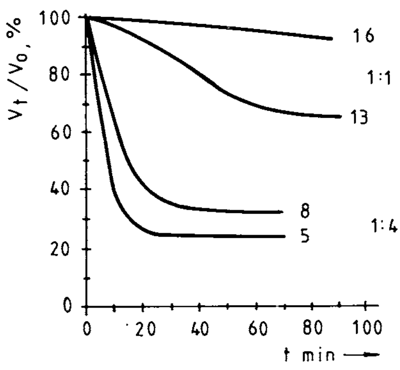


Fig. 3.9 Sedimentation of cement suspensions of different finenesses and water ratios, 5 and 13 fineness 3,200 g/cm^2, 8 and 16 fineness 6,350 g/cm^2 (after Verfel, 1983)

Cement suspensions can be made completely stable by the addition of small proportions of activated bentonite and/or some plastifiers. In Fig. 3.3. the influence of the method of mixing on the stability of a 1C:3W suspension of coarse grained cement from Algiers is shown.

3.32. The dynamic viscosity

Dynamic viscosity is a property of fluids to produce shear resistance when the flow velocity varies in a cross section. For a Newtonian fluid the equation following holds:

$$\tau_{xz} = \eta \, dv/dz \tag{3.4}$$

with denominations shown in Fig. 3.10(a), where η (Ns/cm or mPas) is the viscosity coefficient, and dv/dz is the velocity gradient in the section. Suspensions generally do not behave as Newtonian fluids but as Bingham's liquids for which the equation is given by

$$\tau_{xz} = \tau_o + \eta \, dv/dz \tag{3.5}$$

where τ_o is the threshold resistance. These two relationships are shown in Fig. 3.10(b).

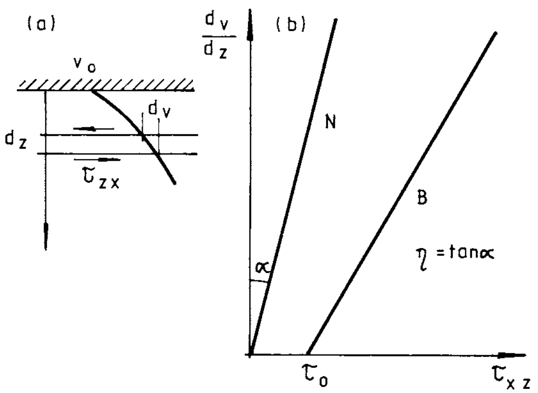


Fig. 3.10 Viscosity of liquids, (a) distribution of velocity in a capillary, (b) ratio between velocity gradient and shear resistance, *N* Newton, ***B*** Bingham liquid

There are several types of rotational viscosimeters, one of which - Stormer's - is shown in Fig. 3.11. The sample of suspension to be tested is poured into a jar, and a metal cylinder open at the lower end is then placed in the jar. The cylinder is rotated by means of an appropriate transmission and the container is loaded with lead shot until the rate of rotation of 600 per min is reached. From this load the viscosity coefficient in mpa s is determined from the calibration curve of the device shown in Fig. 3.11(b).

Simpler methods may be used for pragmatic purposes . The most commonly used is the Marsh cone according to API standard shown in Fig. 3.12. In this case the viscosity is expressed as the time in seconds needed for the outflow of 947 cm^2 (1/4 gallon) of suspension. In Europe the viscosity is expressed as the time in seconds needed for the outflow of 1,000 cm^3 through an orifice of 5 mm in diameter. The viscosity of water at 20 °C is 27 s according to API standard and 29 s according to the European practice. Suspensions with viscosity of up to 60 s are pumpable to a moderate distance from the injection pump. The Marsh viscosity presents a combination of τ_0 and η in equation 3.4(a) of stable suspensions.

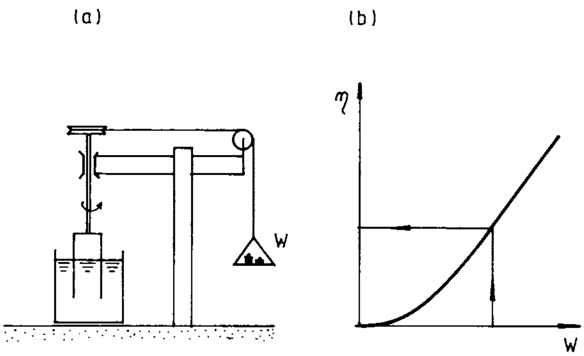


Fig. 3.11 Measurement of suspension viscosity, (a) Stormer's viscosimeter, (b) viscosity diagram

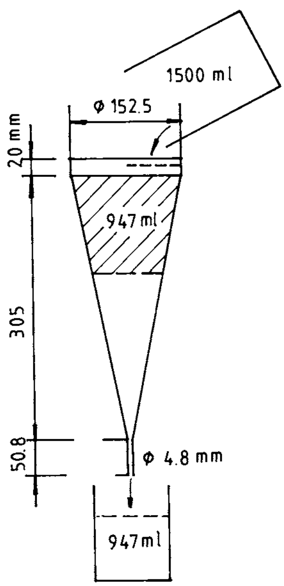


Fig. 3.12 Marsh flow cone after ***API*** for viscosity measurement

In order to simplify the measurement of both viscosity parameters at site laboratories Lombardi (1985), has developed a method to measure the threshold resistance by dipping a thin rough metal plate of 15x15 cm into the suspension. It is then pulled out and left suspended until the drops of the suspension stop falling, and then it is weighed. From the weight difference ΔW between the dry plate and the plate with the adhering suspension the threshold resistance τ_0 is obtained as:

$$\tau_0 = \Delta W/2A \quad (kN/m^2), \tag{3.6}$$

where A is the area of the plate,

Lombardi has found a good correlation between the value of τ_0 measured in this way and the one from Stormer's viscosimeter. With this value of τ_0 the value of η is established from the flow time t measured with the Marsh cone using the diagram in Fig. 3.13.

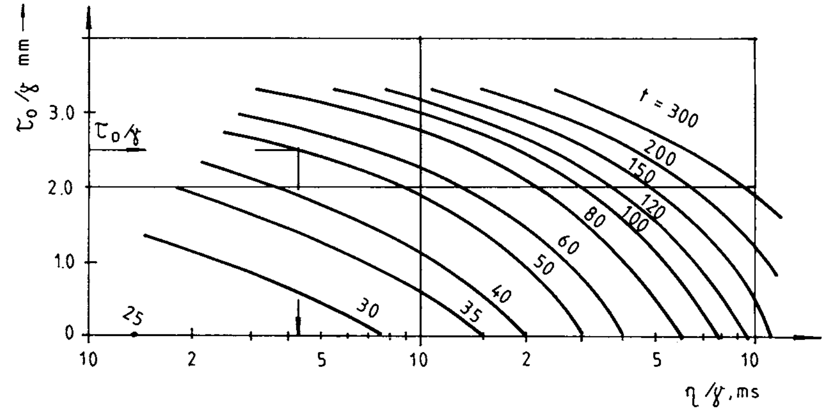


Fig. 3.13 Relationship between viscosity parameters τ_0, η and the Marsh cone flow time t (Lombardi,1985)

3.33. Thixotropy of suspensions

Thixotropy is a property of denser stable bentonite and complex suspensions to form a gel when left at rest. It is due to a small increase of strength caused by electrochemical forces acting among the suspended particles with equal electric charges. The state of equilibrium thus reached provides some rigidity and strength to the assemblage of suspended particles. The process is reversible, i.e. if the suspension is stirred the bonds among the particles are broken up, the suspension becomes liquid, and the gel is formed again when it is left to rest. This process is schematically shown in Fig. 3.14. The threshold resistance τ_0 of the Bingham fluid increases along the line (a)-(b) when the flow is laminar and from (b) to (c) when it is turbulent. When the suspension is at rest for some time, τ_0 increases to τ_t, which is reduced to the starting value τ_0 when stirred at (d), and it becomes liquid again.

The thixotropic strength can be measured with the Stormer viscosimeter. First the line (abc) is obtained with increased rotational speed. After the suspension has been at rest, the line (dec) is obtained (Fig. 3.14). The thixotropic strength τ_t increases with time, and the test should be repeated after successively longer times of rest in order to establish the final value of τ_t. For practical purposes the writer has devised a simpler test using a specially designed hydrometer graduated to measure mass densities on grouting sites of 1-10 g/cm^3 as shown in Fig. 3.15(a). When it floats in a suspension of greater viscosity, in addition to uplift U acting on the submerged area A, the viscous

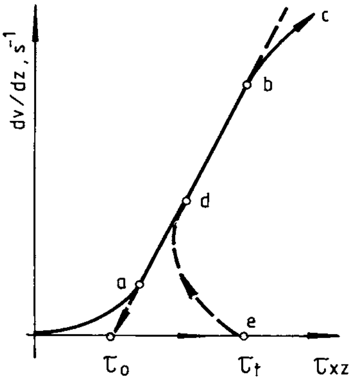


Fig. 3.14 Rheologic properties of thixotropic suspensions

resistance force $\tau = \tau_0\, A$ will act so that it will come to rest at a depth marked on the graduation with a density ra which is larger than the real density ρ known from the composition of the suspension. From the known dimensions (area A, volume V at graduation ρ_a) the threshold resistance of the suspension is computed, with denominations in Fig. 3.15(b) as:

$$\tau_0 = V\, g\, (\rho_a - \rho)\, /A = 0.5\, \alpha\, r\, g\, (\rho_a - \rho) \tag{3.7}$$

where α is a characteristic of the hydrometer shown in Fig. 3.15(c). The hydrometer should be very slowly lowered into the suspension so that the velocity gradient remains low at its surface .When

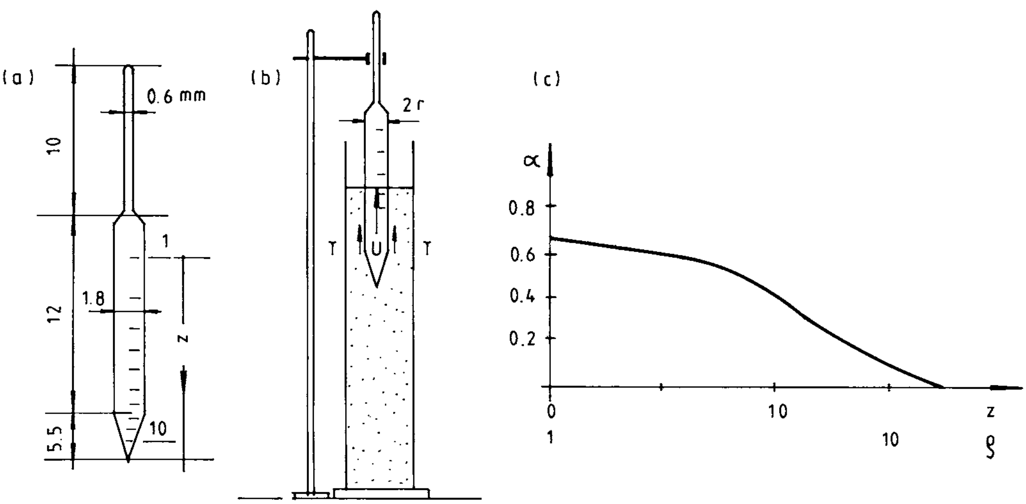


Fig. 3.15. Thixotropy measurement by means of hydrometer, (a) dimensions of hydrometer, (b) equilibrium forces on hydrometer, (c) characteristic α of hydrometer

the test is repeated on the same sample at subsequently longer time intervals, the density corresponding to and the thixotropic strength increases to ρ_t the value of the thixotropic strength τ_t can be established and evaluated by equation (3.7). The ratio between the densities ρ_a and ρ is called the density ratio of the viscosity:

$$R = \rho_a/\rho. \tag{3.8}$$

Experience has shown that suspensions with values of R > 2 exhibit thixotropic properties.

This simple areometer test is a valuable tool for measuring the viscosity and the thixotropy of the suspension and for checking their quality on grouting projects.

3.4. Physical properties of injected grouts

3.41. The strength gain of injected grout

In some projects the strength gain of the injected grout in time is specified as measured in the Vicat test carried out on dense samples of the suspension. Such stipulation can generally not be recommended for two reasons. The first reason is that Vicat's test is only a standard to define some characteristics of cement as carried out on samples of standardized composition. The second one is that water is filtered out from the injected grout that sets in the fissures from a consistency which is much denser than the one of the original suspension. This provides an initial cohesive strength without any strength increase due to the hydrochemical reaction of the cement called setting.

For the purpose of selecting the best composition of grout suspensions when several options exist

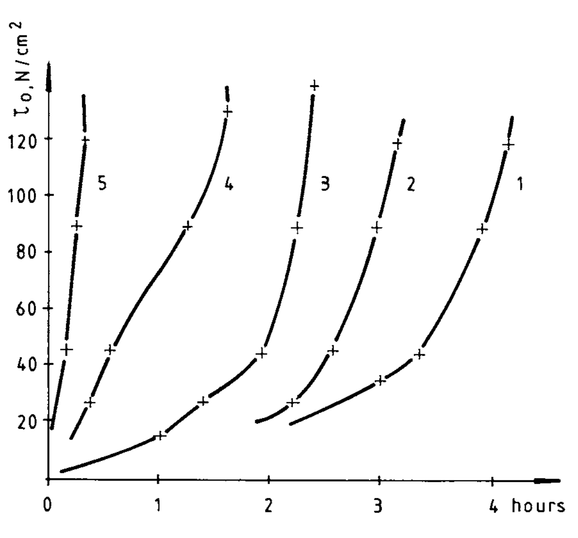


Fig. 3.16 Strength increase of suspensions, 1. 1 C 1 W, 2. 2 C:1 W, 3. 1 C:0.015 B:0.05 $CaCl_2$:1 W, 4. 1 C:0.015 B:0.015 $NaCO_3$:1 W, 5. 1 C:0.03 B:0.02 Na_2CO_3:1 W

for a project, the strength gain of grouts of various compositions can be studied on dense samples prepared with a ratio of 1 W: 0,8 mass can measured by means of the areometer described previously and using a laboratory vane when the strength is higher than its range. In Fig. 3.16 the results of such tests, which were carried out at the site of Derbendi Khan Dam are presented. It can be shown that the final high strength of the grout injected in the fissures is not indispensable for the durability and the safety of a grout curtain. The shear strength τ_f needed to resist the force of the hydraulic head γ_w h_p acting on one side upstream on a δ wide plane fissure, filled with injected grout to length L, as shown in Fig. 3.17 is given by:

$\tau_f / F_s = 0.5\ hp\ \gamma_w\ \delta / L$

and in a tubular fissure of diameter δ is given by: (3.9)

$\tau_f / F_s = 0.25\ h_p\ \gamma_w\ \delta / L$

where Fs is the desired safety factor. The thickness L of the grout curtains is generally such that the ratio $h_p/L<10$. If high safety factor $F_s=10$ is chosen, the needed shear strength of the injected

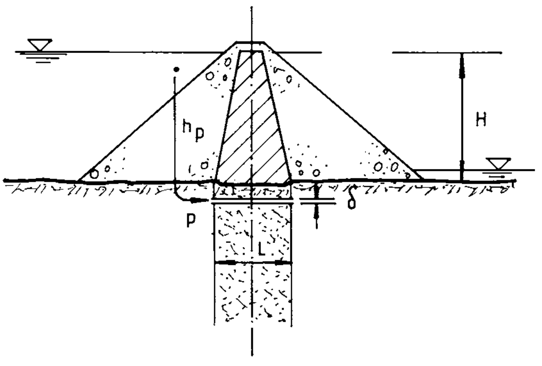


Fig. 3.17 Horizontal fissure in a grout curtain filled with grout

grout is calculated as $\tau_f = 550\ \delta$ in the case of a planar fissure and $\tau_f = 250\ \delta$ (kN/m^2) in the case of a tubular fissure. A strength of $\tau_f = 50$ kN/m^2 is sufficient to safely block the injected grout in a fissure 0.1 m wide, which is a low shear strength even for clay cement grouts.

High strength grout is needed when the deformability of the grouted domain should be reduced, eg. in consolidation grouting of rock foundation of high arch dams, or around pressure tunnels. In such cases the injected grouts should have a high deformability modulus, which is synonymous to high strength. It is nearly impossible to verify the strength of the injected grout on samples taken from the grouted fissures, and systematic tests should be carried out in the laboratory. There are no such tests described in the available literature. The procedure developed and used by the writer shall therefore be described.

In order to have test results which have some relation to the composition and the strength achieved by a grout injected in the fissures the samples should be prepared so that the process is a simulation of the fill forming in the fissures.Based on such a condition it was decided to prepare the samples also by grouting. The device for the preparation of a set of identical samples is shown in Fig. 3.18. It consists of a reservoir containing the grout suspension and a number of cylindrical molds with a

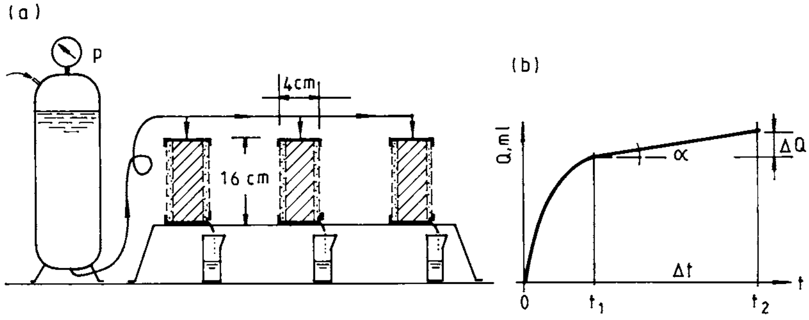


Fig. 3.18 Preparation of filtered grout samples, (a) test arrangement for a simultaneous preparation of several samples, (b) filtered water vs time, t_1 time of flow stabilization

pervious inner coating. The caps of the molds are connected to the reservoir, and the water drained from the pervious coating is collected at the bottom in graduated glasses. Before closing the caps, the lines and the molds are filled with the suspension to be tested. Then air pressure ***p*** is applied, and the suspension in the molds is gradually compressed and water filtered out, the filtered water volume being constantly replaced with fresh suspension from the reservoir. At the porous coating the suspension becomes denser and the thickness of the layer increases until a final equilibrium among the properties of the suspension and the applied pressure is achieved, and the flow of filtered water becomes constant. The samples are kept in the molds for some time under pressure (at least 30 min). The molds are then disassembled and the samples are cured for 7, 28, 56 and 90 days, when their unconfined strength is tested. Results from some tests are shown in Fig. 3.19, where the unconfined strength of samples prepared from the same suspension without filtering is also shown. The strength of all samples increases even 56 days after preparation. Filtered samples have much larger unconfined strength than unfiltered ones, their strength increases with the proportion of cement they contain. It should be mentioned that the density of the samples prepared by this method is not uniform in the cross section. The density is highest at the perimeter where the filter pressure gradient which compresses the deposit was highest and lowest at the center, as shown in Fig. 3.20. Thus the strength and other values determined from the test are average values which depend on the grout composition, the diameter of the sample and the intensity of the applied pressure. The same applies for the grout injected in the rock, the properties of which also vary with the width of the fissures. The described method is very useful for comparative studies of grouts of different compositions and for the selection of the best grout composition for a given project.

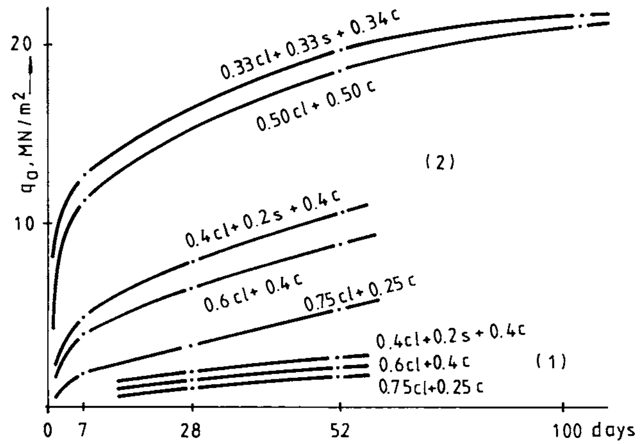


Fig. 3.19 Strength gain in time of unconfined unfiltered and filtered grout samples, 1 unfiltered, 2 filtered samples of the same suspension

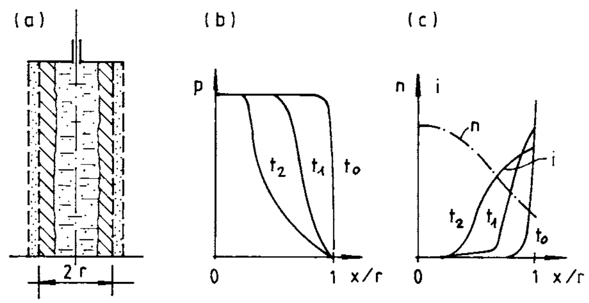


Fig. 3.20 Increase in grout sample thickness during filtering, (a) sketch of sample, (b) distribution of pressure along radius during time, (c) porosity and pressure gradient along the radius at different times

3.42 The permeability of injected grouts

The average permeability of the injected grout can be evaluated from the slope of the stabilized portion of the discharge/time curve shown in Fig. 3.18. By integrating the flow on radial paths from the center to the porous boundary of the sample, the following expression is obtained:

$$k = q\,(\ln r/r_o) \,/\, 2\,\pi\, L\, p \qquad (3.10)$$

with denominations as shown in the Figure and p is the pressure in cm column of water. The radius r_o can be computed from Poiseuille's law (Eg.3.1), but an estimate that $r_o = 1/100$ r to $r_o = 1/1000$ r yields a small range for k :

$$k = 0{,}73 \div 1{,}1 \ (q/L\,p) \qquad (3.11)$$

This may be considered as a rough estimate of the real value of k in the fissures, but it is sufficient for the comparison of grouts of different compositions. The permeability of the injected grout in the fissures varies in a much larger range anyway.

3.43 The resistance of injected grout, to erosion

The process of subsequent detachment of grains from the injected grout,by water percolating along not completely filled fissures,is called erosion. It may also occur at the exit of water percolating through the injected grout when the exit gradient is sufficiently large. Erosion may occur after water impounding a reservoir when the grout in the fissures has completely hardened, if its strength is to low, or when soon after grouting a stage the ground water table rises and causes an increased pressure gradient in the grouted region. In such a case, the early strength gain controls the evaluation of the danger of erosion, as well as the selection of the optimal composition of the grout.

The resistance to erosion is investigated on samples of grout prepared in the filtering device described in Item 3.41 (Fig. 3.18). A hole 8 mm diameter is drilled in the center of the sample, and it is fixed in the device shown in Fig. 3.21. Then water under pressure is circulated through the

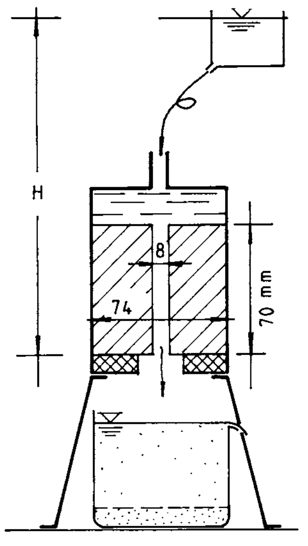


Fig. 3.21 Setup for testing erodibility of grouted compound

hole at some specified head or velocity. The sample is carefully weighed before and after the test, the difference in weight presents the quantity of eroded material that may be called the erosion parameter. The samples may be tested immediately after they were prepared, or after a period of curing, as the case being studied requires. This test does not yield quantitative, but merely indicative results which, nevertheless may prove useful for the selection among grouts of different compositions. It may also help to dispel some existing prejudices about the influence of an increased proportion of clay.

TABLE 3.3

Erosion parameter (% weight loss after test), sample tested after 6 hours 1W:1M, duration of test 24 hours

Project/mix	relative density ρ_a (g/cm^3)	% of weight loss at speed of water 0.7	3.0 m/s
Buško Blato			
0,2C:0,4CL:0,4S:1W	----	----	2.1
0,4C:0,4CL:0,2S:1W	----	----	0.9
0,2C:0,35CL:0,05B:1W	----	----	0
0,25C:0,75CL:1W	----	----	0
Meifedown			
0,5C:0,5CL:1W	>3.5	7.0	14.3
0,25C:0,75CL:1W	>5.0	1.5	5.7
Grančarevo			
1,0C:1W	----	0.4	----
0,35C:0,65CL:1W	----	0	----

In Table 3.3 the test results of some grouting projects in which the author was involved are shown. From the results shown it can be concluded that the erosion parameter of mixes with more clay and less cement is lower than in mixes with more cement. This may be explained by the fact that the strength of the injected grout in early stages depends mainly on cohesion and not on strength developed later on hydration processes of the cement.

3.44 The resistance of grouts to chemical deterioration

When the ground water or the impounded water at the site are chemically aggressive to Portland cement, injected grouts must be selected,which are resistant to chemical dissolution, alteration or their long term effects,which would deteriorate the strength of the grouts and their erodibility, or partially dissolve some of their ingredients. Although the permeability of most injected grouts is low, allowing only seepage of water at very slow speed, after a long period of time substantial deterioration of the grout may occur, thus reducing the efficiency of the grouted curtain. Even worse, water penetrating through not completely filled fissures may more intensively deteriorate the grout and cause deterioration of the grouted works in a much shorter time. In such cases it is necessary to make chemically resistive grouts. When the chemical composition of the ground water and the water which shall fill the lake is suspect, its aggressivity to the components of the grout should be investigated. Aggressive ingredients may consist of:

- sulfates which deteriorate calcium compounds of the cement;
- carbon dioxide which dissolves free lime of the cement, calcite minerals contained in some sands;
- humous acids;
- very soft water which dissolves calcite salts.

Some laboratory standards for testing the action of aggressive water on cement mortars require that mortar samples of various composition are weighed and kept submerged in the aggressive water for a few months. They are then dryed, weighed and visually inspected for integrity, development of fissures, the strength is measured etc. So the samples with best resistance to the aggressive water can be selected. This procedure is very slow, because the water initially acts only on the surfaces of the rather impervious samples.

Comparative results of the chemical resistivity of grouting compounds can be quickly obtained when samples of the grout compound to be tested are left to set and harden under desired conditions, and for a specified time which can be adjusted to simulate the actual conditions. The samples are then ground to a fine powder, which is then poured into the aggressive water. In such tests the active surface of the sample is extremely magnified, and relevant results are obtained in only a few days.

A parameter of resistance to aggressive water can be defined as:

$$A = 1/k\,\varepsilon \tag{3.12}$$

where k is the permeability of the injected grout as measured from percolation during the preparation of filtered samples (Eq.3.11), and ε is the proportion of salts in ppm/g dissolved from the sample, which is determined from a number of dissolution tests on the tested sample. It would be convenient to standardize such tests for studies of grouting mixes.

The test may be carried out on samples of the mix left to set for 24 hours, then hardened in an autoclave for another 24 hours. The samples are then dried at 50 °C, ground into powder and sifted on a 0.1 mm sieve. The powder is then well mixed in a laboratory jar with aggressive water in a proportion of 1 gr on 30 cm^3, and left for one hour. The amount of dissolved salts is determined by titration. The procedure is repeated 15-20 times with the same sample and a fresh portion of water, until the dissolved amount of salt becomes negligible. The total amount of salts dissolved from the sample is expressed in ppm/g.

This kind of tests were carried out in order to establish the most convenient grout composition for the grout curtain in the masonry of the Aswan Dam on the Nile (Nonveiller, Habeković, 1961). The dam constructed in 1900 as a granite masonry gravity wall exhibited intensive chemical corrosion of the cement mortar, which reduced the structural strength of the wall, and its permeability was progressively increased. For the grout curtain an injected grout had to be applied, which was to be proved to be resistant to leaching of CaO by the very soft Nile water. The leaching tests were carried out with grout compounds of cement with the addition of Kieselguhr and of bentonite. The test results are shown in Fig. 3.22(a) and (b). The influence of the water ratio of the samples was studi-

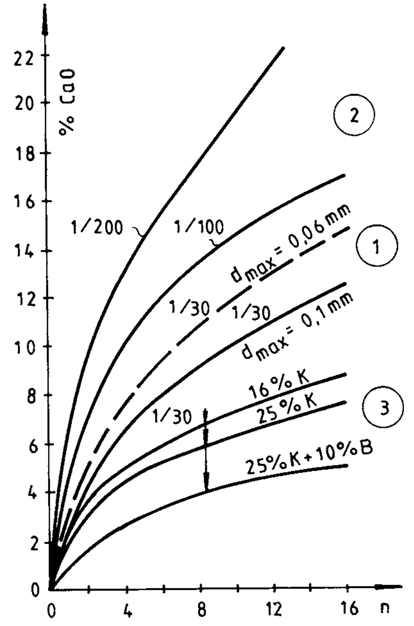


Fig 3.22 Chemical corrosion of grout compound, 1 influence of the maximum grain size of the powder, 2 influence of the concentration of the suspended powder, 3 influence of the Kieselguhr addition, CaO leached in % of cement weight, K Kieselguhr, B bentonite

ed, and it was decided to carry out the test in a concentration of 1 gr powder on 30 gr of water, and to repeat the procedure 30 times. The tests of grouts with Kieselguhr and a small addition of bentonite have provided an interesting result: the greatest resistivity to corrosion was achieved with samples containing some bentonite, although bentonite does not react directly with free lime contained in the cement grout.

3.45. The volume stability of injected grouts

When the use of suspensions with a high proportion of clay is discussed, the objection is often expressed that the injected grout could shrink in the fissures, and thus the grouted region would again become pervious, and inefficient. It was shown in laboratory tests that such an objection is not realistic for two reasons:

- the density of the injected grout in the fissures is greatly increased by the applied saturation pressure;
- the injected grout in the soil is continuously in a moist environment or below the ground water level, so water cannot evaporate and cause volume shrinkage.

Linear shrinkage is measured in the laboratory on samples prepared from a suspension of 0.6W:1M which is poured in molds 16x4x4 cm large. At the ends of the samples markers are fixed to define initial distance L_o between their ends. The samples used are then left to cure for some time in a saturated environment, or submerged in water. Then the distance between the markers is measured again as L_n. The linear shrinkage is then expressed as the percentage of the sample length L as:

$$\lambda = 100\,(L_o - L_n)/L \tag{3.13}$$

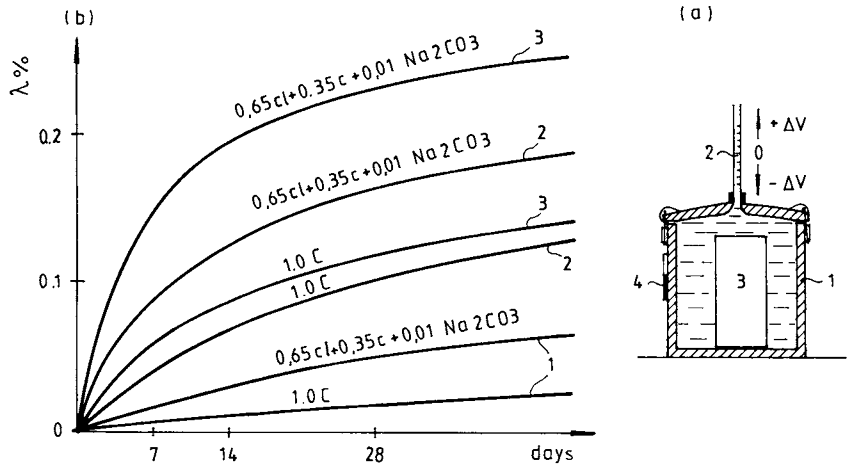


Fig. 3.23 Linear deformation λ % of grout compound samples, (a) picnometer for measuring volume changes of filtered grout samples, 1 closed jar filled with water, 2 graduated tube, 3 sample, 4 thermometer, (b) swelling of samples with time

This method is not precise and it does not include the volume changes from the beginning of setting of the cement.

An improvement can be achieved if the volume change of cylindrical samples is determined, which are prepared by filtering as described in Item 3.41. The samples are immediately placed in a special picnometer shown in Fig. 3.23(a) which is held at constant temperature during the test. The volume differences ΔV are precisely read on the calibration of a narrow transparent glass tube, and the percentage of volume shrinkage can be expressed in percent of the original volume V_o as:

$$\Theta = 100\,\Delta V/V_o \tag{3.14}$$

In Fig. 3.23(b) the measured linear shrinkage of grouts of different compositions for the grout curtain of the Grančarevo Dam is shown. All tested samples exhibited negative shrinkage, i.e. some swelling, which was more pronounced in samples containing more clay than in those prepared by filtering the suspension. Such trends were observed in all tests carried out so far. The experience gained from the observations and from leakage measurements carried out so far in the writer's practice confirm that the injected clay cement grouts are perfectly stable.

3.5. Solutions and resins

Solutions and resins are chemical compounds which in the injected ground form gels that fill the pores, thus reducing the permeability and increasing to some extent the strength of the injected region. The choice of such compounds for a specific grouting job depends on such characteristics as: the durability, the viscosity, the strength of the gel, a reliable control of the gelling time, the toxicity and, last but not least, the cost.

The durability of the gel formed in the injected voids depends on its composition and the possible chemical interaction with some salts dissolved in the ground water. As every gel contains a large amount of water, it shrinks when exposed to dessication in zones above the ground water table. The seepage of impounded water through imperfectly sealed zones, or at their outer boundaries, leaches out some chemically unbound elements from the gel, which slowly deteriorates the initial properties of the gel.

3. 51 . Viscosity

The penetrability of the compounds into the soil voids depends on their viscosity. The following relationship between the viscosity and the permeability of soils in Table 3.4 are rather conservative and purely informative.

TABLE 3.4

Relationship between the viscosity of chemical compounds and the permeability of groutable soil

Viscosity of solution or resin in cp	Limit soil permeability k (cm/s)
2	10^{-4}
5	10^{-3}
10	10^{-2}

The grain size distribution curves of groutable sandy soils are shown in Fig. 3.24 (Baker, 1982).It is evident that the silt content (grains < 0.06 mm) is critical, the soil containing 18% of silt is difficult to inject, and when it contains more than 25% of silt it can not be injected even with resins.

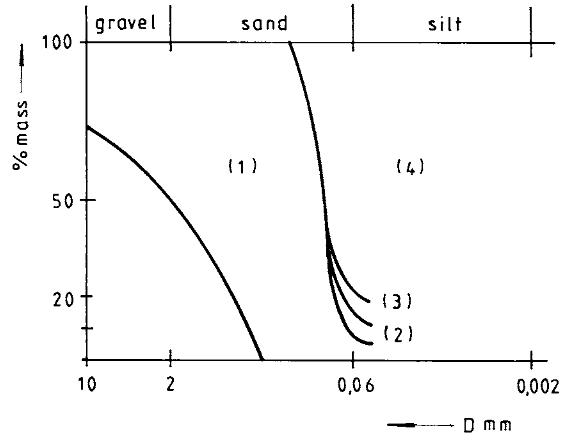


Fig. 3.24 Grain size distribution curves of groutable soils (Baker, 1982), 1 good, 2 difficult, 3 marginal, 4 unsuitable

Balat and Krisch (1982) have performed tests to determine the mechanism of penetration of a silicate grouting solution on model consisting of a 60x60x60 cm cube injected with a tube in the center. The cube was filled with three layers of compacted coarse sand with traces of medium and fine sand and some fine gravel, the uniformity coefficient of the sand was $c_u = 5$, the dry unit mass $\rho_d = 1.9$ g/cm^3, and the permeability coefficient $k = 1.4 \times 10^{-2}$ cm/s. In the first model shown in Fig. 3.2 5(a) a sodium silicate solution with a gelling time of 7.5 min was injected into dry sand. During the injection, differently colored solutions, as marked with A,B,C in the upper part of the Figure, were applied. The pressure applied for grouting is also shown. In the second test the same procedure was adopted, but the sand was saturated and the gelling time of the injected compound was 22 min. After the gelling process was completed, the cubes were cut through the middle section so that the zones filled consecutively with the differently colored compounds were visible. The results are shown in the lower part of Figs. 3.25(a) and (b). It is interesting to note that the distribution of the compounds with the shorter gelling time used for grouting the cube containing dry sand is rather irregular through the grouted region, and that the compounds mixed together. The compounds grouted in the saturated sand in which the compound with the longer gelling time was injected are more regularly distributed through the injected region and the consecutively introduced portions did not mix together. This indicates that the gelification of the already injected compound does not prevent further penetration of a subsequent portion of compound injected in the soil. The freshly injected compound pushes and penetrates through the already gelled one and mixes with it. When grouting in saturated soil the compound is diluted by the ground water at its boundaries, and its gelling time is increased, and penetration into the soil is more uniform. This evidence from a laboratory test is confirmed by practical experience that a compound can efficiently penetrate the soil after the gelling of the already injected compound has started. The same authors have studied the

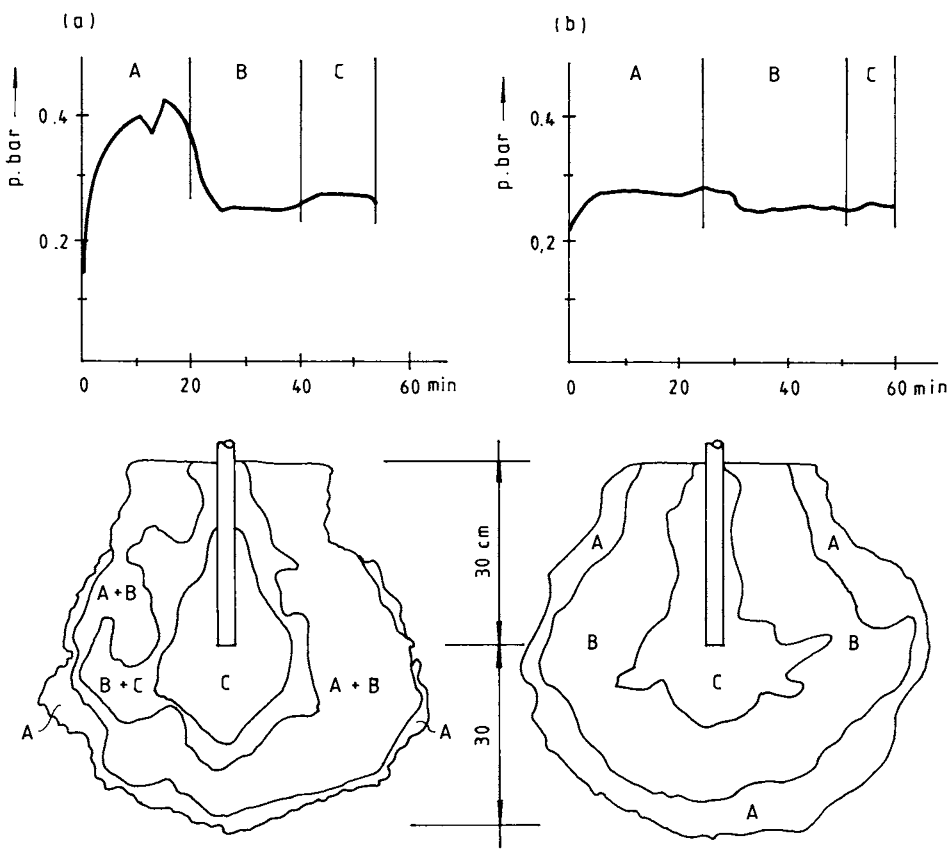


Fig. 3.25 Contours of differently colored silicate grout penetration in laboratory tests, (a) injection pressure, time and colors injected in dry sand, (b) in saturated sand

distribution of the injecting pressure in a cylindrical model shown in Fig. 3.26. A 60 cm long injection pipe with 13 holes 6 mm in diameter was placed at the center of a 22.5 ° wedge-shaped section of a cylinder. The pressure distribution within the injected zone at different gel times is shown in Fig. 3.26.

The gel time of the silicate solution was 24 min, the injection lasted for 47 min without notable pressure increase.

These results were confirmed in tests carried out by Krizek and Peres (1985). Sand was placed in a 3,05 long PVC tube 102 mm in diameter. At one end water was conveyed in the quantity needed to achieve the desired percolation velocity. A silicate solution of known gelling time was injected through a narrow pipe in the middle of the section at a distance of one third of the length of the tube. Injection tests were carried out with sands having different grain size distributions. It was determined that the gel time of the solution injected in the pores was increased because the solution was being diluted during the process. At some critical ratio between the velocity of water in the sand and the velocity of the injected solution, the solution becomes diluted so much that it stops gelling at all. The reduction in permeability achieved with grouting is the highest when the velocity of the percolating water is small and, at some higher velocity no reduction in permeability is achieved.

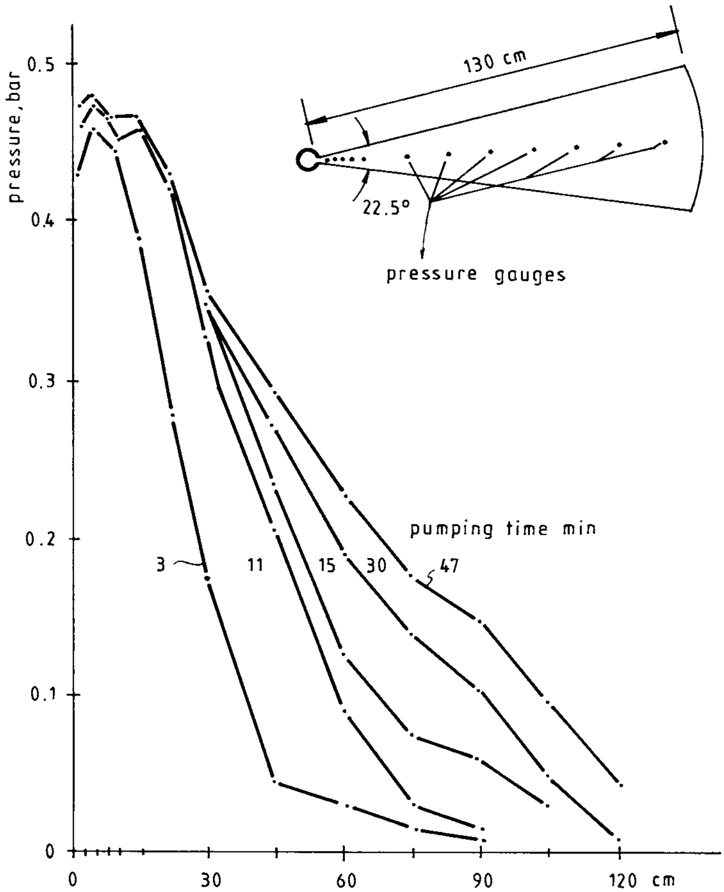


Fig. 3.26 Pressure distribution in injected sand model during time

3.52. The strength of chemically injected soil

The strength of injected chemical gels is low compared to the strength of cement injections, but the interconnected gel structure in the injected pores provides the assemblage of grains with some cohesive strength, while the shear strength angle Φ remains unchanged. Therefore the strength gain of the injected region is relatively large for low stresses and negligible for high additional loads.

The strength of injected soil is tested in the laboratory on cylindrical samples and it is expressed as the unconfined strength. Since loads at the surface cause some triaxial stress changes in the loaded region, it would be more appropriate to test injected samples in triaxial tests. All samples should be submerged before testing, as they are in nature. The strength of dried samples may be up to 10 times higher than the strength of wet ones.

3.53. The gel time

The gel time of chemical grouts depends on the selected reagent and on the concentration of the solution. With some combinations, the viscosity increase is slow, whereas it increases until it rises sharply to the final gel strength, so time available for pumping nearly corresponds to the gel time.

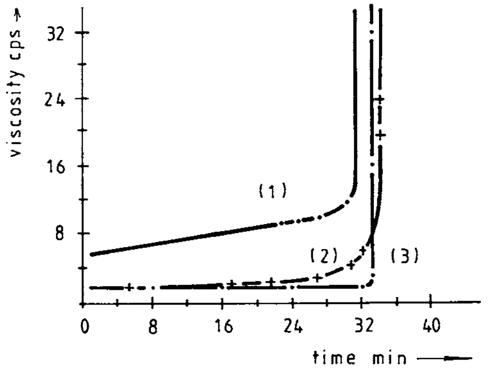


Fig 3.27 Change of viscosity of chemical grouts during time, 1 silicate solution, 50% silicate, 5% formamide, 5% etil acetate, 2 10% acrylamide solution, 3 AC400

In others, the viscosity increases gradually, so that only part of the gel time is available for pumping. Other factors influencing the gel time are the temperature difference between the solution and the injected ground, and the dilution during injection.

From typical results of some laboratory tests (Baker, 1982) shown in Fig. 3.27, it is seen that the viscosity of silicate solutions doubles from its initial value before gelling starts, and that the viscosity of resins is nearly constant, whereas it increases sharply when gel starts forming.

3.54. Toxicity of chemical solutions

This must be considered in two respects, the health hazard to the crews working with the chemicals and the hazard to the environment. Some chemicals used for grouting may be toxic, neurotoxic or cancerogeneous, irritant to the skin or corrosive. In their use, strict compliance with the producers' instructions should be observed, and the working site must be kept absolutely clean. Ground water may slowly leach, and the toxic components can pollute the ground water.

3.55. Economic aspects

The price of various chemicals for grouting may vary in the range of 1 to 20. The cost of preparation of the solutions and the injection may vary in the range of 1 to 3 for various chemicals, so that the total cost of the grouting project must be evaluated in order to decide on the most convenient alternative. In some cases other methods may be more convenient than grouting in order to solve the problems caused by ground water in civil engineering.

The ideal chemicals for grouting should satisfy the following requirements:

- it should be a powder soluble in water (in order to spare the cost of transportation of the solvent);
- it should be insensitive to the storing conditions;
- it should not be toxic, corrosive, explosive etc.;
- the solution should have low viscosity and it should be stable under normal temperature;
- it should be inexpensive, including the needed reactives and catalysts;
- it should not be sensitive to the quality of the ground water and it should stable at PH ≥ 7;
- it should form a stable gel of high strength.

A single chemical will never satisfy all these requirements, but its properties will lead to the selection of the most convenient one that will best meet the critical requirements of a specific project.

During the last 30 years a few hundreds of different chemical grout formulations were patented, but only a few of them were developed commercially. Silicates and acrylamides were used in the USA on up to 90% of all projects, and the rest of the market was covered by no more than six different patented products.

For chemical grouting, predominantly aqueous solutions of the following chemicals are used: sodium silicates, acrylamides, lignosulfites, phenolplasts and aminoplasts.

3.56. Sodium silicate grouts

Sodium silicate (called also water glass) is commercially available as an aqueous solution of mass density $\rho = 1.36$ g/cm^3 (1.38 ° Be). Its chemical formula is nSiO2.Na2O, and the silica/alcali ratio n may vary between 3 and 7. In the range of n between 3 and 4 it forms gels suitable for grouting when some acid or acid salts are added. For this purpose some organic compounds can be used as ethyl acetate combined with some detergent (French patent), or formamayde with some salt to regulate the gel time (USA Patent).

The strength of the gel depends on the silica content of the solution. In order to achieve an unconfined strength of 700 kN/m, a viscosity of 10 Cp and more is needed, which is hardly injectable. The viscosity can be lowered to 3 cp if only impermeabilization of the soil is intended regardless of the strength. These are initial viscosities which double within 3/4 of the gel time (Fig. 3.27). The strength of injected sand samples depends also on the rate of load increase, the strength increases with the rate of load increase. Most formulations yield gels which are regarded as stable in the ground, but this is doubtful for two reasons. Firstly, all silica gels exude water and shrink in the injected voids, the more so the lower the viscosity and the shorter the gel time. Voids, initially fully filled with gel, gain some water, and the permeability of the injected soil increases . Water percolating through the region then leaches free components out of the gel, which is weakened and may be fully destroyed.

Sodium silicate is not toxic and it does not present hazards to the environment, but some of the reactants may be dangerous and the instructions of the producer must be observed.

3.57. Acrylamyde grouts

A new era in chemical grouting technology started with the discovery of AM-9 in the USA in 1951, which has been commercially available since 1953. It is a white powder consisting of a mixture of two organic monomers which could be polymerized at ambient temperature into long molecular chains by means of a few percent of a cross linking agent. Solutions of up to 20% solids have viscosities lower than 2 cp. The gel contains mainly water (80-97%), it is stable in a moist environment and under water. When dried it shrinks to 10% of its original volume, but it swells to regain the starting volume if submerged again. Amonium persulfate is commonly used as a catalyst. Trietanolamine as activator and potassium ferricyanide as inhibitor for checking the gel time are mostly used. The catalyzed solution does not change its original viscosity through the induction time, then the gel stiffens suddenly, as seen in Fig. 3.27. The reaction is exothermic therefore the rate of the strength increase is higher in laboratory samples than in the injected ground. The unconfined strength of injected porous soil may reach values between 350 and 1,450 kN/m (Karol, 1982), but creep is rather pronounced when the stresses reach 1/4 to 1/2 of these values. This chemical is highly neurotoxic so its production was stopped in 1979. It was soon replaced on the market by a new product called AC 400 which is a mix of acryl monomers. It polimerizes with the addition of a cross linking agent (metilenbisacrylamide). It is not toxic, its viscosity is around 2 cp, and the permeability of the gel is about 5.10^{-9} cm/s. The induction time can be reliably controlled, and all other properties are similar to those of AM-9.

Acrylic gels are more expensive than sodium silicate ones and they are used when low viscosity and well controlled induction time are important factors for the success of the grouting work.

3.58. Other chemical grouts

Although 90% of chemical grouting is done with grouts based on sodium silicate and acrylic based grouts, some other compounds should be mentioned.

Lignosulphonate grouts are based on a waste product of wood processing in the paper industry. Its composition varies with the kind of wood and the manufacture process. Lignosulphonates are used for grouting in their original liquid state or dried to a powder, and the gel is formed with the addition of sodium dichromate (highly toxic). The concentration of the liquid for use is between 200 and 600 g per liter, the initial viscosity ranges between 3 and 8 cp, and the gel strength is similar to the strength obtained with acylamide grouts. The gel is stable if continuously submerged, the toxic dichromate may leach and pollute the environment. Lignosulphonate grouts may be an interesting alternative in countries with a developed paper industry.

Several other compounds like phenolplasts, aminoplasts, and polyurethane are seldom used for grouting in civil engineering works.

3.59. Foams

Foam is a gaseous emulsion of bubbles in polyuretane. The size and structure of the bubbles is controlled by some surface active agent. The diameter of the bubbles is uniform, about 80% of the bubbles are within 1.1 d and 0.9 d, ***d*** being their average diameter, which depends on the surface active agent used.

A characteristic property of foams is that the diameter of the bubbles is small. It flows freely through the piping and in the fissures until the grouting pressure decreases in free spaces causing expansion of the bubbles and the grout stiffens because of the high surface tension of the liquid phase.

Foams are characterized by two parameters:

- the expansion coefficient ***e*** is the ratio between the volume of the gaseous and the liquid phases of the suspension, and it is equivalent to the void ratio of soils;

$$e = v_g/v_l \qquad (3.14)$$

- the swelling coefficient ***f*** is the ratio between the final and the initial volumes of the foam;

$$f = (v_l + v_g)/v_l \qquad (3.15)$$

where $v_l =$ volume of the liquid phase;

$v_g =$ volume of the gaseous phase suspended at atmospheric pressure.

The foaming effect is negligible when $e < 0.2$, and when $e > 0.8$ the foam stiffens immediately upon cessation of the pressure. Foams with cementitious suspensions may reach $e = 3$, and organic foams as high as $e = 50$.

Foams can be produced in two ways:

- by adding surface active agents to the mix,
- by adding reagents which chemically produce a gas which emulgates in bubbles.

With the addition of surface active agents the bubbles generate immediately upon mixing, their size is uniform and less than 1 mm in diameter. Chemically reactive agents need 10 to 20 min and more to react,depending on the temperature, and the diameter of the bubbles is not uniform, the largest bubbles are more than 1 mm in diameter. Combining both methods, the range of bubble diameters can be increased, which influences conveniently the properties of the foam, especially its stability.

Foams are very efficient for plugging wide open spaces, caverns or wide fractures at a low consumption of grout mass, especially when these are percolated by water. The foam stiffens immediately after leaving the injection pipe, it is not leached or softened by water, and it can be used conveniently in combination with subsequent injections of thick cement suspensions or mortar.

4. GROUTING TECHNOLOGY

Grouting is a procedure which can not be strictly standardized because of the erratic nature and variability of the soil. Grouting consists of a sequence of different operations:

- drilling a hole of an appropriate diameter in the established arrangement and depth,
- preparation, proportioning, weighting and mixing of the selected grout suspension,
- injecting the prepared suspension into the designated section of the bore hole from which the fissures are filled.

There is a variety of procedures which may be applied in order to achieve the best grouting result economically, depending on the local conditions of the rock, which normally vary within a wide range. Practical decisions can be based on some general know how rules, but a bit of intuition based on past experience is nevertheless fundamental. Grouting is to some extent an art, for the best fulfillment of which a close cooperation between the design and the contractor engineers is essential. Both of them should be well acquainted with the means and the methods of grouting, they must have some feeling for the characteristics of the local rock properties which, in addition to the results of field explorations become evident during the progress of drilling and grouting activities. It is possible to standardize various phases of activities which are routinely carried out as shall be described in some details in Chapters 6, 7 and 8, so that the activities are rationally directed to the intended purpose - permeability reduction, increased strength and reduced deformability of the injected region. The effect that may be achieved by grouting is naturally limited to a lower bound, because pores and fissures narrower than some limit size can not be penetrated by the injected compound.

The lower limit of permeability which can be reached by means of grouting fissured rock is in the range of 1,0 - 0,1 LU so that the ratio of the initial permeability to the permeability of the grouted rock is the lower the lower the initial permeability. This applies also to the effect of consolidation grouting. Since the permeability of rock also indicates the possibility of penetration of the injected grout into the pores or fissures, it gives an indication whether under the natural conditions grouting is necessary or feasible. The art of grouting can be mastered only trough experience, careful observations and evaluations of the behavior of rock during the achieved injection. Strict compliance with a formalistically conceived grouting plan generally causes a waste of material or an unsafe structure.

4.1. Drilling and sequence of works

The first in the sequence of activities on grouting projects is drilling the injection holes at spacings foreseen within some limits in the design. The spacing of holes is then adjusted according to the actual results obtained on one or more test grouting plots injected at the site.

Rotary or percussion rotary drilling rigs are used for drilling with water as the drilling fluid, but in some cases the holes can be also air flushed. Usually some of the primary grouting holes are drilled with rotary rigs and cored, so that additional data on the composition and the geology of the strata are collected. Rotary drilling is two to five times more expensive than the percussion drilling. The cost can be lowered and the drilling speed can be increased by using non coring instead of coring bits causing the crushing of the material which is then carried to the surface by the flushing water.

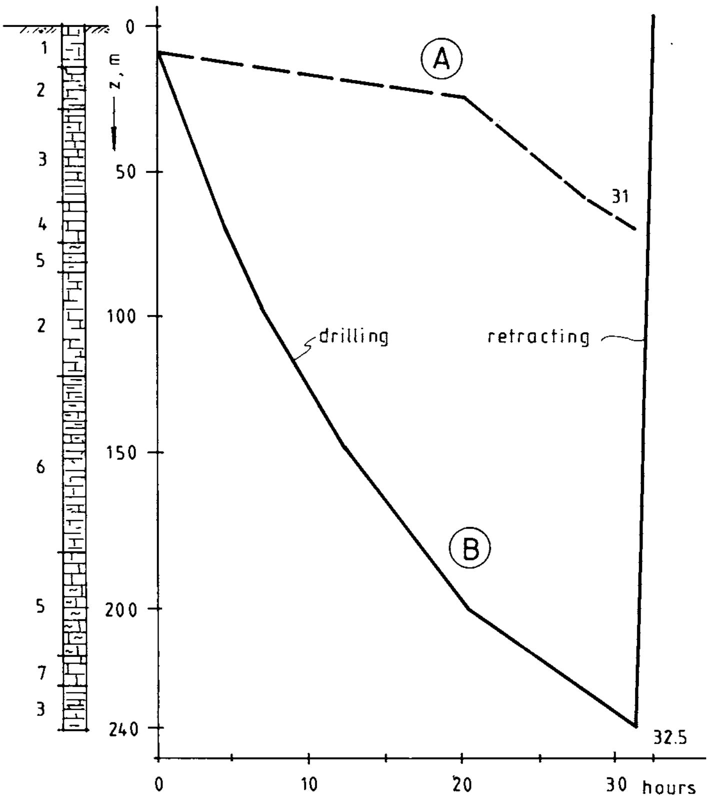


Fig. 4.1 Drilling progress with (A) rotary and (B) percussion rig in same formation; 1 clay with crushed limestone, 2 marly limestone, 3 soft marly limestone, 4 medium hard limestone with chert, 5 hard marly limestone with chert,6 hard limestone with chert, 7 soft limestone with chert

On grouting projects mainly rotary percussion rigs are used with which drilling speed of 20 m/h can be reached and depths up to 180 m. In Fig 4.1. the drilling speed achieved on a test grouting plot in the Lebanon (Fingerhut, 1971) with a rotary percussion rig is shown and compared to the speed obtained at the same site with rotary drilling. More details on rigs and accessories are given in Chapter 5.

The grout is introduced into a section of the hole with the length *L*, to be injected by means of a pipe at the lower end of which a plug called ***packer*** is fixed, which isolates the grout from the upper section of the hole, as shown in Fig 4.2. The length of the section *L* is chosen to match the permeability of the formation and the pumping capacity. If the permeability is low, *L* may be as much as 10 m in homogeneous rock, and in highly fractured and permeable rock it may be as low as 1 m. Generally it is chosen as L = 5 m.

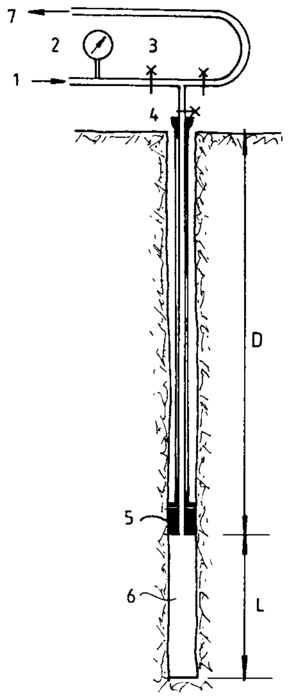


Fig. 4.2 Bore hole with standpipe and packer for grouting section L, 1 injection line, 2 manometer, 3 regulating valves, 4 screwing device, 5 packer, 6 injected section, 7 return line

The diameter of injection holes has no influence on the effectiveness of grouting. Since drilling smaller diameter holes is cheaper and faster, the smallest diameter suitable for the maximum hole depth is chosen. The extremes are between 40 mm and 100 mm diameter, mostly 46 to 56 mm, according to European Standards, will be satisfactory.

The sequence of drilling and grouting can be from top to bottom or from the bottom up. In the first case the hole is drilled to the bottom of the next section to be grouted (Fig. 4.3). The rig is removed and the grouting tube with the packer is inserted. When the packer is fixed at the top of the stage the section is injected to refusal. Then the packer and the grouting tube are removed, the drill is placed again, the grouted section and the next one are drilled, and the procedure is repeated until the anticipated depth of grouting is reached. This method of grouting is applied in highly fractured or in

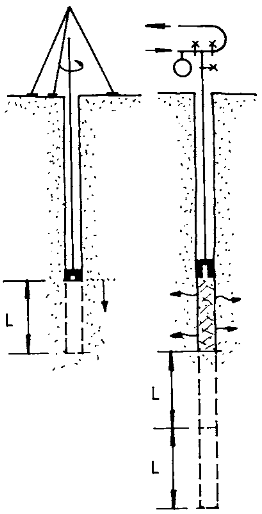


Fig. 4.3 Drilling and grouting in sections form top to bottom

where the bore hole walls may collapse and block the equipment. When the formation is compact the hole may be drilled in one operation to the bottom, the grouting is started in sections, subsequently from the bottom to the top, as shown in Fig.4.4.

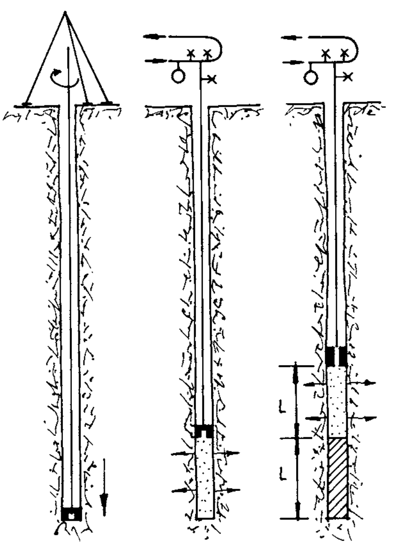


Fig. 4.4 Grouting in sections from the bottom up

The working progress is faster, the only danger is from grout bypassing the packer and filling the upper part of the hole, because it is always fixed in ungrouted rock. In order to avoid such a danger, a hose for flushing the packer should be lowered into the hole.

In some cases a combination of both methods is convenient. The hole is drilled to a depth which can be reached without danger of collapsing the walls. Then it is injected upwards in stages, redrilled and drilled to the next feasible depth, and injected upwards in stages. Thus the number of moving the equipment and setups can be reduced too.

In some cases (mainly in the USA) a different approach, which may be called ***zone grouting*** is used. The hole is first drilled to a modest depth, as shown in Fig. 4.5, then a pipe is set in cement mortar, and the hole is drilled to the depth of the first injection zone. The circulation line is then connected to the pipe and the zone is injected at a moderate pressure, so that leaks to the surface are

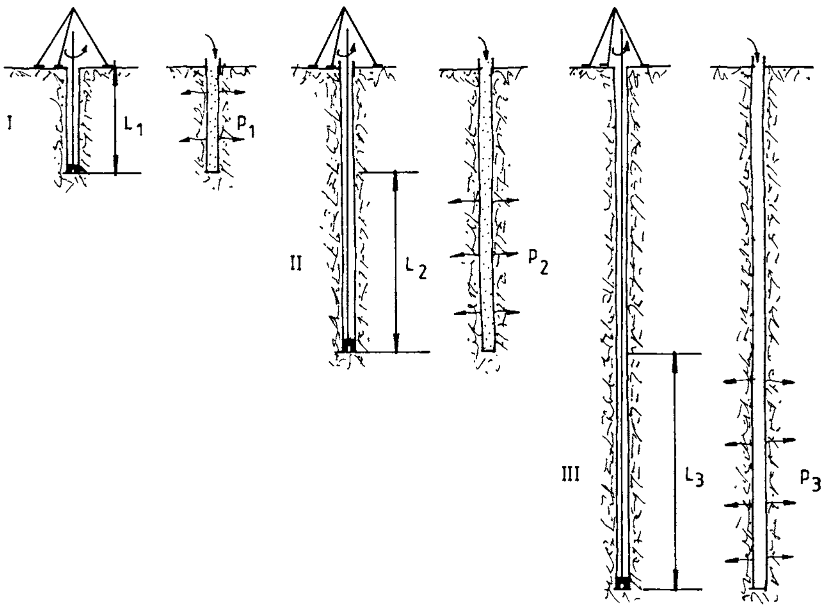


Fig 4.5. Grouting in zones from top down, zones I - III

prevented. After the grout has set, the hole is redrilled to the depth of the subsequent zone, the pipe is connected to the circulation line, and the whole depth is injected again, but with a higher pressure, so that the upper zone is also regrouted. The procedure is repeated until the final depth of grouting is achieved. Usually the depth of grouting is divided into three zones. It is assumed that by regrouting the upper zone at higher pressure, together with the deeper one improves the grouting result. This is doubtful in many cases, because the grout composition and the injection procedure must be selected to suit the newly grouted zone, which may not be the best suitable choice for the already grouted zone exhibiting low permeability. In the European practice grouting in sections is always preferred.

The number of rows of grouted holes depends on the needed thickness of the grouted zone and on the initial permeability of the formation. For practical reasons an uneven number of rows should be used. If only two rows of holes are injected, as shown in Fig. 4.6(a), the result is effectively the same as the one achieved by increasing the spacing between the holes. When three rows of holes are grouted, which is convenient in formations of high permeability, the external holes are injected first with lower saturation criteria, the internal row follows which can then be grouted at higher pressure and saturation criteria without excessive grout waste.

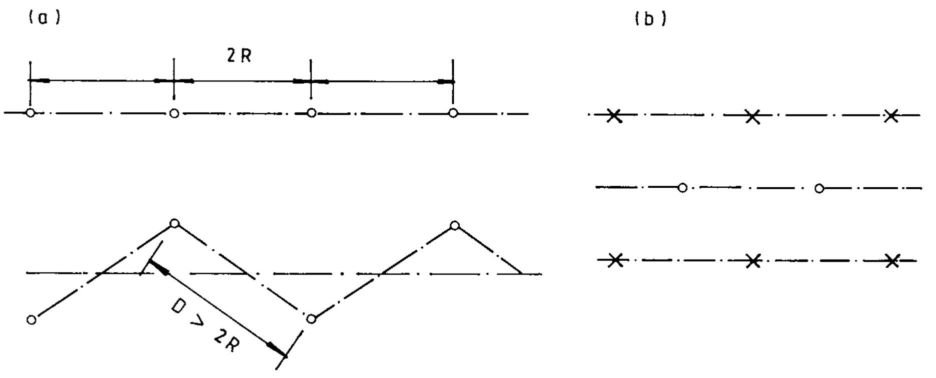


Fig 4.6. Grouting density, (a) in a single and in a double row curtain (c) in a three row curtain

Experience has shown that the sequence of grouting different holes influences dominantly the end result. The approach mostly used today is that of "*split spacing*" the distance between subsequently injected sets of holes. The first set of holes called the primaries are drilled and injected at a distance which exceeds the expected average reach of the injected grout. The next set of holes is then injected in the middle of the spacing between the primaries, the sequence is repeated again with the tertiaries in the middle of the spacing between the secondaries and so on, until the last set drilled shows that the permeability or the specific grout take is lower then the specified value. The basic distance between the primaries is determined from previous experience in similar ground conditions, or based on the results of one or several grouting test plots. The distance should be selected so that the specified result of grouting is achieved mainly after injecting the tertiary holes at a spacing between 3,0 and 1,5 m. The closure spacing depends on the character of jointing and fissuring, the grout composition, the injecting pressure, the specified criteria of saturation and on the initial permeability of the formation. Quaternary holes should only be occasionally needed, provided the distance of primaries is well chosen. In this procedure the sets of holes of higher order should always be injected after all holes of lower order are completed. The holes of higher order can be drilled and grouted before the previous ones have reached the final depth, but these should be injected at least two sections deeper than those of the new set.

The evaluation of the injection results achieved in the last set of holes can be based either on permeability measurements before injection, or on the specific grout consumption. The first alternative requires permeability measurements in new check holes between the injected set of holes, which is time consuming and expensive. The second alternative is based on some practical experience and the classification of specific grout consumption, as proposed by Deere (1976) and shown in Table 4.1. The grout consumption is ranged from very high to very low, the specific consumption in the last set of stages should be low or very low. The advantage of this alternative in the decision making is that the decision whether or not to inject the next higher order set of holes can be made as soon as the results of the previous set are available, without any additional testing.

TABLE 4.1

Specific injection grout consumption (Deere, 1976)

Designation	specific grout consumption kg/m
very high	>400
high	400 - 200
medium high	200 - 100
medium	100 - 50
medium low	50 - 25
low	25 - 12.5
very low	<12.5

It should also be remembered that in order to fill 1 m of hole of 70 mm and 46 mm diameter resp. about 10 kg/m and 5 kg/m of injected grout is required resp. So any grout take less than 10 kg/m is a complete waste of time and money.

Whenever the drilling bit strikes an inclined discontinuity the axis of the hole may deflect from the vertical axis and the bottom of the hole will not meet the required position, the spacing between the holes is the more altered the deeper the holes . It takes great skill and care to prevent such unintended ***hole deflections***. Sometimes great importance is placed on the strict control of the ***verticality of the holes*** and some limits to the deflection are specified (e.g.in Yugoslavia, USSR). In the USA the drilling holes deeper than 20 m is avoided whenever possible, in case of deeper holes grouting galleries are excavated from which again 20 m deep holes are drilled. In the relevant publications this item is seldom discussed or elaborated on. In fact the reach of the injected grout increases with the applied grouting pressure, which increases with depth, where the formations are less fractured and the hydraulic gradient is much lower deeper down than at the surface, so that the exact location of the hole at the bottom should not be very important for the overall result of grouting. This is confirmed by observations made on grouting projects, that the depth of secondary holes is generally smaller than the depth of the primaries, and the depth of tertiaries is smaller than the depth of secondaries. It shall therefore be assumed that it is sufficient to specify measures which are able to control the hole deflection reasonably. The causes of uncontrolled hole deflection are mainly:

- inclined or not steady fastened drill rig,
- use of bent drilling rods,
- too wide clearance between the wall of the hole and the diameter of drilling rods (e.g. 33 mm rods in a 56 mm hole),
- drilling through heterogeneous formations, with open wide fissures.

The hole deflection can be very much reduced by an appropriate choice of the drilling equipment and by careful drilling which is not expensive, although it may somewhat slow the progress. Specifications of strict tolerances, with penalties for not complying, forces the contractor to cover the risk with higher unit rates. When geologic conditions cause deflection, it is in the same direction in all the holes, resulting in little change of their relative location at the bottom. The experience on a very carefully done deep test grouting plot in the Lebanon shown in Fig. 4.7 (Fingerhut, 1971) is instructive. The holes were 240 m deep, the deflection of every hole was checked. Rotary and percussive

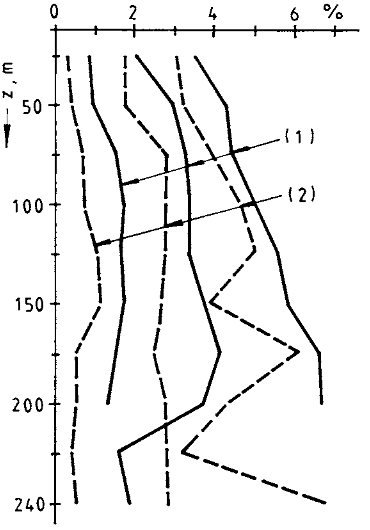


Fig. 4.7 Deviation of holes from the vertical axis, injection test plot Meifedoun, 1 percussion drilling, 2 rotary drilling

rigs were used for drilling, and special care was devoted to the deflection control during the drilling. The formation was bedded jointed Eocene limestone containing chert and beds of marl. Evidently the deflection was smaller with rotary than with percussion drilling, generally it increased with depth. The minimum deflection was about 1% of the depth with rotary and 2% with percussion drilling, the maximum was 6,5 % of the depth in both cases, the average was 2,8 % with rotary and 4,1 % of the depth with percussion drilling.

4.2. Grouting compounds

The choice of the appropriate grouting compound for a specific project depends on the characteristics and the properties of the ground, and on the purpose to be achieved by grouting. There is a great variety of the ground characteristics , the properties and the behavior of grouting compounds which would be difficult to describe systematically, therefore a tabular presentation in Table 4.3 is preferred.

From tables 4.2 and 4.3, as well as from the diagrams shown in Fig. 3.2 and 3.3, it may be concluded that:

- cement suspensions are best suited for injection for the purpose of reducing the permeability and the deformability of fissured rock foundations,
- cement-clay suspensions with sand and some bentonite or other fillers are appropriate to reduce the permeability of fissured rock,
- mortar with cement and fillers to inject wide fissures and small caverns,
- cement-bentonite and bentonite suspensions to inject very porous materials such as gravel and coarse sand,
- only chemical solutions and resins to inject sandy beds, or with a previous injection of cement bentonite suspensions,
- foams and combinations of foam, thick mortar and other fillers to stop water flowing trough wide fissures or crevasses.

TABLE 4.2

Limits of application of various suspensions and solutions according to permeability

Type of compound	Description	Range of use	Injection control (Cambefort,1977)	Relative cost
cement suspensions	unstable, stable	Fissured rock, $k > 5x10^{-5}$cm/s	to saturation, specified pressure and time	1 - 4.2
cement - bentonite, bentonite suspensions	stable	gravel and sand $k = 10^{-3}$ cm/s	to saturation, specified pressure and time	1 - 1.2
chemical solutions,	dense diluted	$k > 10^{-3}$ cm/s $k > 10^{-5}$ cm/s	specified injected volume	2 - 6 10 - 500

TABLE 4.2

Review of ground characteristics, grout compounds and injection techniques

Purpose of grouting	Ground characteristics	Grouting compounds	Injection techniques
Reduction of permeability of soil and fissured rock	Porous media with small interconnected voids, low to medium porosity, as sandstone, fine sand, sandy gravel	Chemical solutions, resins, bituminous emulsions, suspensions of bentonite and colloidal cement	Quantity controlled injection into short 30-50 cm long sections, sleeve injection with double packer
	Soil with large interconnected voids, high permeability, as uniform medium sand, gravel	Thick cement, clay and bentonite suspensions	Quantity controlled injection, 30-50 cm long sections, sleeve injection, reinjected as needed
	Fine interconnected fissures, medium to low permeability	Thick cement, clay and bentonite suspensions	Downstage or upstage grouting in 3 to 10 m long sections to specified saturation criteria
	Rock containing many narrow interconnected fissures, large permeability	Stabilized cement bentonite suspensions, chemical solutions	Downstage grouting in collapsibile holes, else upstage, sections 5 m long or shorter
	Highly fractured rock, high permeability	Thick cement and clay suspensions with some fine or medium sand, addition of bentonite for thixotropy	Downstage grouting,5m long sections or less, breaks for setting if consumption exagerated or surface leaks
	Highly fractured rock containing wide open fractures and/or karstification channels	Thick cement clay suspensions with bentonite for thixotropy, if take excessive followed to saturation by plastified cement mortar	Separate injection of identified extremly pervious zones, injection breaks for setting, injection of thick mortar by means of special pumps

	Very much karstified rock with many open fissures	Thick plastified cement mortar, thick thixotropic suspensions for final saturation, foams	Separate injection of caverns or open wide fissures, saturation of sections with thick thixotropic grout
	Crushed rock injected grout leaks to the surface	Thick thixotropic suspensions with fine sand added, additives for quick setting	Intermittent injection, caulking of surface leaks, wedges, reinforced concrete slabs etc.
	Injected grout leaks into adjoinging open holes, flowing water in fissures erodes injected compound	Thick or/and thixotropic suspensions, pouring one grain gravel (pea gravel), subsequently injected with thixotropic suspensions; foams folloved by cement suspensions	Simultaneous injection of both holes, following ground characteristics, large holes for high capacity pumping
Reduction of deformability of rock or soil and/or strenght increase of injected region, foundations around tunnels and underground spaces, caverns etc.	Small interconnected voids, capillary fissures, low permeability	Chemical grouts or resins	Specified quantity of solution injected, short sections, sleeve grouting in soil
	Small interconnected voids, medium to high permeability	Colloidal cement suspensions, combined with resins if efficient	Specified quantity injection, short sections, sleeve grouting in soil
	Wide empty joints or fissures, high permeability	Thick stabilized cement suspensions, fine sand added as needed	Injection in short sections to saturation
	Narrow or wide joints and fissures, filled with sand or clay	Cement suspensions or mortar to match character of joints and fissures	Water and air flushing fissures from surrounding holes followed by injection to saturation of whole flushed group

4.21. Compounds for injecting fissures and joints

For a long time only Portland cement was used for grouting and it was injected in unstable suspensions. Generally it was maintained that suspensions should be dilute and unstable in order to best fill the injected fissures, a view often supported by grouting specialists to this date. More light on this matter was shed by tests carried out and published by Houlsby (1982). It was shown that fissures injected with thin unstable pure cement suspensions were not completely filled with cement, and that some channels filled with clear water still remained after grouting. Through these channels the surplus water is carried away during the injection, and the resulting fill in the fissures is porous and pervious. In the cement paste, hardened by the surplus water, remains a lot of free lime, which is later leached out by water percolating the porous fill in the fissures. A sketch of the infill of fissures injected with a diluted cement suspension is shown in Fig. 4.8 from observations made with a bore hole periscope in two check holes, both injected with 1C:5W suspensions. The first is from Blowering Dam taken in 1972, four years after the completion of the grout curtain, parts of which were already completely leached out at that time. The second is from Copperdole Falls Dam, where water was flowing from the opening in the fissure during the examination.

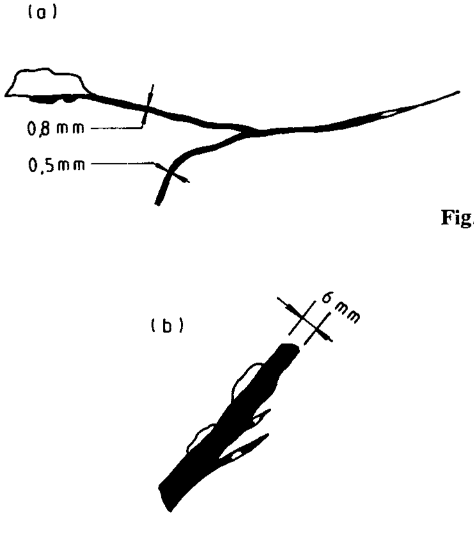


Fig. 4.8 Partly filled fissures injected with 1:5 suspension, sketches from observations made with a bore hole periscope, (a) Blowering Dam, (b) Copperdole Dam (Houlsby, 1982)

It is now generally accepted that the suspension to start the injection should not be thinner than 1C:3W, in many cases 1C:2W or denser suspensions are specified and they were successfully used. Lombardi (1985) recommended to inject even narrow fissures with suspensions 1C:0.6W, which is supported by the fact that circulation of the suspension in the fissures is not governed by their viscosity, but by the maximum grain size of the cement particles. It was explained in Item 3.21 that this (the maximum size of cement particles) can be substantially reduced by intensive mixing in high turbulence mixers.

Another subject for debate among grouting specialists is the use of stable or unstable suspensions. Cambefort (1977) maintains that cement from an unstable suspension settles during the injection and that it fills easily even wide fissures, while a stable suspension which has filled a fissure will nevertheless shrink a little after the injection is completed, so that the process must be repeated in order to properly fill the fissures by additionally injecting some unstable suspension. A lot of data and experience show that the fill in fissures injected with stable suspensions is consolidated by the gradient of the injection pressure and the seepage force during the saturation stage, the resulting fill in the fissures is much denser than the initial suspension, and no further volume change occurs after setting. Fig. 4.9 shows the example of a wide fissure injected with a stabilized cement clay suspension in the jointed cretaceous limestone at the Peruča dam which was disclosed after the excavation of the foundation trench for the grouting gallery. Such a stiff fill could not be achieved without water being filtered out of the 1C:CL:1W suspension used for grouting. Based on such positive experiences with grouting in the world and in Yugoslavia,the selection between cementitious grout suspensions can be suggested, as shown in table 4.4 .

Fig. 4.9 Fissure filled with clay/cement grout in the excavation for the Peruča Dam foundation

TABLE 4.4

Cementitious grout suspensions for various applications

Type and permeability of rock formation	Composition and properties of suspensions
<u>Curtain grouting</u>	
very high permeability, wide fissures N < 50 LU	cement-clay, some bentonite and sand proportions down to 0.25C:0.75CL:bB: nS:(1 + b + n)W and denser
N > 50 LU	same as above, increase bentonite proportion on expense of clay and sand to get thixotropy
high permeability, narrow fissures, N > 30 LU	cement-clay-bentonite proportions about 0,40(10.40-b)C:0,60CL:bB:3W
low permeability, narrow fissures, 10 < N < 30 LU	cement-bentonite suspensions (1-b)C:bB:3W
very fine fissures N < 10 LU	activated and cyclonized cement-bentonite suspensions, microcement
water percolated wide fissures, open spaces, caverns	plastic cement-sand: bentonite mortar, fillers (Item 3.23), foams
<u>Consolidation grouting</u>	
wide fissures	cement mortar with some bentonite for plastification
narrow fissures	stable cement suspensions with additives

The following comments may be added:

- sand is added as the cheapest filler when fissures are generally wider than 1 mm, it increases the strength of the injected grout;
- bentonite contains the smallest solid particles which easily penetrate very fine voids and fissures, added in smaller proportions it does not reduce the strength of the injected grout, larger proportions of bentonite cause thixotropy;
- clay is used as a filler if it is cheaper than cement, it reduces the permeability of the injected compound, improves durability of the fill especially if the ground water is aggressive to portland cement.

It is a good practice to select the initial density of the grouting suspensions, as it was suggested by Houlsby (1982), depending on the average width of the fissures as shown in Table 4.5.

TABLE 4.5

Initial density of grout suspensions

Average width of fissures mm	Water ratio
<1	3 : 1
≈1	2 : 1
>1	1 : 1

During the saturation phase, when grouting very pervious formations, it is often necessary to use suspensions denser than 1:1.

4.22. Plugging wide spaces and caverns

This kind of work requires a careful study before it can be decided on grouting as a feasible option. Whenever possible, detailed speleological investigations should be carried out, so that the best planing can be done and the most suitable method of work can be devised. When grouting is a feasible option, the method to be applied depends on whether or not the space is percolated by water . Dry spaces and caverns and those filled with water, but not percolated by it, are best pumped full with plastic concrete through a number of wide diameter holes. Smaller caverns or crevisses may be filled with plastic mortar by means of helicoidal pumps. Care should be taken not to pour concrete or mortar through water which would wash out the cement, It should be conveyed to the bottom trough pipes which are gradually raised to the top as the space is being filled (the tremie method).

Careful planning is needed in the case of caverns percolated by water which would wash out the injected compound. In the worst case the space is first filled with uniformly graded medium gravel. Then grouting lines, open at the bottom, are placed and a dense stabilized plastic cement mortar is injected through them until it fills all the voids of the gravel. The percolation is reduced at critical locations or zones by the gravel fill, so that the cement grout is not exposed to erosion as it would be without the fill. Some examples of such applications are given in Chapter 6 (Buško Blato, Keban, Sklope).

4.23. Grouting compounds for granular material

Granular materials contain a network of interconnected void spaces of different sizes located between the grains of the material. The size and the distribution of voids can be related to the grain size distribution curve of the material. Sherard et al. (1984) have described a method to measure the interlaced structure of the voids. A sample of gravel of 9,5 - 25 mm in diameter, with D_{15} =11 mm was vibrated into a dense state in a cylindrical container, then molten wax was poured which, filled the voids completely. The container was removed after the wax was cooled, and the sample was cut into several pieces; the gravel was pried out and a network of wax with the configuration of the voids was left. From the study of the wax skeleton the following relationship were established:

- over the length of about $10D_{15}$, the maximum and the minimum sections of the highly irregular flow channels are repeated many times, and there were not any dominantly large or small flow channels,
- the minimum and the maximum dimensions of the flow channels ranges between $0.1D_{15}$ and $0.18D_{15}$,
- the equivalent average dimension of the flow channels was estimated to be between $0.25D_{15}$ and $0.35D_{15}$, the maximum dimension was everywhere less than $0.6D_{15}$.

Although these qualitative results were obtained on a rather small number of samples, it is interesting to note that they agree well with the practical experience that the maximum particle size of a groutable suspension should be less than 1/3 of the average void size (Bell, 1982). The range of the grain size distribution of the injected cement is as shown in Fig. 3.1. These relationships give a general orientation for the selection of cement and clay suspensions for injecting alluvial soils of a given grain size distribution curve. Suspensions with maximum particle size $\boldsymbol{D_S}$ can safely penetrate voids of a compacted soil when $D_S = 0.1\ D_{15}$. For saturation of the voids in a limited reach $D_S = 0.2\ D_{15}$. Cambefort (1977) maintains that the permeability coefficient $\boldsymbol{k}$ is a sufficient criterion for the evaluation of the groutability of soils.

The purpose of grouting, which is either permeability reduction, increase of strength or reduction of deformability, is another criterion for the choice of the grouting compound. High strength of the injected compound is not necessary for ***permeability reduction*** of rock (see Item 3.22) so cement-clay - bentonite suspensions with a low proportion of cement are satisfactory for this purpose. For ***consolidation grouting***, high strength of the injected compound is essential, and this is satisfied with a cement suspension with a low bentonite content.

Suspensions of very fine grained compounds (cement and clay) are needed for grouting granular soils. Cement should have Blaine fineness more than 4.500, and D_{85} of the clay should be less than 0.03 mm. When such materials are not commercially available they can be fabricated inexpensively in a properly equipped batching and mixing plant at the site. It is essential to use high turbulence activating mixers for the preparation of suspensions, and a hydrocylone at the exit of the mixer for the separation of coarse particles. Fine grained cement can also be produced at the site from a coarse grained one by wet milling and processing the resulting sludge in the batching plant. The best arrangement depends on the local conditions.

The initial permeability of granular materials can be reduced by about one order of magnitude when injecting suspensions; solutions can reduce it by two orders of magnitude, and for this purpose mainly silicate based compounds are used because of their low cost. Reinjecting of already grouted hole sections is possible when the ***sleeve injection*** method (tube à manchette) is used. In the first run cheaper cement-clay suspensions are injected to fill the larger flow channels, followed in the second run by solutions. If needed, intermediate holes are drilled, equipped with sleeve tubes and injected with solutions.

4.24. Grouting compounds for other applications

Prestressed anchors

High strength of the injected compound is required in the fixed section of rock anchors, or of prestressing bars in concrete structures which is attained when a thick cement mortar is injected with additives to provide adequate plasticity and fluidity, so that it can penetrate irregularities and fissures at the interface. These properties are obtained using low water:cement ratio mixes with additives which are commercially available for fluidification, plastification and air entraining agents normally used to prepare pumpcrete.

Joints and fissures in concrete structures

Joints between blocks of mass-concrete structures may be anything between 0.01 mm and a few mm wide. Stabilized cement suspensions are adequate for the purpose, but fine grained cement is to be preferred in order to have the injected suspension can penetrating fine cracks less than 0.1 mm wide. Some small addition of activated bentonite is favoured, since it prevents the injected grout from shrinking after setting. The mixing ratio should be in the range of 1C:2W. When very fine cracks have to be filled, chemical compounds, preferably resins, have to be used, sometimes as a second run following the injection of a suspension cement.

Restoration of defective masonry and concrete

For the restoration of defective or decayed structures injection of cement compounds is sometimes used either for remedying structural defects or to achieve impermeability. Cement suspensions are injected to fill narrow fissures or voids, plastic mortar to fill wider cracks and spaces. Surface leaks must be prevented by previously treating the surface with mortar. For very fine cracks chemical compounds, preferably resins, are injected.

Lifting and levelling of leaning structures

In such applications of grouting, penetration of fine fissures is not required,so that less expensive cement based compounds can be conveniently used, such as cement mortar with fine sand, clay, some bentonite for plastification, and air entrainer for fluidity. The addition of a set retarder may be appropriate in order to prevent setting before the expected result is achieved. The final strength of the injected compound is not a factor which governs the grout composition, but since only low injection pressure can be applied, the water: cement ratio should be low.

4.3. <u>The grouting pressure</u>

The efficiency of grouting depends to a high degree on the correct saturation pressure which designates the maximum pressure at which a grouted section of a bore hole is being saturated. It is normally regulated in the technical specifications for the performance of grouting projects. If the saturation pressure is too low, the injected grout will permeate only a small volume of material around the injected section. In order to have a continuous rock permeation with grout, the spacing between the holes must be small, when the specific drilling length (m/m^3 or m/m^2) is large. Since drilling is expensive, the total cost is increased. When the saturation pressure is too high, wider fissures are opened, new ones may develop by hydraulic jacking, creating new voids which are then filled with grout. This causes an unnecessary waste and longer working time with the equipment. The proper value of the grouting pressure should be established between these extreme values depending on the rock conditions (permeability, nature of fissures, strength) to best meet the purpose of grouting (impermeability strength, deformability).

The injection pressure p is recorded on the manometer at the injection standpipe on the top of the packer assembly. The pressure p_i is increased in the injected section by the weight of the suspen-

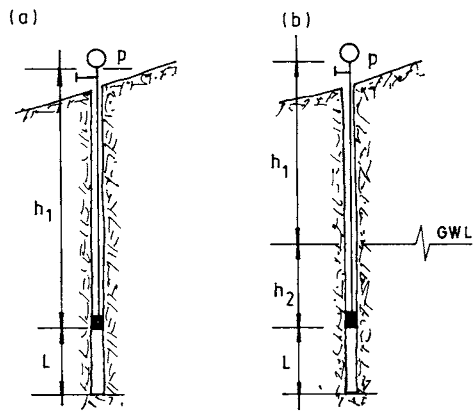


Fig. 4.10 Denominations for the computation of injection pressure in grouted stage, (a) above the ground water level, (b) below the ground water level

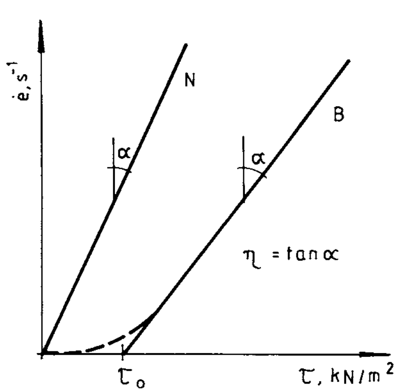


Fig.4.11 Diagram of viscosity, *N* Newton liquid, *B* Bingham liquid

sion in the, packer tube as shown in Fig.4.10, and it amounts to:

$$p_i = p + (h_1 + L/2)\,\gamma_s \tag{4.1}$$

when grouting above the ground water level, and to:

$$p_i = p + h_1\,\gamma_s + (h_2 + L/2)\,(\gamma_s - \gamma_w) \tag{4.2}$$

below the ground water level. With the specific mass of cement $\rho = 3{,}1$ t/m^3 the unit weight of different suspensions are shown in Table 4.6, from the expression

$$\gamma_s = (W + C)/(W/\gamma_w + C/\gamma_c). \tag{4.3}$$

TABLE 4.6

Unit weight of cement suspensions

Ratio C:W by weight	γ_s kN/m^3	$(\gamma_s - \gamma_w)$ kN/m^3
1:3	12,0	2,0
1:2	12,9	2,9
1:1	15,2	5,2
0,8:1	16,1	6,1
0,6:1	17,4	7,4

Since a great precision is not required in the computation the unit weights from Table 4.6. can also be used also for composite grout suspensions.

The resistance to flow of the suspension injected in the fissures is governed by Eq. (3.5) for a Bingham liquid, by means of which the distance ***R*** reached when injecting in a planar fissure ***2a*** wide can be computed. Wittke (1968) has developed the expression for injecting

$$R(\phi) = (r_o + a\,p_i/\tau_o)\;/\;1 + (a/\tau_o)\,(\gamma_s - \gamma_w)\,\sin\alpha\,\cos\phi \tag{4.4}$$

into a planar fissure ***2a*** wide inclined by $\alpha°$ which is cut by a vertical injection hole of diameter $2r_o$. This is the equation of an ellipse with a hole is in its upper focus, as shown in Fig. 4.12.

For a horizontal fissure this gives the radius of the circular space filled with injected grout as

$$R = r_o + a\;p_i/\tau_o \tag{4.5}$$

It is seen that the condition for the penetration of grout into a fissure to a distance $R r_o$ is satisfied even with a very low injection pressure $p_i \rightarrow 0$. The reach of injection with a Newton liquid ($\tau_o = 0$) is infinite.

Wittke (1968) has verified these results on models of planar fissures, and the results shown in Fig. 4.13 are in good agreement with the theory.

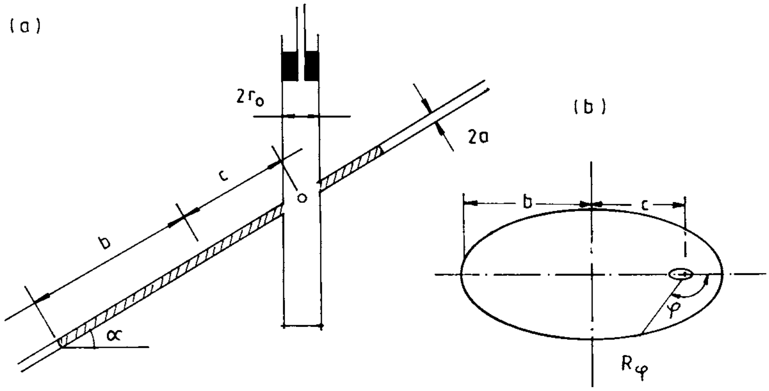


Fig. 4.12 Reach of injected grout in inclined fissure from hole of 2 r_0 ID, (a) section, (b) view in fissure plane (Wittke, 1968)

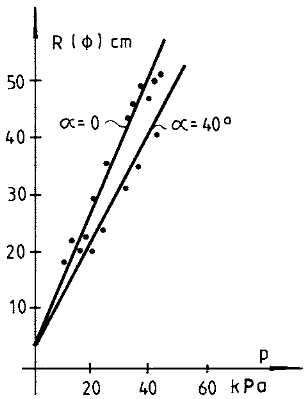


Fig. 4.13 Comparison of injection reach according to Eq.(4.4) and as measured in model (Wittke 1968)

This theoretical approach is only a guide, since conditions in the field are very complex. It shows, however, that two parameters, p_i and τ_0 govern the reach R of injected grout which can be influenced by the pressure and by the threshold value of the grout strength τ_0. The fact that the fissures are not uniformly planar and that the grout suspension contains solid particles is disregarded in the theory, the results are therefore only illustrative. In order to inject a suspension into a fissure wide 2a = 0.2 mm to a reach of R = 5 m, an injection pressure of p_i = 800 kPa is needed, but the solid grains contained in the suspension may block the penetration much closer to the injection hole at much higher pressures.

Lombardi (1985) has obtained similar theoretical results. In Fig. 4.14 the disposition of the fissure is shown, and Fig. 4.15 (a) shows the theoretical results for a set of properties of the suspen-

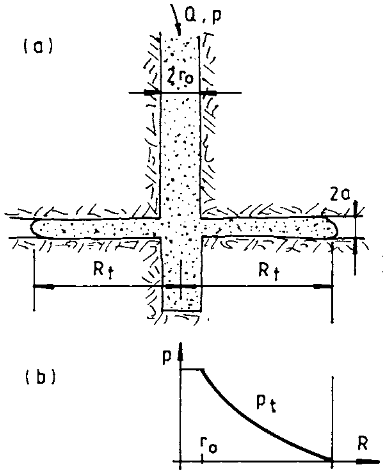


Fig 4.14. Sketch of: (a) injection reach in a horizontal fissure, (b)pressure distribution along the fissure (Lombardi, 1985)

sion and the fissure. A hydraulic computation yields the reach $\boldsymbol{R}$, the pressure increase p_i in the hole and the flow Q in time as shown in Fig. 4.15(a). The pressure vs flow characteristics p_i/Q of the injection pump is shown in Fig. 4.15(b).

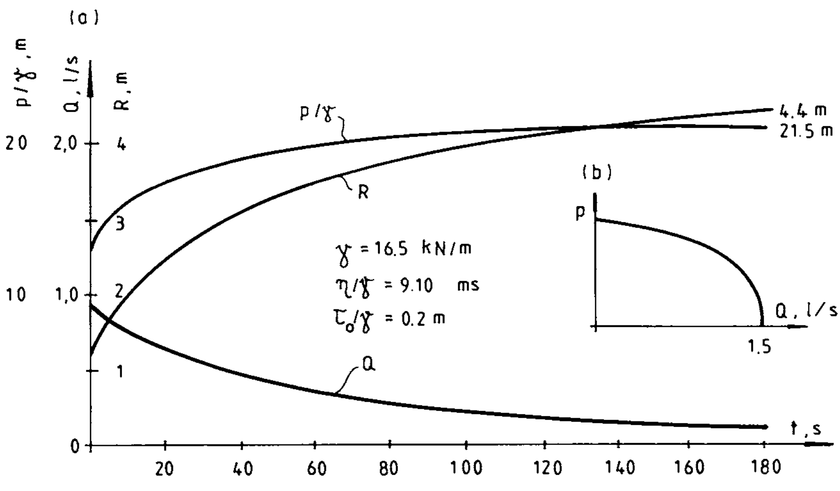


Fig. 4.15 Penetration of grout into 2 mm wide fissure, (a) radius of reach $\boldsymbol{R}$, injection pressure p/γ_s, volume of suspension Q, (b) characteristic Q/p of the injection pump (Lombardi, 1985).

The gradual saturation of the fissure, as well as the maximum reach of the grout are presented in Fig. 4.16. The theoretical maximum reach max $\boldsymbol{R}$ is never attained in practice, because the saturation condition specified a minimum flow and a specified pressure when the injection is stopped. In the example shown in Fig. 4.15 and 4.16 a reach of 4.5 m is attained 200 s after the start of injec-

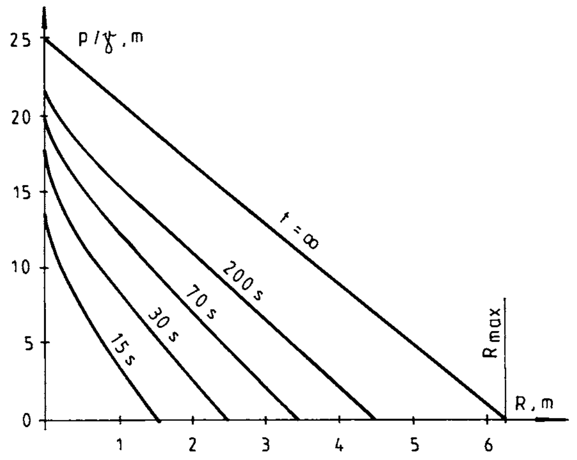


Fig. 4.16 Injection pressure along the fissure in time (Lombardi, 1985)

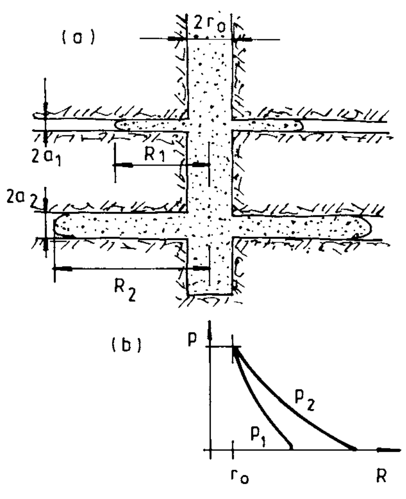


Fig. 4.17. Simultaneous grouting (a) of two fissures with different widths, (b) injection pressure distribution

tion, the pressure is $p_i = 21.5x16.5 = 355$ kPa, and the flow is Q = 0,2 l/s. The maximum reach after infinite injection time would be R=6,13 m at a pressure of $p_i = 25x16.5 = 412$ kPa. The reach which can be achieved is less than the theoretical one, also because water is filtered during the process into finer fissures and the suspension gradually becomes thicker, so that the parameters which enter the computation are not constant in time and space.

The width and the roughness of fissures and joints vary erratically from one location to another. During the injection at the same pressure from one section of the hole, the fissures fill with different velocities farther from the hole in wider fissures, and closer to it in narrower fissures as sketched in Fig. 4.17. The aspect of injected orthogonal fissures from a section of the hole at time *t* after the start of injecting is shown in Fig. 4.18. The different reaches R_1 and R_2 depend on the dip of the fissures and the influence of gravity on the flow and on the pressure distribution along the fissures.

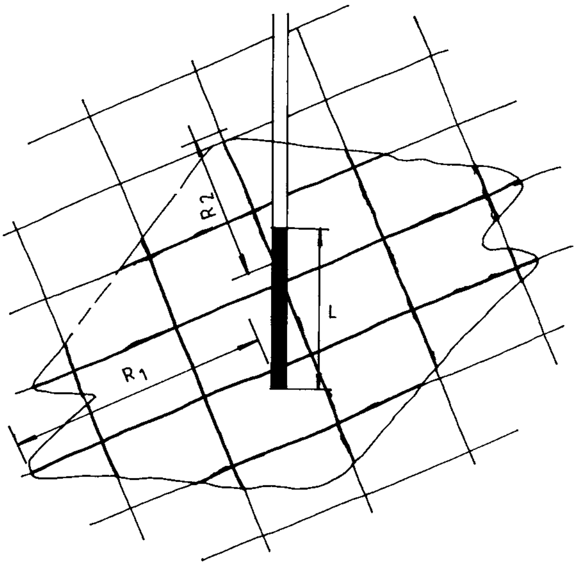


Fig. 4.18 Injection of a system of fissures from a grouted hole section

The relationship may be more complicated in reality, but it is evident that the reach of the injected grout is not uniform in the space. The reach increases with the injection pressure and with the fluidity of the injected suspension.

Some other factors may limit the injection pressure p_i. The effective stresses are reduced, and so is the effective strength of the medium, with increasing pressure. In Mohr's presentation the stress circle is moved towards the origin, and when it becomes tangent to the strength envelope the failure strength (σ_n, τ), which acts on inclined planes, is attained. Shear planes are formed if $\sigma_n > 0$ and tensile cracks if $\sigma_n < 0$ and $\tau = 0$ as shown in Fig. 4.19, which is the condition for hydraulic fracturing of the formation.

The pressure at which hydraulic fracturing occurs can be estimated from the solution of the stress field in a thick wall cylinder of homogeneous elastic material with internal pressure p (Timoshenko, 1957, pp. 58-60).

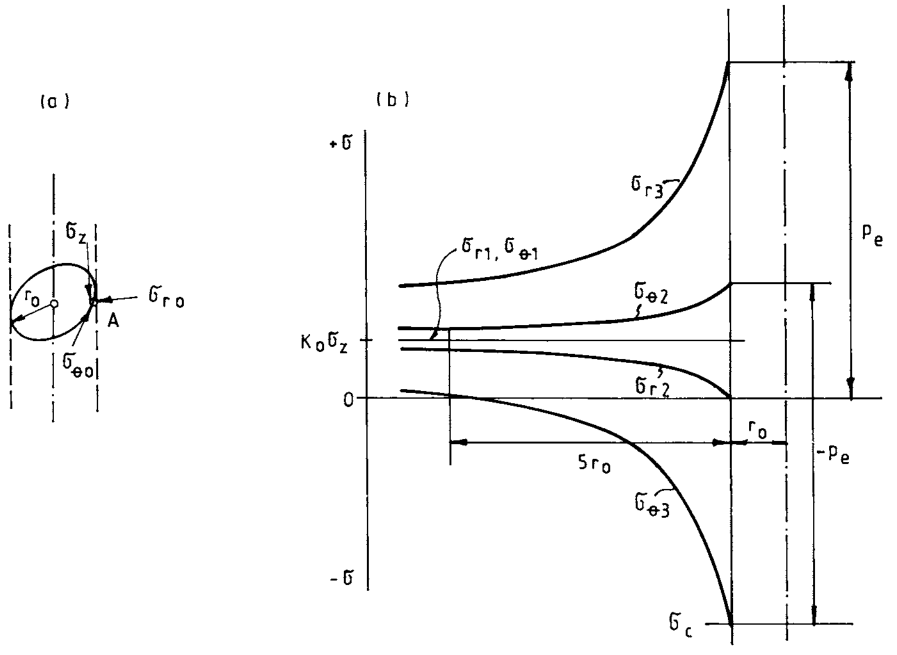


Fig. 4.19 Stresses: (a) at the perimeter of the injection hole, (b) on a radial horizontal plane through the hole

The principal stresses at point *A* in Fig. 4.19, at depth *z* below the ground surface are:

$$\sigma_1 = \sigma_{z1} = \sum_0^z \gamma\, \Delta_z \qquad \sigma_2 = \sigma_\theta\,(r_o)_1 = K_o\,\sigma_{z1} \tag{4.6}$$

$$\sigma_3 = \sigma_r(r_o)_1 = K_o\,\sigma_{z1}$$

The principal stresses after drilling the injection hole become

$$\sigma_1 = \sigma_{z1}\,, \quad \sigma_r\,(r_o)_2 = 0, \qquad \sigma_\theta\,(r_o)_2 = 2\,K_o\,\sigma_z \tag{4.7}$$

The injection pressure p_i changes the stresses at point *A* on the hole perimeter to

$$\sigma_1 = \sigma_{z1}, \quad \sigma_r\,(r_o)_3 = p_i, \quad \sigma_o\,(r_o)_3 = 2\,K_o\,\sigma_{z1} \;\leq\; \sigma_t \tag{4.8}$$

where σ_t the tensile strength of the material.

From Timoshenko's solution the distribution of $\sigma_r(r)$ and $\sigma_\theta(r)$ along the radius of the cylinder, from the stress reduction $\sigma = -K_o\,\sigma_{z1}$ after drilling the hole is:

$$\sigma_r = -K_o\,\sigma_{z1}\; r_o^2/r^2, \qquad \sigma_\theta = +K_o\,\sigma_{z1}\; r_o^2/r^2, \tag{4.9}$$

and from the injection pressure p_i

$$\sigma_r = p_i\, r_o^2/r^2, \qquad \sigma_\theta = -p_i\, r_o^2/r^2. \tag{4.10}$$

When $\sigma_\theta(r_o) > \sigma_t$, tensile cracks will develop on vertical radial planes through the axis of the hole at the injection pressure

$$p_e \geq \sigma_c + 2\,K_o\ \sigma_{z1}. \tag{4.11}$$

At the instant the medium cracks, it is not homogeneous any more, and the crack will continue to propagate along the radius r unless or until the pressure is reduced.

In normally bedded and fissured rock the injection pressure causes some elastic deformations which widen the fissures and increase the permeability, thus facilitating the penetration of grout even before hydraulic fracturing begins.

Generally speaking, the stresses in Eq. (4.6-4.11) increase linearly with depth, thus it is reasonable to increase the injection pressure according to some linear relationship with the depth. Whether or not hydraulic fracturing is desirable for grouting is still a controversial issue in the profession. Hydraulic fracturing caused by high injection pressures increases the groutability of the formation and the reach of the injected grout, because new interconnections in the net of fissures are created. Thus, the efficiency of grouting is increased. The permeability of the formation is reduced to a greater distance from the injection hole, the spacing of holes can be increased and an economy is achieved by reducing drilling costs. This is an advantage over the alternative to achieve the same reduced average permeability with a lower injection pressure and a closer hole spacing. Hydraulic fracturing may be indispensable for efficient grouting of predominantly vertically fissured formations, in which case any bore hole may miss the connection to many of the fissures. Fissures created additionally by hydraulic fracturing open new ways connecting the injection holes with the existing fissures, thus highly improving the groutability of the formation and the efficiency of grouting through vertical, instead of much more expensive inclined holes.

An example of the computation of the hydraulic fracturing pressure is shown in Fig. 4.20. The strength of specimens of a homogeneous bed of marble from tests of Karman (Skempton, 1961) is assumed in the model of the injected rock. The critical injection pressure is computed graphically for a hole section at the depth of 50 and 100 m . The ratio of stresses and the shape of the strength envelope are such that radial vertical fissures result at the boundary of the hole when the injection pressure is increased to the critical value p_e. Only at a greater depth would the stress circle be tangent to the strength envelope above the axis of the abscissa indicating shear failure. Since formations are never homogeneous in nature, this computation is only a qualitative illustration of real conditions.

Another possible approach is to model the injected medium as a granular material with shear strength $\tau_f = c' + \sigma_n \tan \phi'$, as proposed by Morgenstern and Vaughan (1963), which is valid for

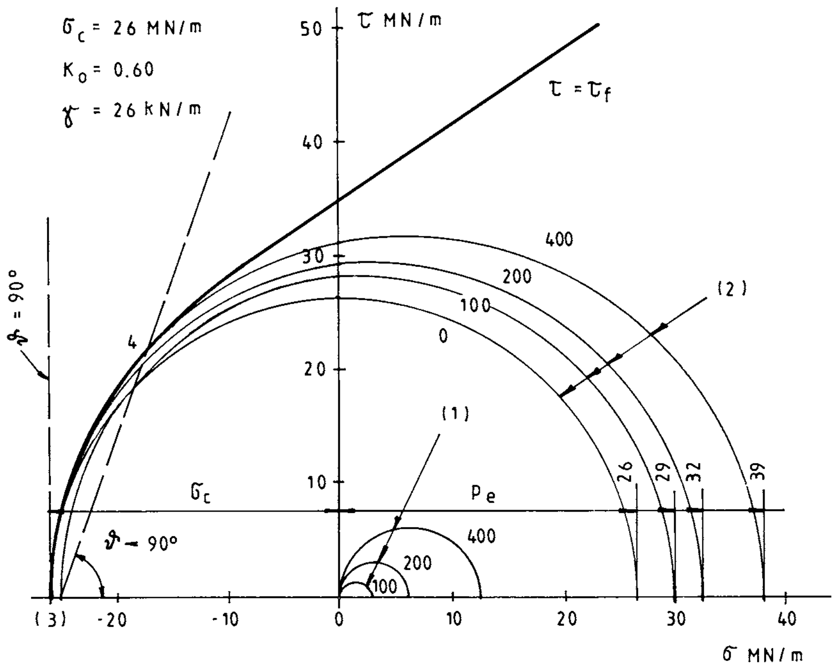


Fig. 4. 20 Computation of the critical pressure p_e (Eq. 4.9) at which cracks develop in marble, $\tau = \tau_f$ strength envelope of a cylindrical sample, 1 stress circle for gravitational stresses in 100-400 m depth, 2 stress circles for critical stress p_e, 3 tensile rock strength σ_c, 4 tangent point of strength envelope and stress circle for 400 m depth

soils, but it can be used also as a rough estimate for a large volume of densely jointed rock. The limit equilibrium state is expressed as:

$$0{,}5\,(\sigma_1' + \sigma_3')\sin\phi' = 0{,}5\,(\sigma_1' - \sigma_3') - c'\cos\phi' \qquad (4.12)$$

The ratio between the principal stresses on horizontal and vertical planes at depth z is $\sigma_1' = \gamma_z - \gamma_w h$, or $\sigma_3' = K_o\,\sigma_1'$ where h is the depth of the ground water level. The injection pressure p_i for hydraulic fracturing on planes orthogonal to the direction of the minor principal stress is

$$p_i = \{[K_o\,(2\sigma_1' + \gamma_w h) + 2\,c'\cos\phi'/\sin\phi'] \,/\, (1 + \sin\phi')\} - \gamma_w h \qquad (4.13)$$

where $K_o = 1 - \sin\phi'$. With the relationship $K_a = (1 - \sin\phi')/(1 + \sin\phi')$, the following equation

$$p_i = K_a\,(2\,\sigma_1' + \gamma_w h) - \gamma_w h + 2\,c\,(K_a/\sin\phi')^{1/2} \qquad (4.14)$$

is obtained, and with $c = 0$ for a purely frictional material

$$p_i = 2\,K_a\,\sigma_1' + (K_a - 1)\,\gamma_w h \qquad (4.15)$$

The circle of effective stresses (1) in Fig. 4.21 shifts to (2) by the gradually increasing injection pressure p_i until at the critical pressure p_c, the failure circle (3) touches the strength envelope at the point A. The failure planes are inclined by ϑ_f and the normal pressure at the point A on the failure

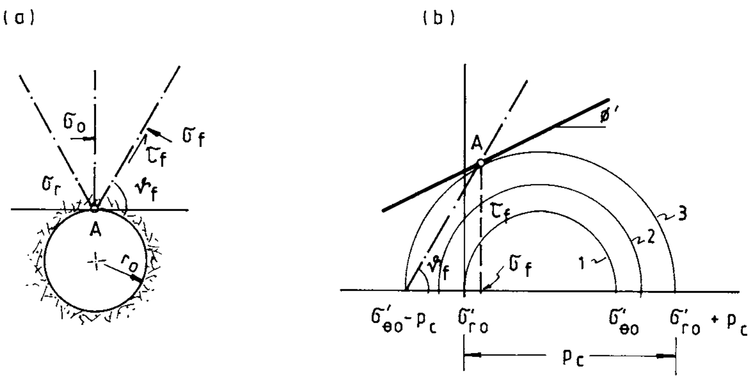


Fig. 4.21 Critical stress in a frictional material at point A on the hole wall, (a) inclination of critical stress planes, (b) stress circles, 1 stresses at A before drilling, 2 before grouting, 3 for critical injection pressure p_c, ϑ_f direction of critical plane at A

plane $\sigma_f < p_c$ is positive, so that no open cracks can develop. The stresses on radial planes through the hole axis are also positive, therefore no open fissures can develop. When the injection pressure exceeds p_c large plastic deformations of the material surrounding the hole will occur. The injection pressure should not exceed the overburden stress because lifting on horizontal planes could occur. From this condition the pressure rule generally applied in the Anglo-Saxon countries, of 1 psi per foot of depth, which is equal to 22 kN/m^2 per meter of depth, was derived. The penetration of grout could be facilitated at higher pressures in stiff materials in which the shear planes might be undulated.

Both models give only rough estimates of the injection pressure at which hydraulic fracturing might occur, since the formations are nonhomogeneous, erratically fissured and jointed and the fissures may be either open or closed. Therefore, some empiric rules are used to determine the relationships between the injection pressure and the depth, and between the injection pressure and the vertical effective stress. In Fig. 4.22 the lines (a) and (b) show the ratio of the injection pressure to the depth, as given by the US Corps of Engineers (Burwell, 1958), and the line (c) is the previously mentioned Anglo-Saxon pressure rule. The lines 1, 2, 4 present the corresponding ratio between the injection pressure and the weight of overburden at depth d, full lines for grouted sections above the ground water table and dashed ones below the ground water table. It is interesting to note that the range of recommended injection pressures are high, and that the applied injection pressure for successfully achieved grout curtains, was between medium and high, or above, the high recommended pressure range. The highest injection pressure between 50 and 100 m of depth was seldom more than 60 bar. It should be noted that the grout curtain of the Dokan Dam had to be regrouted 10 years after its completion, because it began leaking at several sections, and it was grouted with a pressure ratio lower than the lowest recommended ratio represented by the line (c).

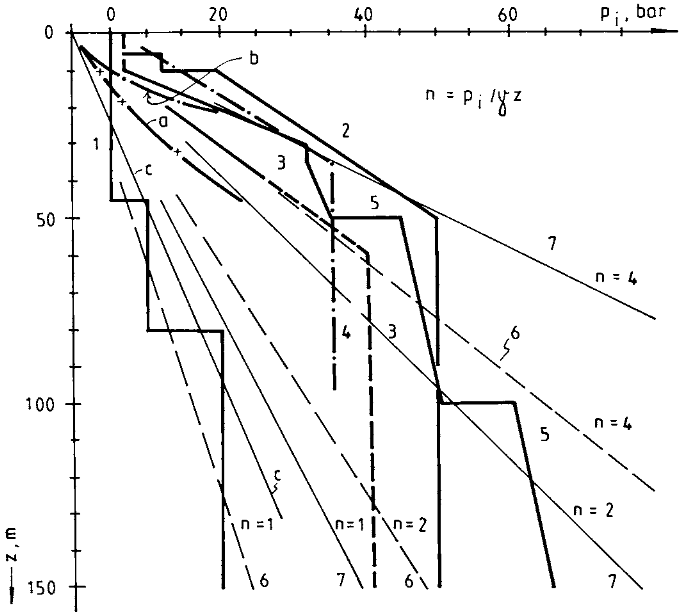


Fig. 4.22 Injection pressures of some grout curtains, 1 Dokan Dam, dolomite and limestone beds, 2 Peruća Dam middle row, 3 same outer rows, 4 Buško Blato, 5 Meifedoun grouting test plot, 6 overburden weight above, and 7 below the groundwater table both for n=1, 6, 7 for n=2 double overburden weight, n=4 four times the overburden weight, a US Corps of Eng. fractured rock, b same massive rock, c 1 foot/1 psi rule

The injection pressure which would cause hydraulic fracturing is higher than the highest recommended pressure. Bedded and fractured formations open up even under much lower applied injection pressures than the critical one. The best results are achieved when the pressure is definitely specified after careful evaluation of injection results on test plots and of current production grouting.

4.4 The injection process

The process of injecting a section of a hole should comply with the frame of the specifications in terms of the choice of the composition of the injected suspension, the pumping rate and the pressure, so that the ***saturation criteria*** are satisfied in a given time interval. The development of pressure and of the rate of flow furnish informations on possible unusual conditions.

Grouting starts with the thinnest specified suspension, which will usually be between 3:1 and 2:1 as stated previously. When the flow is modest and the pressure increases gradually, injection is continued to saturation with the starting suspension density. In the case when the pumping flow rate is high and the pressure remains low, gradually denser suspensions are injected until the pressure begins to increase and the flow to decrease, decisions are made in time intervals of about 15 min based on the development of pressure and flow. Gradual flow decrease at increasing injection pressure is the normal development when saturation is approached. A sudden flow reduction can be the consequence of untimely injection of too thick suspensions. When the flow increases sharply during the injection, and the pressure decreases simultaneously, it indicates that grout has broken through some filled fissures or that hydraulic fracturing of the formation has occurred. In such events the first reaction is to thicken the suspension and to continue injecting at lower pressure until a gradual increase of pressure and a reduced flow are established again.

The normal development of pressure $p(t)$, mixing ratio of suspension $n(t)$, flow rate of suspension $q(t)$ and the total injected mass $Q(t)$ during the injection is shown in Fig. 4.23 from the start of the injection to complete saturation, as specified. The thinnest mix 1:3 was used during the first 15 min and a total of 110 kg of compound was injected. A 1:2 suspension followed with increased pressure,in order to increase the reach. During the next 2 hours 530 kg of compound was injected at low pressure

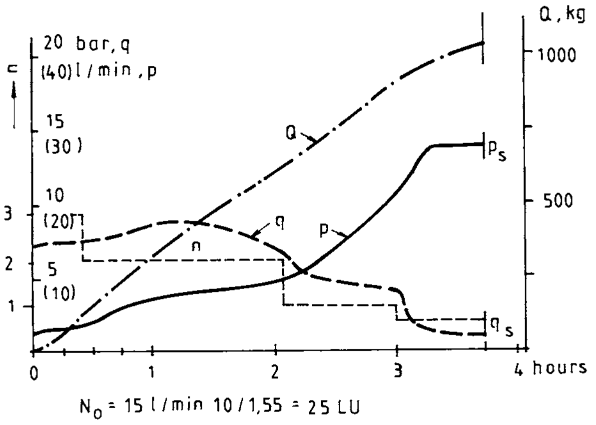


Fig. 4.23 Injection diagram of a grouted stage, p pressure, q flow rate l/min, Q injected compound, kg, 1:n suspension water ratio, N_0 initial permeability

when the suspension was thickened to 1:1 during the next hour, and 280 kg of compound was injected at slowly increasing pressure to the specified saturation pressure, then the suspension was thickened to 1:0,7 and injected until the specified saturation flow rate was achieved. The total consumption of compound in the 5 m long stage was 1,035 kg, corresponding to a specific take of 207 kg/m. The permeability of the formation before and after the injection was computed from the respective flow rates $q(t)$ accounting for the viscosity of the used suspensions.

Fig. 4.24 shows the case of an abnormal course of injection of a section during which a substantial leak occurred. Injection was stopped as stipulated in the specifications when the specified maximum quantity of compound was injected without reaching the saturation criterion. Injection was resumed after a specified time, as shown, and it continued normally to saturation. In some cases it is necessary to repeat the injection until satisfactory saturation is achieved. In obstinate cases the injection of thick and thixotropic suspensions is necessary, or other means to block excessive grout

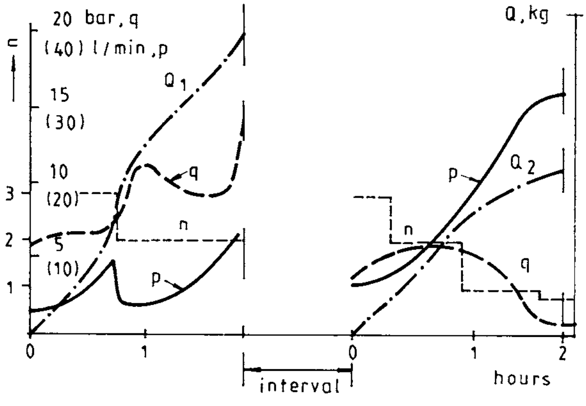


Fig. 4.24 Injection diagram with grout outbreak from stage

consumption, and injections of thick mortars etc. should be applied. Another approach is to stop grouting, to start additional investigations and to apply other means than grouting, as for example in the case of large caverns in the axis of the grout curtain of the Sklope Dam (see Section 6.7).

Because of the erratic nature of fissured rock and the wide range of its initial permeability, a great practical experience of the grouting engineer is beneficial. He should be open to optional approaches intended to best meet the possibilities offered by the variation of injection pressure and pumping rate, and adapting the water ratio of injected compounds, in order to achieve the specified result in the most economic way. Too quick saturation of the grouted section results in economy of the compound, but a too small volume of rock is treated. By delaying saturation a too large volume of soil is injected and compound is wasted. A sound feeling is essential for deciding on the correct approach between such extreme options, which leads to an optimal ratio between the cost of drilling, the grouting and expenses for injected compounds,in order to satisfy the purpose of the operation. These facts should be kept in mind when the technical specifications and the criteria for injection activities are prepared. The essential parameters must be unambiguously fixed, but dealing with details, which are to be decided upon when further data from drilling and grouting performance are available, should be flexible.

The final decision on the hole spacing, the number of split spacing sets of injected holes, the choice of the grouting compound, the injection pressure and the procedure to saturation should be set after detailed analyses of the results of grouting test plots carried out in dominant locations typical of geologic, lithologic or tectonic characteristics.

Although grouting is more an art than a technical discipline, it may be interesting to quote some general rules proposed by Houlsby (1982):

- compound to water ratio of injection grouts can be limited to 3:1, 2:1, 1:1 and thicker, intermediate ratios can be omitted;
- in most cases it is sufficient to start injection with a 2:1 mix, except in cases when fissures are narrower than 0,7 mm, then start with 3:1 mix, when fissures are wide (1 to 2,55 mm) start with 1:1 mix or thicker;
- when the initial flow ratio into the hole is low injection should proceed with starting mix to saturation; when the flow ratio is high and only low pressure develops the next mix should be already thicker;
- the injection process of a section should be checked every 15 min, from the development of the process decisions on pressure and mixing ratio should be taken for the next 15 min time interval of grouting;
- gradual reduction of grout flow rate in successive 15 min intervals signifies an appropriate grouting strategy. On the contrary constant flow rate during three or more 15 min intervals requires application of thicker mix;
- a sudden decrease of the grout flow rate signifies blockage of the grouting section from a too thick mix. A sudden increase of the flow rate is a sign of hydraulic fracturing due to excessive injection pressure, break through of fissure fillings or surface leaks. Proper reaction is to immediately thicken the mix and reduce the pressure, unless hydraulic fracturing is intentionally caused in order to increase the injection reach.

The correlation between the specific grout consumption ***Q*** (kg/m) and the permeability *N* (LU) measured in the same section of the injected hole is still the subject of professional discussions. Systematic studies have not been published so far, and the analysis of injection results shows a wide scatter of the ***Q/N*** values from actual grouting results. This is due to the fact that the permeability *N* is usually measured at 10 bar pressure, while the injection pressure may be lower or significantly higher than this value. Also, the same value of permeability *N* may be the consequence of one large or many narrow fissures in which the grout may even not penetrate. In this respect the results obtained by Rakić (1986) from the grout curtain of the 170 m high ***Karakaya Dam*** on the Euphrates may be of some interest. The foundation consists of granite and hornblende rock, generally of low permeability, but in some hole sections more than 50 LU were measured. The permeability was measured in 1101 sections of injected holes in which the specific grout consumption was also determined. The results presented in Fig. 4.25, systemized for intervals of consumption and of permeability, suggest that some statistical correlation exists between the two entities. The lower curve

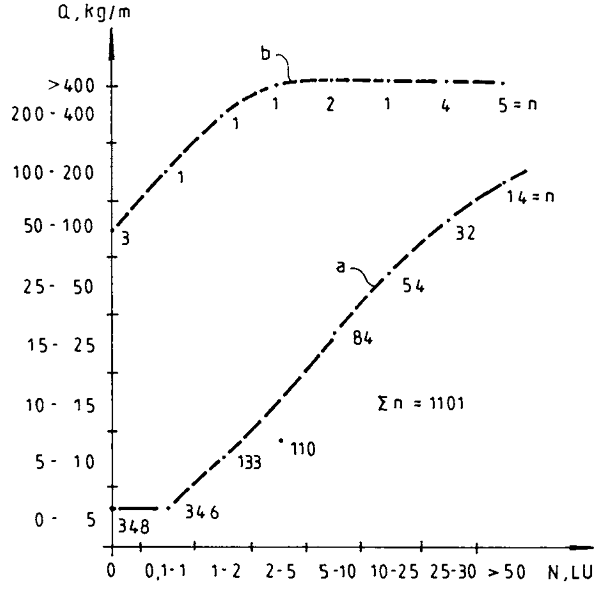


Fig. 4.25 Relationship between permeability and grout take (Rakić, 1986), (a) average values (b) maximum values, n number of measurements

presents the average values, the upper line the maximum values of specific consumption for any of the permeability intervals. The number of measurements for any interval is also shown on the curves. A more reliable correlation would probably result if the permeability of any injected section were computed from the flow rate q_o at starting injection pressure p_o and this is compared to the specific consumption in the same section. Selimovi (1977) has published the results of such an analysis of the grout curtain of the ***Kazaginac Dam*** which shows that such a correlation exists, as shown in Fig. 4.26.

4.5 Grouting alluvial deposits

The flow paths in granular alluvial deposits represent an interlaced system of irregular channels of various dimensions, which is quite different form the systems of joints and fissures which are injected in rock. The groutability of granular alluvial material is therefore much worse than the groutability of rock having the same permeability, and different methods of injecting and different kinds of grouts are needed in order to achieve satisfactory results.

Injection holes in alluvium have to be cased during the drilling, otherwise they would collapse. Drilling may be rotary or percussive, preferably with rigs that simultaneously case the hole, and the flushing fluid is solely water.

Injection of very permeable formations can be done in parallel with the drilling. Every 30-50 cm the drill rods are withdrawn by this amount above the bottom of the hole and the section is injected

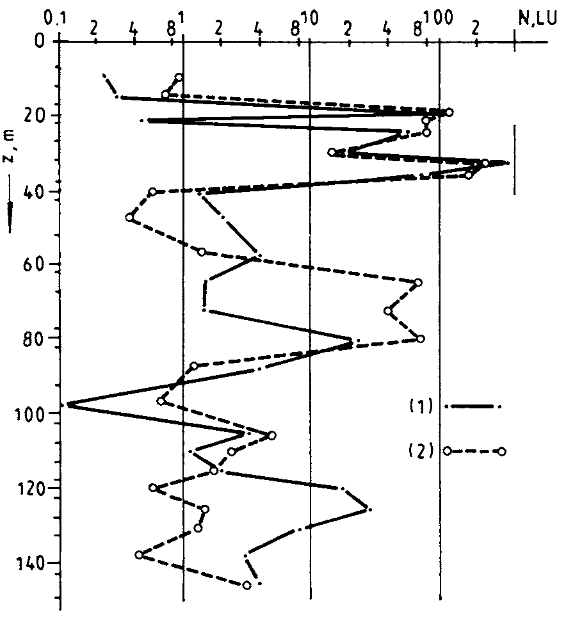


Fig. 4.26 Relationship between permeability measured before grouting (1) and the one computed from initial grout flow (2)

until the desired quantity of grout is used. The procedure is repeated until the required depth is treated. The effect of injection through the drill rods is very irregular, the reach and the degree of saturation vary within wide limits depending on the ground composition and the permeability. It is therefore used only for temporary or auxiliary structures.

When accurate grouting results are required, the method of ***sleeve grouting*** (***tube-à-manchette***) is adopted, the investigation of Mr. Ichy (Glossop et al., 1962). A hole about 100 mm in diameter is drilled to the bottom of the curtain and cased. Then a narrower plastic or steel tube usually of 38 mm ID with rings of holes along its length at 30-50 cm intervals, which are covered by a soft rubber sleeve (manchette), is introduced. The space between the ground and the inner tube is injected from the bottom up with a soft grout rich in clay - the sleeve grout, while the casing is being simultaneously pulled out. When the sleeve grout has developed sufficient strength (up to some 4 MN/m) the injection is started. A double packer is introduced and fixed so that a ring of holes is injected at a time. The injection pressure is increased until the sleeve grout is cracked and the suspension penetrates through the cracks into the alluvium. When the injection is stopped, the rubber sleeve closes the ring of holes and it prevents the injected grout from entering the grouting tube. The advantage of this method is that every ring of holes can be reinjected as needed to achieve a uniform permeation or penetration of the surrounding ground.

The details of the method are illustrated in Fig. 4.27. The injection holes are arranged in groups of primary, secondary and tertiary holes,as shown in Fig. 4.28 with a basic spacing ***a***.

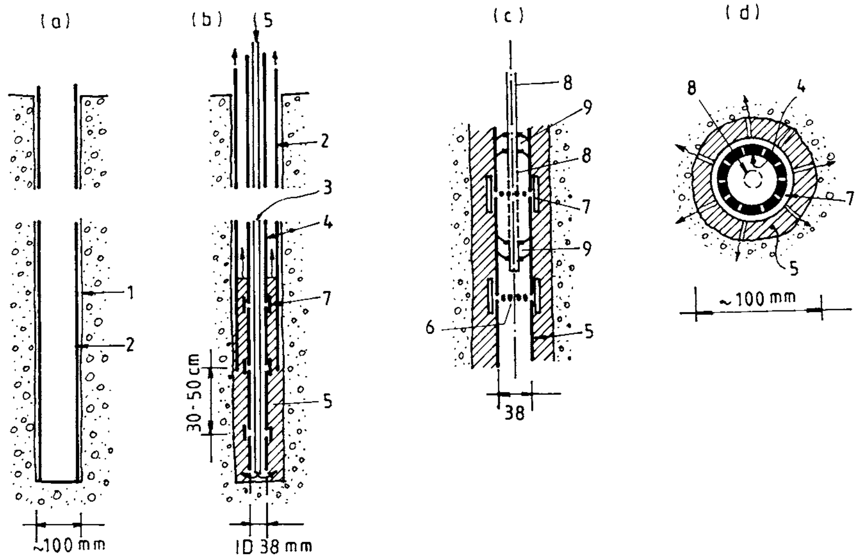


Fig. 4.27 Construction phases of sleeve grouting hole; (a) drilling and casing, (b) filling of, 5 plastic mortar around 4, sleeve tube with 7, sleeves, simultaneously casing 2 is withdrawn, (c) details of sleeve grout tube with injection pipe and, 9 double packer, 6 ring of holes and, 7 rubber sleeve, (d) cross section of sleeve injection hole with cracked grout sleeve during injection

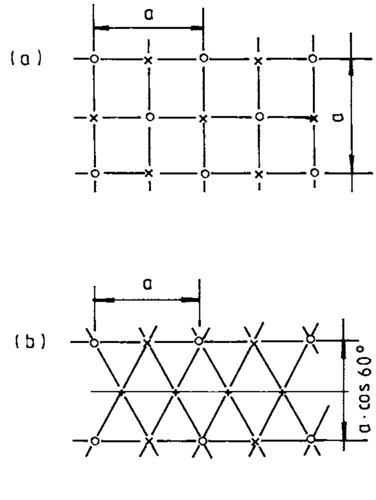


Fig. 4.28 Dispositions (a) and (b) of alluvial grouting holes in several lines with two to three grouting sets, o primary, x secondary, + tertiary holes, *a* basic hole spacing

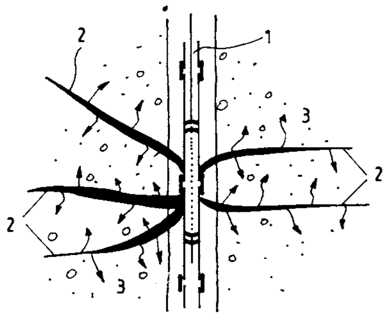


Fig. 4.29 Grouting of sand by means of hydraulic fracturing and with permeation: 1 sleeve grouting tube, 2 hydraulic fractures filled with grout lenses, 3 penetration of voids by grout

The purpose of alluvial grouting is either to reduce the permeability, which is used in foundation and in hydrotechnical engineering, or to improve the strength and the deformability of the ground, used in foundation engineering.

If the purpose of grouting is to reduce the permeability, the desired effect can be achieved by a combination of permeation and penetration grouting. The lenses of grout created by intentional hydraulic fracturing will compress the soil and increase the area of penetration of the injected compound into the ground, as shown in Fig. 4.29. Caron (1982) stated that at the pumping rate of 500 l/s of injected compound from one sleeve, two to three disks of 50-60 cm in diameter are formed around the hole.

The method of fracturing can be applied only at a greater depth where surface heave is prevented by the passive resistance of the soil, which would limit the effect of compressing the ground. The strength of the injected grout need not be high, it should only be capable to prevent erosion by the seepage of water. The injected grout should have high strength if the purpose of injecting is improvement of strength and deformability of the ground below foundations of heavy structures. Again, permeation grouting and displacement grouting with hydraulic fracturing may be applied.

The main parameters for the proper selection of the best type of grout is discussed in Chapter 3. Caron (1982) has suggested the data given in table 4.7.

TABLE 4.7

Criteria for the selection of injection compounds for alluvial grouting

Parameter	Cement Bentonite	Injection compound: Colloidal solution	Injection compound: Chemical solutions,resins
Effective grain diameter D_{10} mm	0.5	0.02-0.05	0.02
Permeability coefficient cm/s	10^{-3}	10^{-1}-10^{-3}	10^{-3}
Specific grain surface cm/cm^2	10	100-1,000	1,000

Permeation injection is not possible in cohesive soils which can be penetrated only by means of hydraulic fracturing, or through fissures which exist in some highly overconsolidated clay beds, when the permeability coefficient is of the order of $k = 5x10^{-5}$ cm/s or more.

Cement bentonite-suspensions can be injected only into sand and coarser strata, fine cement is preferred to facilitate permeation of the void spaces, the coarsest cement particles being less than 1/3 of the average size of void spaces. The use of high turbulence mixers for the preparation of injected suspensions is very convenient.

The permeation of the ground by the use of colloidal and chemical solutions depends on the viscosity of the compound and on the pumping flow rate. It was shown that fine sands with permeability of $k = 10^{-4}$ cm/s can be permeated with colloidal silicate solutions if the pumping rate is less than 100 l/h. However the compound is diluted by a slow pumping rate, the gel time gets longer and the strength of the injected compound becomes very low, so that the lower limit of the pumping rate should be determined by appropriate in situ tests.

The permeability of the ground is reduced when the flow channels through the void spaces are filled with grout, but some effect may be gained by compressing the ground intentionally by hydraulic fracturing (claquages) when lenses of injected consolidated grout are formed.

Raffle et al. (1961) have computed the rate of flow q injected at the hydraulic head H m as:

$$q = 3.77\ k\ H\ (2\ L\ r)^{1/2} / \eta \qquad (4.16)$$

and the time necessary to permeate a cylinder of radius R around the grouted stage:

$$t = \eta\ n\ R^3/90\ k\ H\ (s\ L\ r)^{1/2}\ . \qquad (4.17)$$

When injection is done in two sets of holes the disposition of holes is the one shown in Fig. 4.28(a), and when three sets of holes should be injected the disposition is shown in Fig. 4.28(b). The sleeves of all primary holes are injected first from top to bottom, in the first run cement-bentonite suspensions are injected, and the mixing ratio with water should correspond to the properties of the ground. A quantity of compound set in advance is injected in every sleeve. Sleeves in which the specified pressure is not attained are regrouted in the second run. In the secondary holes sleeves opposite to those which took a large volume of grout are injected first with cement-bentonite suspensions, then all sleeves are injected with chemical solutions. The same procedure is followed if a tertiary group of holes is injected. Check holes are drilled at locations which require a grout consumption above average and the permeability is measured by injecting water at a pressure that does not exceed the injection pressure. When the check is not satisfactory the sleeves of the surrounding holes are reinjected and the check is repeated until the specified permeability is attained.

For preliminary planning of the drilling and the grout consumption it can be assumed that 0,6-1 of drilling per 1,0 m^3 of injected ground is needed, and that 60-80% of the void volume will be filled by the injected compound. The unit weight of the injected compound is approximately 1,1 t/m^3. Flexible technical specifications for grouting alluvial ground should be set in the design documents of alluvial grouting so that the results obtained on grouting test plots can be adequately implemented at the site.

5. DRILLING AND INJECTION PLANT AND EQUIPMENT

5.1 Drilling machinery and accessories

Drilling has a large and important share in the total cost of grouting works. The total length of drilled holes on a single project may exceed 100,000 m, the depth of single holes may be more than 100 m, even depths to 200 m have been exceeded. Drilling is a highly specialized discipline, and specialists should be engaged for planning the equipment and carrying out exploratory and production drilling for grouting projects. Only the essential information is given here, which is needed by nonspecialists to understand the problems and basic procedures of drilling for grouting projects.

Only rotary drilling rigs are suitable for the exploration drilling with coring equipment so that best undisturbed samples of the formation can be extracted. Rotary rigs with non coring bits or percussion rotary rigs are used for production drilling which reduces the cost of drilling to 1/3 - 1/4 of the cost of rotary drilling, with a substantially greater speed, as shown in Section 4.1. But, in order to get more information on the properties of the formations to be grouted, some of the production holes may be specified to be drilled with a rotary rig, cored and water pressure tested as exploration holes; some times every third or fourth of the primary holes are drilled as exploration holes when the formation is rather nonhomogeneous, in other instances, when the formations are very homogeneous there may be no need for additional exploration and all injection holes may be drilled with percussion rigs.

For efficient grouting the holes must be flushed properly with clean water during the drilling and additionally before grouting is started, so that the fissures at the hole walls remain clean and open. The best method of drilling to achieve this should be determined by in situ tests. Observations of the sediment carried out of the hole by the flushing water will provide clues for the selection of the best drilling and flushing method. When small chips and fine sand appear in the sediment of flushing water, there is a better chance that the fissures at the hole walls will be clean than when sticky or even cohesive sediment is present. Direct observations of the bore hole walls with a bore hole periscope or a closed circuit television camera gives the best grounds for reliable comparisons. Appraisal of the cleanliness of the fissures at the bore hole walls from the quantity of injected grout is not reliable, because only a statistical elaboration of many tests would yield significant results, which is impractical. It is widely considered that rotary drilling produces cleaner fissures, but in many cases it was found that percussion drilling gave better results.

Light weight rigs are preferred for production drilling on grouting projects because a lot of activities must be carried out on steep slopes and on difficult terrains. On more accessible sites the rigs may be truck or crawler mounted for mobility, or on a platform moved on tracks as shown in Fig.5.1. When working in galleries the rigs can be mounted on sleds and moved along with their own winch.

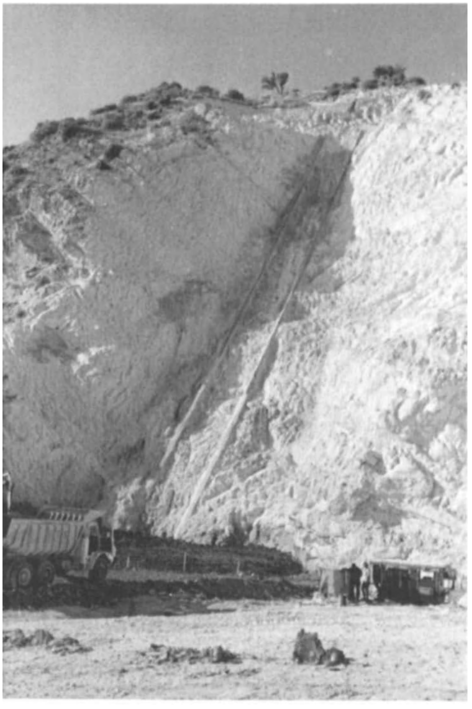


Fig. 5.1 Rail track for the drilling and grouting platform of the steep slope (Evretou Dam, Cyprus)

Modern rotary drilling rigs have an electro-hydraulic drive (Fig. 5.2) and hoists for manipulating the drill rods and casing tubes. The rotary head is moved and driven hydraulically so that the rotational speed and the thrust on the bit can be regulated accurately. The drilling bit is fixed at the tip of the core barrel which is about 3 m long, and the length the progress of drilling in every manoeuvre is limited to this value.

Fig. 5.2 Series of rotary rigs for the drilling of grout curtain through granite masonry at Aswan Dam (Egypt) and Diamec 250 rotary drill

The string of drilling rods is pulled out when the core barrel is full and the core samples are stored in crates for inspection. There is a multitude of types of drilling bits from which the best suited for drilling at a specific site can be chosen (Fig. 5.3). The main types are:

- Thin or thick walled coring carbide tipped bits.
- Thin or thick walled diamond bits.

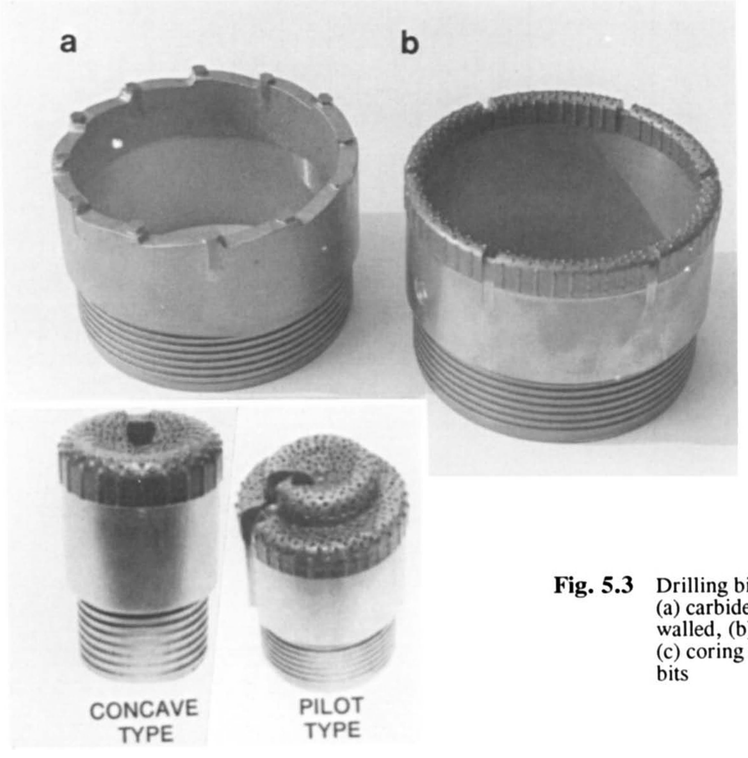


Fig. 5.3 Drilling bits for rotary drilling; (a) carbide inlaid coring bits, thin walled, (b) same thick walled, (c) coring and noncoring diamond bits

Noncoring bits are:

- carbide tipped or with large carbide inserts as fishtail chopping or dragbits with three to four wings;
- diamond type bits of the concave type, pilot type and taper,
- Rollbits for larger diameter holes.

The diameter of drilling bits is standardized in Europe and in the USA, as shown in Table 5.1. Drilling is started with the smallest possible bit diameter for the final depth of the hole. The smallest diameter used for grouting is 36 mm, and the starting diameter of 66 mm is sufficient for about 100 m deep holes.

Table 5.1

Diameters of standard drilling bits

Diameter in mm			Casing		
nominal	of hole	of core	notation	dimensions mm D_a/D_i	weight kg/m
European standard					
145	146			143/134	15.4
129	131			128/119	13.8
114	116			113/104	12.4
99	101	84		98/89	10.5
85	86	72		84/77	7.2
75	76	62		74/67	6.3
65	66	52		64/57	5.2
55	56	42		54/47	4.4
45	46	32		44/47	3.5
35	36	22		-	-
USA standard					
3"	76.20 mm		Nx	74/67	6.3
3/8"	60.33 mm		Bx		
2"	50.80 mm		Ax		
1 1/2"	38.10 mm		Ex		
1 3/8"	34.93 mm		LM		

The best type of bit depends on the hardness and fissuring of the rock. Diamond bits are very expensive but, a greater speed of drilling is attained and diamonds are very durable if the right rotational speed and pressure are maintained. The smallest diameter of rollbits is 60 mm. They are seldom used for injection drilling. Drilling with full bits may be more expensive but a greater output is achieved, because a great depth of drilling can be achieved without manoeuvres for emptying the core barrel, resulting in a greater drilling speed. The string of drill rods is usually pulled out and lowered at speeds of some 120 m/h.

Diamond bits are recommended for drilling of hard rock and concrete, bits with carbide inserts for medium hard rock. Rollbits are used in all kinds of rock and in the alluvium. Non coring bits reinforced with carbides and fish-tail bits are used in soft rock and soil. The best choice can be made from in situ drilling tests.

A sufficient supply of flushing water or air which cools the bit and carries out the drilling chips, is very important for good and economical drilling, such as the appropriate selection of the rotational speed and the pressure on the bit. If all parameters are not well adapted to the character and hardness of the rock the bit may get stuck, resulting in a loss of equipment or excessive wear of the bits.

Percussion drills are driven by an air or hydraulic motor, which is mounted on top of the drill rods and it is moved on a guide. A mechanism rotates the bit step by step between the strokes. Holes in the range of 31 to 63 mm in diameter, and up to 280 m deep were drilled. The holes can be flushed by air when dry rock is drilled, but only water flushing should be practiced for injection purposes . Percussion drilling is faster and cheaper than the rotary drilling. The drilling efficiency on the Meifedoun grouting test plot is shown in Fig. 4.1 . During 31 working hours the hole was drilled 60 m deep by rotary drilling and 231 m deep by percussion drilling. The average drilling speed was 2.3 m/h with rotary and 7.7 m/h with percussion drilling, while the speed of percussion drilling to the depth of 60 m was 14.5 m/h.

Percussion drills with simultaneous casing are very convenient for drilling alluvial ground.

Clean sluicing of the injected section of the hole prior to injecting is very important. For this purpose a pipe with a sluicing head, which directs the water jet perpendicular to the hole walls, is lowered. When grouting from top to bottom, the injected section may be sluiced out after cement has started setting with a sluicing head shown in Fig. 5.4 at the bottom of a 12.5 m ID pipe, which is rotated and struck to the bottom of the grouted hole. The crushed material is carried to the surface by the stream of the upsurging water.

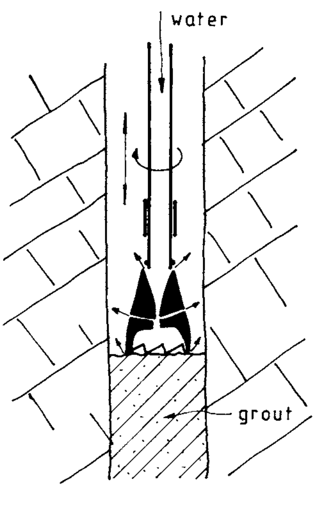


Fig. 5.4 Washout chopping bit for cleaning the grouted section of hole

Great care should be taken in production drilling to accurately keep the specified direction of the hole axis (vertical or inclined) as discussed in Section 4.1. In order to achieve this, the following main points should be observed:

- the drilling rig should be firmly fixed on the platform;
- only straight drilling rods should be used, and their diameter should possibly be close to the hole diameter in order to prevent deformations of the string of rods;
- the rotational speed and the pressure on the bit must be adjusted to meet the character and quality of the rock;
- after every repeated setup the rig must be precisely centered and checked for drilling direction.

Only qualified organizations should be engaged for drilling with highly experienced specialized personnel and modern equipment.

5.2 Batching and mixing equipment

For successful grouting it is essential that the individual components of the grout should be weight batched. The traditional batching by sacks of cement and volume of water should never be used even on small jobs. Modern weight batching plants are assembled on site from standard basic units for storing, proportioning of the components of mixes, and feeding into the mixing units.

Water is pumped from a reservoir to the mixers through controlling water meters, as well as the ***liquid additives*** which are stored in appropriate dispensers.

Cement and bentonite are delivered to the site preferably in bulk and stored in silos from which the needed batches are weighed through automatically controlled scales and fed to the mixers with belt or screw conveyors.

Clay may be delivered and handled dry and pulverized, and it may be stored and batched in the same way as cement. It can also be prepared at the site from some local borrow area, batched from a premixed basic dense suspension and fed by pipe through an automatic meter to the mixer. A special mixer for that purpose was developed for grouting the curtain of the Kazaginac Dam. A mud pump with 15 kW electric motor circulates the slurry through a 800 lit capacity vessel,as shown in Fig. 5.5, until it is completely homogenized. Then it is pumped into an agitating reservoir from which it is fed into the mixers.

Sand, which always contains some moisture, is stored in hoppers from which the needed portion is automatically weighed and fed through belt conveyors or screw drives to the mixers.

Mixers

The main types of mixers used for grout suspension preparation are paddle and colloidal mixers. Usually the mixed grout is transferred to an agitator tank from which the grout pump is fed. Two stage mixers are used in some cases. They transfer the mixed batch to the second stage from which the injector is fed.

A two-stage paddle mixer is shown in Fig. 5.6(a), the paddles rotate at about 150 rev/min, the volume of the drum is about 200 l, and the mixing time is 5-10 min

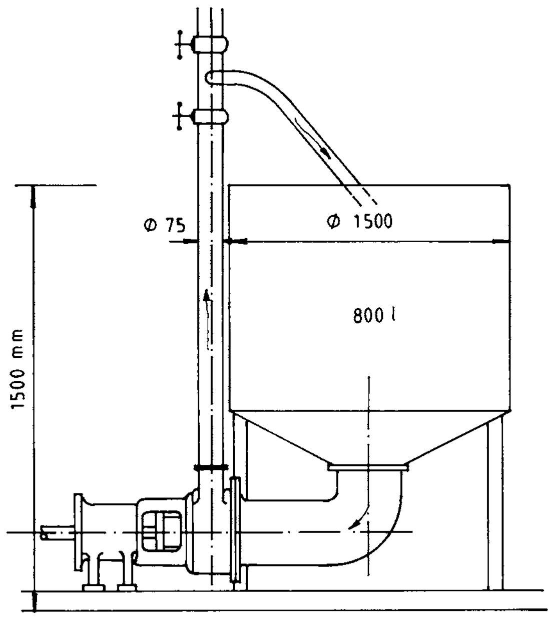


Fig. 5.5 Mixer for the preparation of a basic clay and bentonite suspension with a centrifugal sump pump

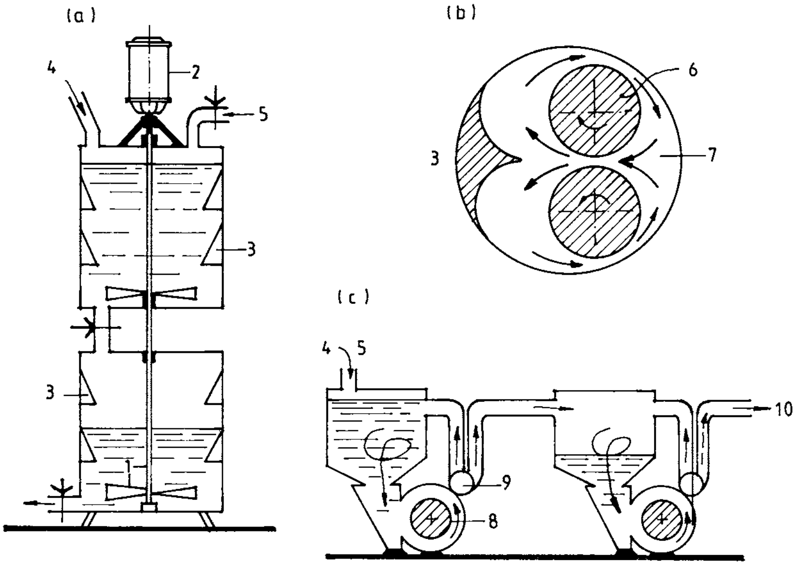


Fig. 5.6 Grout suspension mixers; (a) two stage paddle mixer, (b) Bachy type colloidal mixer, (c) two stage colloidal mixer with centrifugal pump, 1 axis with paddles, 2 driving unit (motor), 3 deflectors, 4 feed of pulverized compounds, 5 feed of liquid compounds, 6 high speed rotating cylinders, 7 narrow slot , 8 centrifugal pumps, 9 valves directing suspension, 10 leading to injection pump

There are two types of colloidal mixers (called also high turbulence mixers). In the Bachy colloidal mixer shown in Fig. 5.6(b) two smooth vertical cylinders revolve at a high speed in opposite directions in a cylindrical vessel, forcing the grout into a narrow slot where strong shear stresses break the suspended particles into small units. The grout emerging from the slot is diverted back by a deflector and it is thus recirculated back to the slot. The mixing time is about 3 min.

The second type of mixer consists of a drum with a conical bottom where a centrifugal rotor is placed in a horizontal or vertical plane,as shown in Fig. 5.6(c). The mixing rotor rotates at 2.000-3.000 rev/min and it forces the mix through a control valve either back into the drum, to an agitator, or to the injection pump. A vortex is formed in the drum which acts as a centrifugal separator forcing the thicker part of the mix back into the rotor until a uniformly homogenized mix is attained. The mixing time is about 15 s.

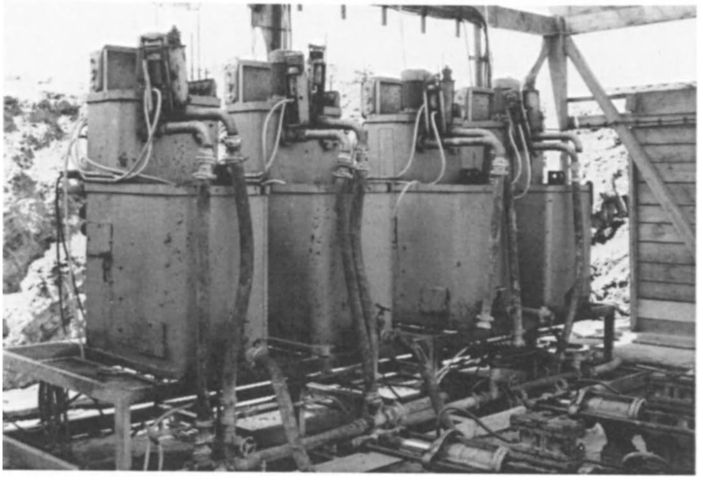


Fig. 5.7 High turbulence mixers in a grouting station

The suspension is fed from the mixers either directly to the injection pump, or to an agitator from which it is fed to the pump. A long mixing time, at a slow pumping rate, into the injected hole would overheat the colloidal mixer. A paddle mixer, or a horizontal vessel with a rotating screw at the bottom, can be used as an agitator. The capacity and the size of mixers should be such that mixes of different compositions are available in the grouting process as needed in the holes which are injected.

5.3.Injection pumps

Mostly piston double acting pumps are used for injecting suspensions, their capacity is up to 100 l/min and the pressure up to 100 bar. A type frequently used is shown in Fig. 5.8. Two pistons acting in two opposite (head-on) cylinders are fixed on a ram which is driven by a hydraulic motor so that in every cycle one of the pistons pumps grout into the injection line. The circulation is regulated by

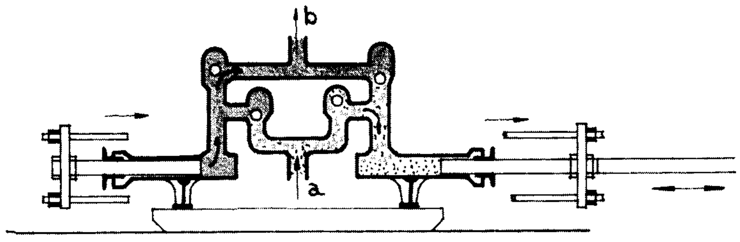


Fig. 5.8 Schematic section of a double-acting injection pump, a from mixer, b to injection line

two ball valves on each cylinder. The pumping rate and pressure are regulated by the speed of the hydraulic motor. This type of pump can pump suspensions with sand grains up to 2-3 mm in diameter. They are mounted for easy cleaning in the case of malfunction (blockage). The capacity of such pumps is about 70 l/min at a maximum pressure of 60 bar. With reduced capacity the pressure can be raised to 120 bar, and the power is 20 kW. The pressure at the outlet pulsates and the installation of some pressure equalizer is convenient for smooth grouting.

Helical screw pumps are used especially for injection of thick mixes containing a high proportion of sand, and even mortar can be pumped. They have continuous output at a constant pressure which is regulated by the rate of rotation. The section of a helical screw pump is shown in Fig. 5.9. The rotor is of a constant circular section along the helical axis which is eccentric to the rotor axis.

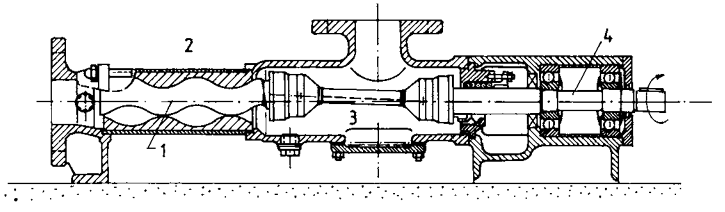


Fig. 5.9 Schematic section of a helical screw pump, 1 rotor, 2 stator, 3 driving axis with washer, 4 motor drive

The stator, usually made of rubber or some elastomer material, is an internal helix with a pitch which is twice the one of the rotor. The rotor is driven by an electric motor with a gear for smooth variation of the rotational speed.

The pressure at the output depends on the length of the screw, and in a single step about 10 bar can be attained. If more pressure is needed, two ore more pumps are coupled in series. The output depends on the size of the stator, and up to 60 m^3/hour can be achieved. The driving power in kW amounts to $N=0{,}45Q$, Q in m^3/h at a pressure of 6 bar.

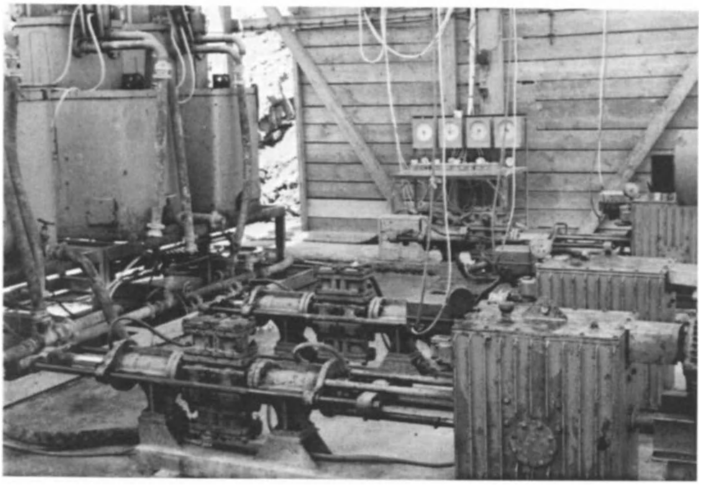


Fig. 5.10 Injection pumps and registering pressure gauges in a grouting station

5.4. Injection lines and accessories

In order to prevent clogging the lines by sedimentation of unstable suspensions, the diameter of the injection lines should not be too large. Pipes of 10 to 25 mm ID are usually taken; they should not contain restrictions which might cause plugging. One advantage of narrow pipes for the lines is that a small volume of grout in the lines has to be wasted when the grout mix is changed during the injection. The capacity of the pump to overcome the hydrodynamic resistance at a higher output on very long lines should be verified.

Two possible dispositions of the injection line are shown in Fig.5.11. The installation is simpler and cheaper with a single direct line ending at the standpipe, the flow rate is controlled at the injection pump, the grout in the line is wasted when the grouting of a section of hole is completed. An alternative disposition has a return line leading from the standpipe to the mixer , or to the agitator from which the grout is pumped. The injection pump in this case can be operated at a constant speed, and the pressure is regulated at the standpipe by manipulating the valve of the return line.

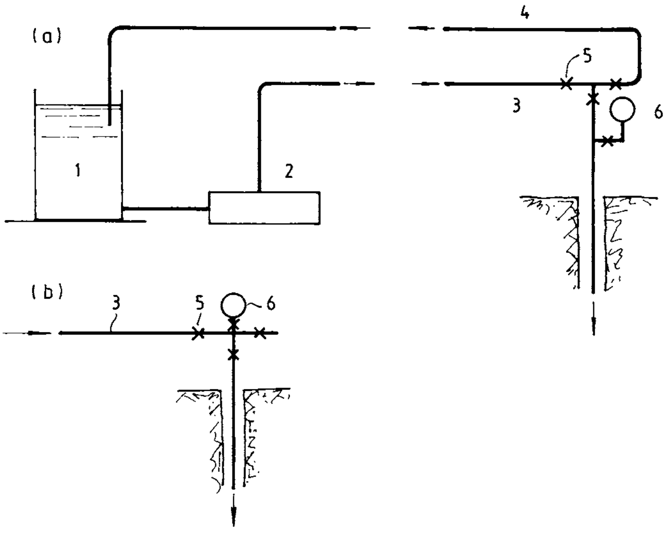


Fig. 5.11 Alternative layout of injection lines, (a) circulating with return line to agitator, (b) direct line to standpipe, 1 mixer or agitator, 2 injection pump, 3 injection line, 4 return line, 5 valves, 6 manometer

The standpipe with the fittings shown in Fig. 5.12 connects the injection line with the packer pipe on the top of the grouted hole. It is fitted with the valves for controlling input and output, and with the manometer for checking the injection pressure. In an automated and centralized injecting station, fittings for the connection with centralized pressure and flow registration units are added.

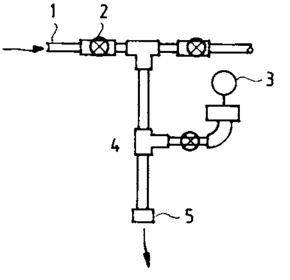


Fig. 5.12 Standpipe with fittings; 1 from injection line, 2 valves, 3 manometer, 5 to packer pipe

Packers of different constructions are used to seal the top of the grouted section of a hole. Their diameter ranges between 36 and 100 mm. Three types of packers used for injection and for water pressure testing are shown in Figs. 5.13 and 2.12.

In packers with hide plug the plugs are fixed at the end of the packer pipe. It is easy to install in the hole drilled in hard rock, and it is set tight to the bore hole wall automatically by the injection pressure which may reach up to 60 bar. However, such packers are seldom used for grouting.

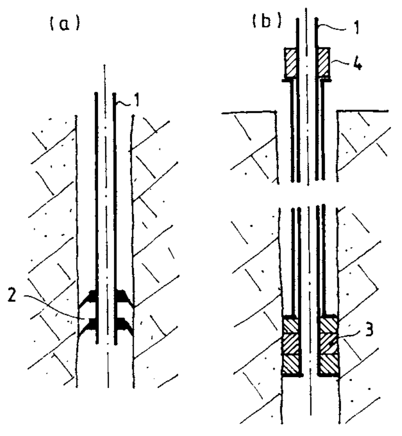


Fig. 5.13 Packer types; (a) with hide plugs, (b) with mechanically activated rubber plug, 1 packer pipe, 2 hide plugs, 3 rubber sleeves, 4 screw device to tighten the packer

The mechanical packer in Fig. 5.13(b) consists of a set of soft rubber rings on the packer pipe. The rings are expanded and pressed tight to the hole wall by pressure exerted by an outer tube pushed down by a screw device on the top of the pipes. The diameter of the rubber rings is 1-2 mm less than the effective inner diameter of the hole. The length of the assembly is usually about 30 cm and even 50 cm in highly fissured rock in order to prevent the grout from bypassing and blocking the packer. It is conveniently used in holes with uniformly smooth walls for injection pressures up to 60 bar.

The pneumatic packer (Fig. 2.12) was described in Item 2.31.

When piston pumps are used for the grout injection, it is often necessary to have some kind of *pressure equalizer* between the pump and the injection line in order to reduce the pressure pulsations in the line. A pressure absorber is used for this purpose, either a pressure vessel with a volume of air on top of the suspension, or an elastic rubber hose which absorbs the maximum energy of the pulses and releases it at the minimum. Two possible solutions are shown in Fig. 5.14.

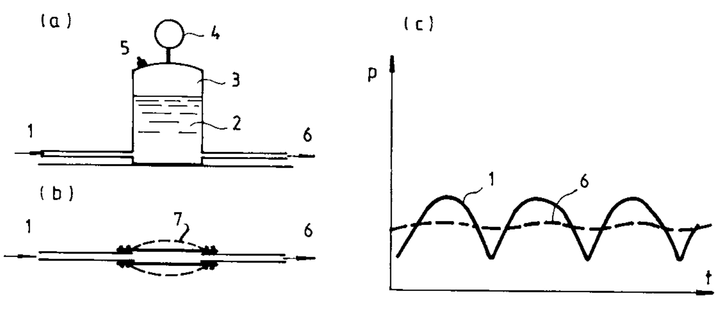


Fig. 5.14 Absorbers of pressure pulses; (a) pneumatic absorber, (b) rubber hose absorber, 1 from the piston pump, 2 grout suspension, 3 compressed air, 4 manometer, 5 safety valve, 6 to the i njection line, 7 elastic armored rubber hose, (c) diagram of pressure pulses in and out of the equalizer

5.5. Recording instruments

The main data,as described in section 4.4. and shown in Fig. 4.23, ought to be registered during the injection of any single hole section. A separate unit is used on small grouting projects for pressure recording,and one for the flow rate. Recording may be centralized on large projects in the grouting control room, and data may be collected and recorded by special electronic units.

The *pressure* is measured with manometers in the pump house and on the grouting standpipe. For the use on grout lines, the manometer must be protected from being blocked by the injected grout. Several types of preventers are used, as shown in Fig. 5.15. The manometers must be treated with care, kept clean and regularly checked for accuracy on a check board.

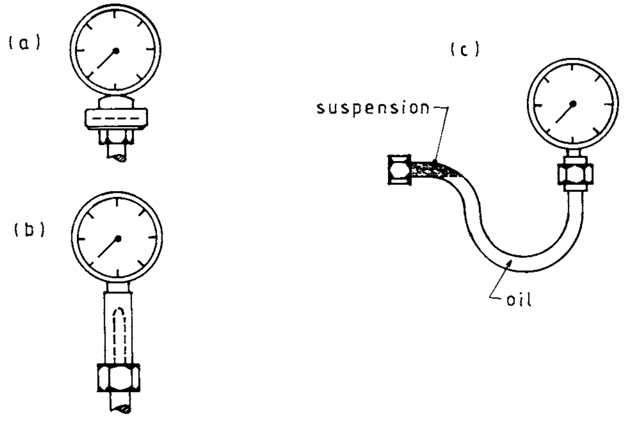


Fig. 5.15 Preventers used on manometers for injection lines; (a) with flat membrane, (b) with cylindrical membrane, (c) with oil filled U-tube

The *quantity* of injected grout is recorded by the number of batches of known volume with different mixing ratios, or by integrating the records of a special electromagnetic flow meter which registers the flow velocity at the standpipe.

The development of electronic registering instruments makes it possible to collect all data in the central control room, so that for any grouted stage the development of pressure, flow, composition of mix and injected compound is available for adequately directing the process from the start of injection to the final saturation of a grouted stage, with all data needed for statistical elaboration and presentation of the grouting results in the final report. A double packer is used for injecting individual sections above the bottom of a hole. It consists of two sets of soft sealing rings one at the end of the injection pipe, the other rubber at a distance above it corresponding to the desired length of the section. The injection pipe is plugged at the bottom, and the grout is injected trough lateral holes on the injection pipe. The rubber seals are simultaneously expanded by screwing down the outer pipe with a distance between the upper and the lower rubber seal.

5.6. The grouting plant

In a grouting plant for a larger grouting project, all machinery for weight batching, mixing, pumping and the command and recording instrumentation is assembled. The units for storing materials and for separate processes as weighing, mixing and pumping can be containerized for easy transportation and assembled on the site. A fully automated grouting plant of large capacity was installed for the alluvial grouting the foundation of Mattmark Dam where cement-bentonite and chemical compounds were injected, as described by Blatter (1961).

A much simpler plant was installed for grouting the first large grout curtain in very karstified Cretaceous limestone for the Peruča Dam in Dalmatia. A total of more than 110 km of holes up to 200 m deep were injected, and about 49.000 t of cement, clay, bentonite and sand were injected.

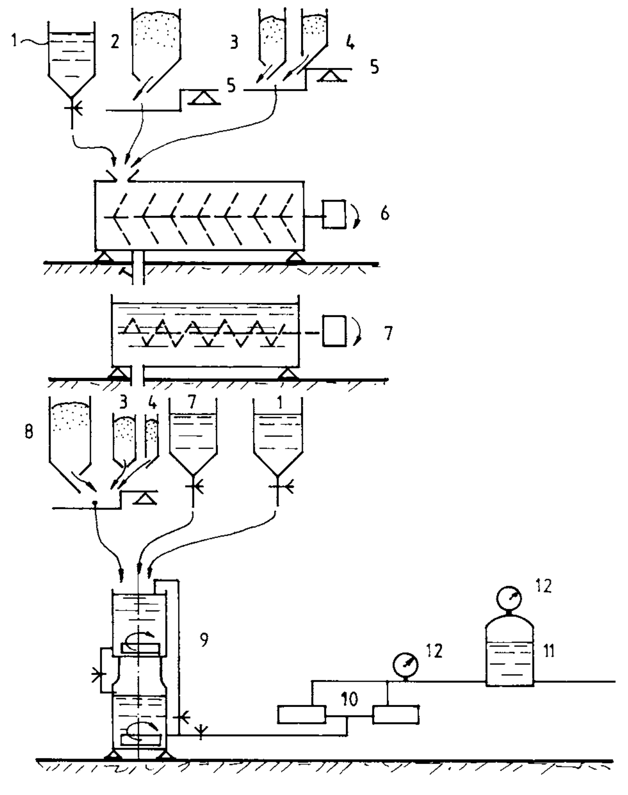


Fig. 5.16 Schematic presentation of the grouting plant for the Peruča Dam; 1 volume water feeder, 2 clay weighing hooper, 3 additives, 4 bentonite supply, 5 scale, 6 basic clay mixer, 7 agitator for basic clay slurry, 8 cement hooper, 9 double stage high turbulence mixer, 10 ram type double acting injection pumps, 11 pressure equalizers, 12 manometers

The plant was erected on sloping ground and gravity was mainly used for the transportation of the compound from one to the following stages of weighing, batching and mixing. On the upper level was the store of raw clay which amounted to nearly 75% of the mixes. The raw clay enters via a scale the wet-milling mixer for the preparation of the basic thick clay suspension which is then fed to the agitating supply tank, as shown in Fig 5.16. On the next stage were the containers for cement, water and additives from which the batches were fed into the grout suspension mixers and to the injection pumps.

All scales were operated manually, liquids were proportioned by volume, the quantity of injected compound was calculated from the number and density of the batches injected into the single sections of the holes. The injection pressure was registered by manometers. The maximum capacity of the plant was 2.500 m of injected holes per month with a mean grout compound consumption of 360 kg/m.

Fig. 5.17 View of the grouting plant for the Peruča Dam

A telephone net was installed for communication between the injection sites and the plant. CB radio communication has been used recently for this purpose. At some sites all working places are connected to the central control room by a closed circuit TV.

6. THE GROUT CURTAIN

6.1. Seepage through the dam foundation

The impounded water in a reservoir radically changes the pattern of seepage through the foundation under the dam. The seepage of ground water in natural conditions is predominantly perpendicular to the river flow. The ground water level in the abutments may be high in regions with high precipitation and low ground permeability, it may be close to the river level if the permeability of the rock is high, and even below the river level, which is often the case in karstified regions. When impounding the reservoir, the potential head on the valley bottom and the surface of the abutments is increased, and the pattern of ground water flow is changed drastically. This would cause a radically increased ground water flow through the foundation below the dam with the following consequences:

- increased uplift on the dam foundation plane, which may impair the stability of the structure;
- seepage flow through fissures and pores in the foundation material may cause regressive erosion which would progressively: increase the natural rock permeability and cause hydraulic failure in a granular soil;
- water losses by seepage flow may impair the function of the reservoir and it would be economically unfeasible.

Unacceptable consequences of these processes can be alleviated by some of the following measures:

- implementing drainage facilities at the exit of the seepage flow;
- placing an impervious blanket on the valley floor and the abutments upstream of the dam;
- constructing an impervious curtain in the foundation material which diverts the seepage flow vertically into deeper less pervious strata, or by combinations of these measures.

The influence of draining, of a blanket and of a curtain on the seepage flow under several types of dams is shown in Fig. 6.1.

A vertical impervious curtain under the dam can be constructed as a diaphragm wall made of plain or reinforced concrete in stiff materials, of clay-cement plastically deformable concrete in compressive alluvial deposits, or as an injected grout curtain in both types of materials.

A diaphragm wall is constructed in weak rock or in alluvial granular deposits and it makes a barrier of low permeability ($k < 10^{-6}$ cm/s); plastic clay-cement concrete is preferred in alluvial materials because its deformability matches the one of the surrounding ground. Since stress concentrations and large strains on the contact with the dam core are avoided, such a diaphragm is cheaper than the one made of normal concrete.

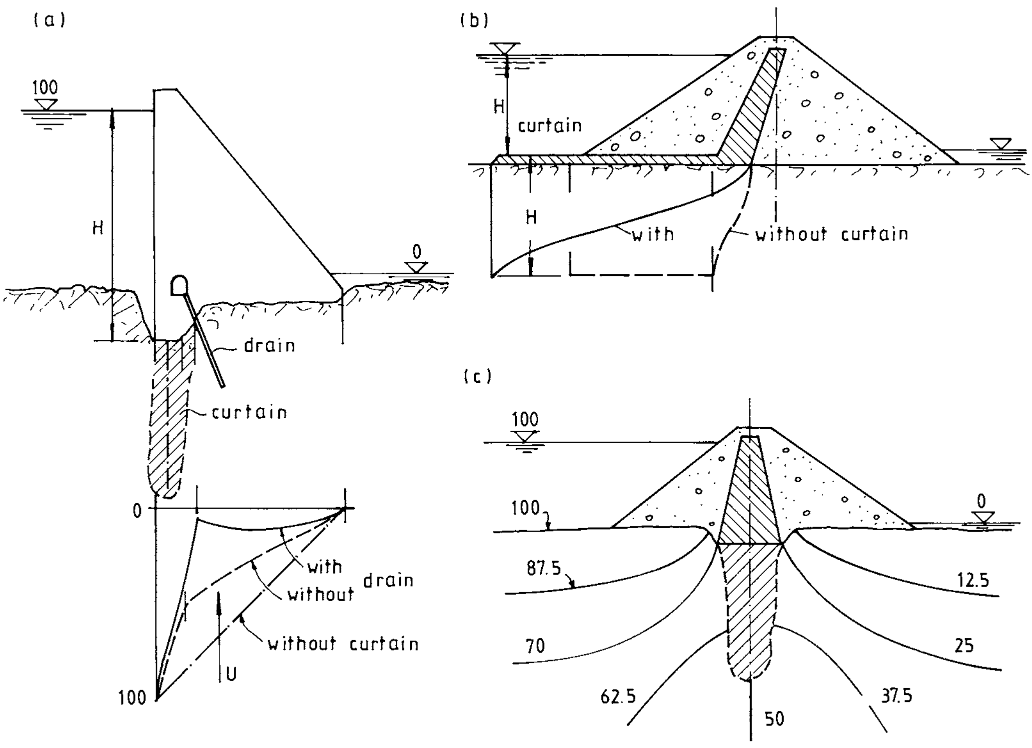


Fig. 6.1 Influence of remedial measures on seepage flow and uplift under dam foundations; (a) massive dam without and with drainage, (b) embankment dam on a pervious foundation without and with upstream impervious blanket, (c) equipotentials under the dam with a grout curtain

A grout curtain must substantially reduce the permeability of the ground and it must be wide enough in order to influence significantly the flow net and the seepage flow. Its bottom should preferably reach less pervious strata in order to reduce substantially the flow and the exit gradient of seepage. The minimum average permeability of an injected rock that can be achieved by grouting is in the range of 0.3-0.6 LU, but average permeabilities in the range of 1-3 LU can be regarded as very satisfactory.

A grout curtain should therefore never be regarded as an impervious barrier to seepage flow. Its purpose is to treat the ground systematically so that all existing paths of high permeability are intercepted and sealed, an average low permeability in the grouted region is secured, and the possibility of concentrated large leaks is prevented. Grouting can be regarded as an active continuation of the foundation exploration.

6.2. Design parameters for grout curtains

The principal design parameter for a grout curtain is the permeability of the ground below the dam and the character of the joints and fissures of the ground. From the geology of the region, the geotectonic features and the data on the permeability of the formations, which are determined from the results of the exploration, a model for the study of the problems should be prepared. It must include the following data:

- the range of permeability values for every formation along the grout curtain;
- the permeability on particular faults or other tectonic features;
- the ground water level in different lithologic formations along the longitudinal section of the foundation.

The permeability of the formations is generally higher at the surface than at greater depths as a consequence of stress release during the valley formation. This general trend can be altered by the consequences of tectonic disturbances, dissolution and leaching by percolating water, or by hydrothermal processes. When soluble rocks as limestones, dolomites and anhydrates are present the formations at a great depth may be very pervious, and the permeability may be rather erratically distributed. Examples of such formations are given in Item 6.61. The possibility of the occurrence of such singularities may be decisive for the selection of the best location for the dam axis and the axis of the grout curtain.

On the other hand, the pressure gradient through the injected zone of the grout curtain is highest at the ground surface and zero at the bottom. In addition, the possible injection pressure is lowest at the ground surface and highest at the bottom of the curtain. Thus the conditions for grouting are contradictory - at the ground surface where the greatest grouting efficiency is needed the possible injection pressure is low, while at the bottom, where it is low, the greatest injection pressure can be applied resulting in very efficient grouting. The solution of this controversy is in a denser hole spacing and a wider injected zone in the upper part of the curtain and wider hole spacing at a greater depth of the curtain.

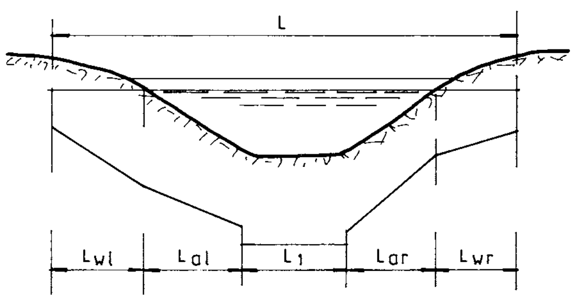


Fig. 6.2 Different zones of grout curtain along the dam axis

Three zones are distinguished along the axis of the grout curtain, as shown in Fig. 6.2:

- the deep valley or central section on which the full hydrostatic pressure of the impounded water acts, with a length L_1, ;
- abutment sections L_{ar} and L_{al} to the end of the dam crest;
- wing section L_{wr} and L_{wl} from the end of the dam crest into the abutments.

The total length of grout curtain L depends on the properties of the foundation material in the abutments.

The criterion for the determination of the depth of the grout curtain was first defined by Lugeon (1933). He proposed that the curtain should extend the into rock with permeability less then 1 LU if the dam is more then 30 m high, and less than 3 LU where it is less than 30 m high. These criteria were intended for concrete gravity and arch dams in the European Alps, but until recently they were generally adopted for the design of grout curtains. As the average permeability of grouted rock is generally in the range of 1 to 3 LU as mentioned previously, it is not necessary to grout rock less permeable than 1 to 3 LU. This is the limit of permeability which can be achieved by means of grouting.

Experience gained over years on successfully constructed dams has demonstrated that Lugeon's criteria are very restrictive in most cases. The first question to be answered on every project is whether a grout curtain is needed at all, and what should be achieved by injecting a grout curtain. In order to rationally answer these questions the influence of a grout curtain on the volume of seepage flow and on the safety of the dam structure should be studied systematically.

An important question is the limit permeability which must be attained in the grouted rock volume, which in many projects is specified as 1 LU or less. This is very restrictive and often it can not be attained at all, or at an extremely high cost. For this purpose Houlsby (1977) has elaborated a useful flow chart,which is reproduced in Fig. 6.3, as a guide for decisions to which permeability limit the curtain should be injected. The chart shows that the reasons to grout a curtain tighter than 3 LU can be an economic loss by seepage of very valuable water from the reservoir (as from pumped storages) or the need to control erosion of scree in the joints. In other instances much higher average permeability of the grouted curtain can be accepted. In cases when the seepage water has a low value,the limit permeability, to which the curtain should be injected, depends on the type of the dam and on the danger of erosion. The chart is a general guide for decision making on rock grouting in the shallower zones. At greater depths,where the pressure gradient of impounded water through the curtain is lower, even higher permeability may be acceptable. In any particular case the final decisions should be made based on a detailed study and on the consideration of all relevant exploration data and characteristics of the dam project.

6.21. The effect of the grout curtain on the hydrodynamic potential field

From the impression that the permeability of a grout curtain is very low its effect on the hydrodynamic potential field is often greatly overestimated in dam design. In fact, a permeability of 1 LU

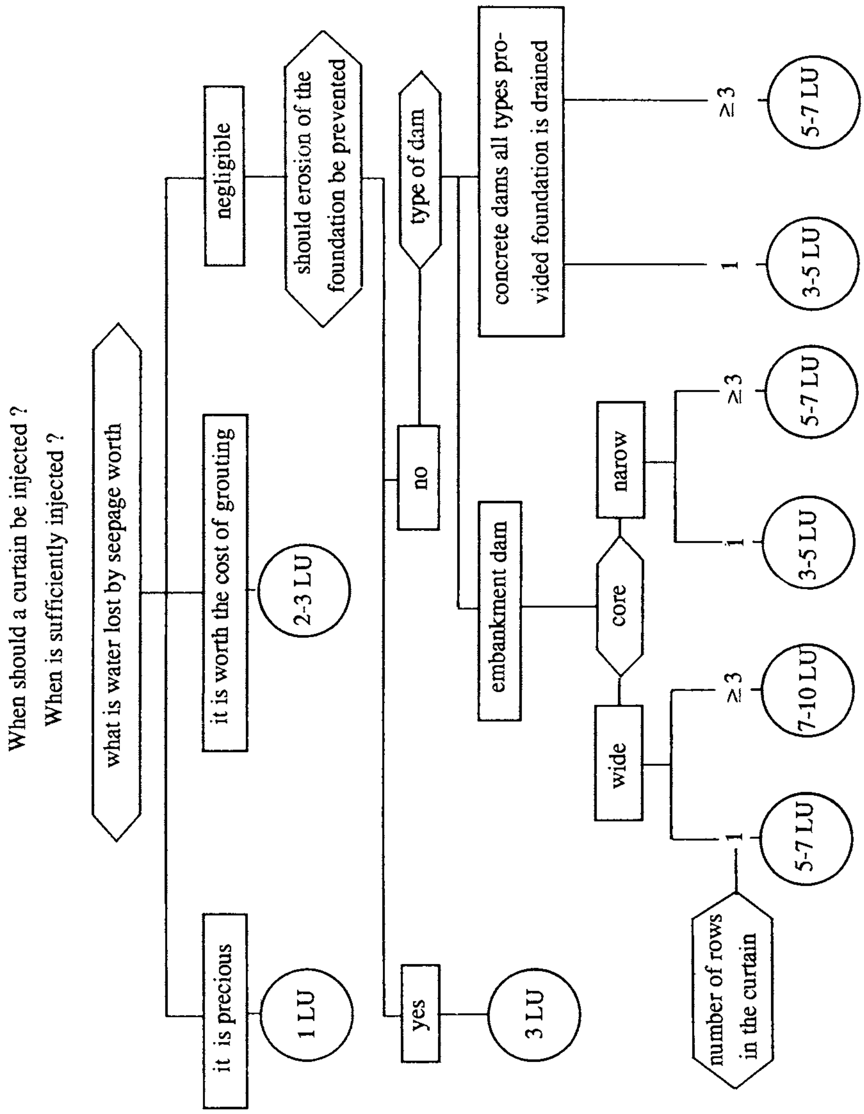


Fig. 6.3 Flow chart as a guide for deciding when grouting is needed and, if needed, to which limit permeability

corresponds to the permeability coefficient $k = 1{,}3 \times 10^{-5}$ cm/s and it is limited to a relatively narrow injected zone of the pervious foundation ground where the pressure gradient from the impounded water is concentrated. This effect can be visualized by computer solutions of the equipotentials with boundary conditions and permeability coefficients of the geotechnical model of the foundation domain, which was shown in Fig. 2.3 for a stratified foundation and a grout curtain of a different depth. From this solution (Nonveiller, 1970) the effect of the curtain depth on the seepage flow was computed and it is presented in Table 6.1. It is seen that with grouting to 2 LU only, the upper bed

TABLE 6.1

Effect of the grout curtain depth on the seepage flow rate in stratified ground

Depth of curtain m	permeability of bed LU	permeability of grouted zone LU	seepage flow l/s m	reduction of flow without curtain %
0	-	-	0.66	0
25	80	2	0.18	73
75	20	1	0.08	88
95	5	1	0.07	89
255	5	-	-	-
deeper	0	-	-	-

of 80 LU permeability reduces the seepage flow to 27% of the one without grouting, further deeper grouting through the 20 LU bed reduces it to 12%, and 25 m deep grouting into the 5 LU bed to 11% of the seepage without any curtain. Evidently, in such a case deep grouting which is an expensive undertaking, brings very little benefit.

The length and the depth of the wing curtains depend on the local geotechnical conditions, but the depth and grouting criteria suggested in Fig. 6.3 can be lessened with the distance from the shore line, because the hydraulic pressure gradient decreases with the distance from the shore line of the reservoir, and only the economic benefit from reduced seepage loss could justify grouting. The wing curtains can be injected along a line perpendicular to the slope of the abutment, or alternatively parallel to the shore line. The first alternative requires rather deep idle drilling or, in the case of steep abutments, drilling from an adit. But, since the grouted curtain is deep below the abutment surface, sounder rock is generally found. For the second alternative no grouting adit is needed, but a worse rock condition may be present and the cost of grouting is increased. The sketches in Fig. 6.4 illustrate the fact that the influence of the length of the wing curtain on the exit gradient and on the seepage flow is not substantial.

In order to illustrate the difference between the disposition of the curtain along the shore and the one deeper in the abutment, the results of exploration for the wing curtain of a dam in the West of Cyprus are presented. The embankment dam is 90 m high, the left abutment is a narrow ridge which consists of chaulky limestone and marl formations. Very high permeability was determined from exploration drilling and from a grouting test plot on the shore of the reservoir. Grouting proved that even with an extremely high grout consumption of 1,600 kg/m cement, the desired permeability reduction could not be achieved. On the suggestion of the Panel of Experts, the owner decided to displace the grout curtain into the middle of the ridge, where exploration indicated much lower permeability, and an average grout consumption of 270 kg/m cement with efficient closure was achieved.

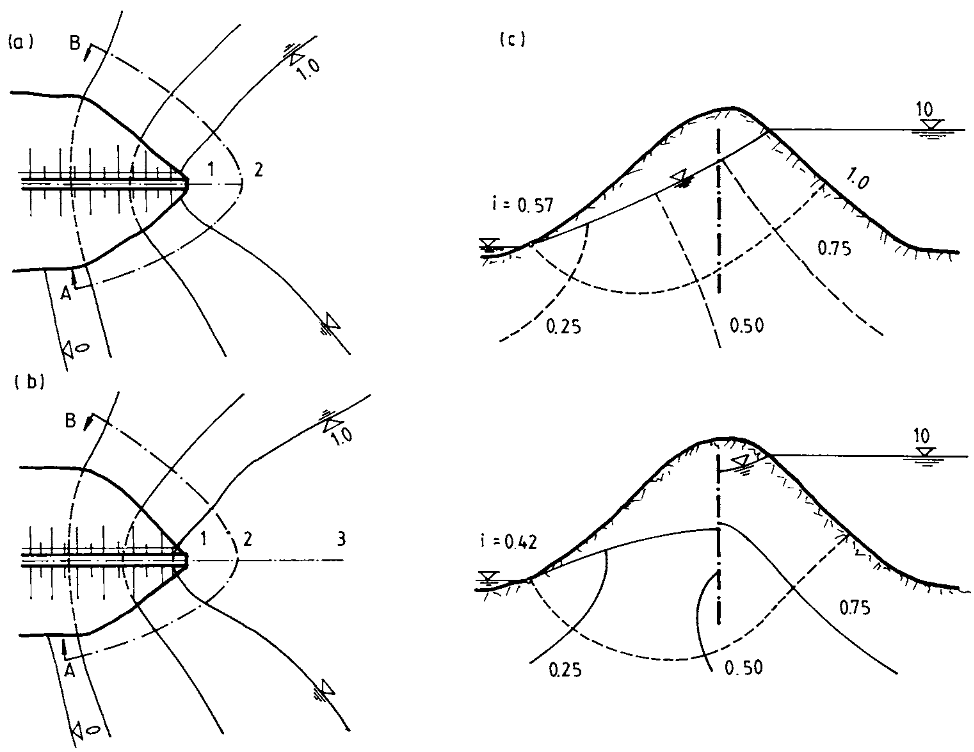


Fig. 6.4 Sketch of the influence of wing curtain on the seepage flow and exit gradient in section AB; (a) plan of short -, (b) of long wing curtain, (c) section along the line A-B with hydrodynamic flow net of both alternatives with exit gradients, 1-2 wing curtain in (a), 1-3 wing curtain in (b)

In order to evaluate the effect of the depth of the curtain, the equipotential field was computed with a FEM program for a typical cross section of the 250 m long ridge. The main features of the geologic stratification and permeability data are shown in Fig. 6.5. The seepage flow was computed for the section without a curtain, with a curtain through the upper more pervious layer, and with a curtain 30 m deep into the deeper marly bed of lower permeability. The computed seepage flow amounts to 0.32 l/s m through the section without a curtain, 0.27 l/s m when only the upper more pervious layer is grouted, and 0.25 l/s m when the curtain reaches 30 m deep into the lower less pervious layer. The downstream exit pressure gradient was in the first case $i = 0.74$, in the second $i = 0.74$, and $i = 0.74$ in the third one. It is evident that in such geologic conditions the benefit of grouting the deeper less permeable layer is marginal.

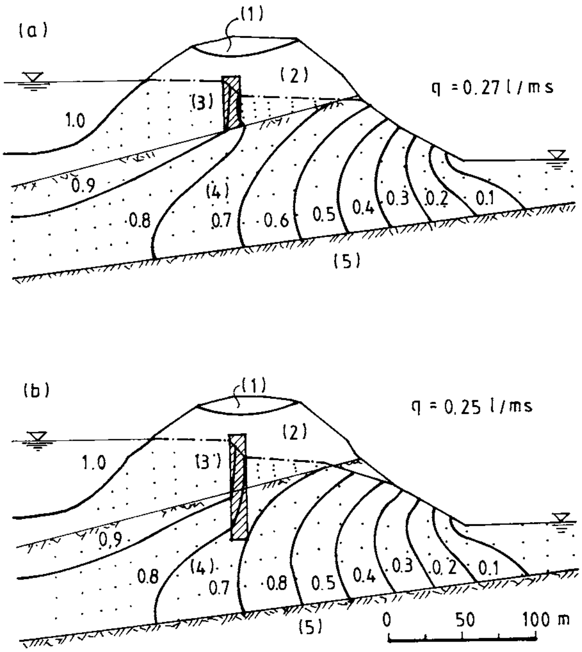


Fig. 6.5 Influence of the depth of wing curtain in a stratified abutment on the seepage flow; (a) curtain trough the upper more pervious bed, (b) curtain 30 m deep into a less pervious lower bed, 1 chaulky limestone bed, k_h=6.5x10^{-6} m/s, k_v=1.3x10^{-6} m/s, 2 terrace gravel, 3 grouted zone, k=0.26x10^{-6} m/s, 4 marly chaulk, k_h=1.3x10^{-6} m/s, k_v=0.2x10^{-6} m/s

6.3. Injection of grout curtain to required permeability standard

The permeability standard of the injected grout curtain, as described in Fig. 6.3, should be flexibly defined in the design specifications of the grout curtain, so that the contractor can best adapt the grouting procedures to the variable conditions of jointing and the permeability of the rock. The means to achieve the best and uniform permeability standard are:

- systematic split spacing of the injection holes;
- one or an odd number of injection hole rows should be grouted;
- the appropriate grout composition should be selected.

The choice of the basic spacing of holes, the number of rows of injection holes and the best grout composition depend on the character of the rock formations, joints and fissures, the presence of the percolating ground water, of large voids or caverns. In order to define the grouting parameters and the most convenient details of the injection procedure, one or several grouting test plots are injected, usually on the line of the curtain in different rock formations. In order to obtain statistically relevant results, the length of a test plot should encompass at least three primary injection holes. It is useful to core and water pressure test injected sections of all holes in the test plot. A detailed geologic interpretation of the cores is essential, including the study of the systems of joints and fissures so that relevant details for the evaluation of the test grouting results are collected. The results of grouting every set of holes should be interpreted currently and correlated to the geotechnical data and permeability, so that the appropriate hole spacing, depth of the sets of holes and injection pressure can be defined, and the most appropriate grout composition and water ratio selected, from which the saturation criteria of the grouted hole sections can be specified.

It is convenient to start the production grouting of the grout curtain by first dividing it into sections encompassing from three to four primary holes and to drill first these marker holes at the ends of the sections as exploration holes which are cored, water pressure tested and then injected. The results obtained are compared with the results of the grouting test plots and details of the injection process in the corresponding section are set. Then production grouting proceeds with the remaining primaries between these marker holes, followed by the secondaries, tertiaries and higher order sets as required. Important data from marker holes, beside the usual ones, are the character and the width of fissures and joints, their interspace, the permeability of the rock before grouting and the grout consumption to achieve the saturation criteria. From these data, in addition to the data from the grouting test plots, the detailed injection procedure for all holes in the corresponding curtain section is determined.

The depth of the secondary holes should lag behind the primaries by a few stages, usually not less than two, so that a clear distinction of the ground permeation from one and the other set of holes is retained. According to our experience, it is not important to limit the time between injecting a stage of the next set of holes at the depth of the previous set, because the grout in the fissures gets stabilized during the saturation phase, and the cohesion alone is sufficient to prevent any disturbance from the newly grouted neighboring stage of the higher set of holes.

The effect of grouting of successive sets of holes before injecting the following set can be best evaluated by measuring the permeability of the set before grouting. This is however, expensive and time consuming and it introduces unnecessary water in the stage to be grouted. A convenient alternative is to compute the permeability from the grout flow rate and injection pressure p_o at the start of grouting as mentioned in Chapter 4.4. Eq. (2.1) can be modified for a greater accuracy by introducing the ratio of viscosity η_s of the grout suspension, and the one of water η_w resulting in:

$$N = 10\,Q\;\eta_s / p_o\,L\,t\,\eta_w \qquad (6.1)$$

The precision of such a permeability evaluation is not great, but it is sufficient as a guide so that it enables the making of the correct decisions in directing the grouting process and to decide whether the required permeability standard is achieved, or grouting of the next set of holes is necessary. The ratio of the grout take between the previous and next set of holes is another practical criterion for quick decision making. Klosterman et al. (1982) maintain that a reduction to 65% of the take in the higher order set is a satisfactory result. Fergusson et al. (1964) regard a reduction between 20% and 80% as satisfactory.

After the grouting of a longer section of the curtain is completed, check holes may be drilled for water pressure testing at locations where the results of the ultimate set of grouted holes may indicate that the permeability may not satisfy the required standard. Then additional holes are drilled and grouted where the standard is not satisfied. Some specialists prefer to drill inclined check holes in the plane of the curtain for permeability testing. Such practice is not reasonable when systematic split spacing is adopted for grouting, because the unsatisfactorily injected zones may be reasonably expected to be always parallel to the injected holes, and between the last injected set of holes and the adjacent ones. In any case if split spacing is methodically carried out, the data from the last set of injected holes will give a reliable indication of the result achieved, and check holes for water permeability tests will only exceptionally be needed where in the last set of holes the specified permeability standard was not satisfied.

The observation of the ground water level in piezometers after impounding the reservoir will provide the final proof of the efficiency and the permanence of the grout curtain.

6.31. Number of grout hole rows

A single row of injected holes should be implemented whenever possible. Two exceptions to this general rule may be:

- in the zone below the foundation surface;
- when the rock is very permeable.

In the first instance the hydrostatic pressure difference from upstream to downstream is high and a greater thickness of the injected zone is required, which could not be efficiently grouted from a single row of holes. The holes of all rows should be injected to the same tightness standard so that the permeability is uniformly low along the whole curtain. There is not a generally accepted rule for the selection of the thickness of the grouted zone and some judgement will be necessary to define the appropriate thickness. The thickness of the zone will depend on the:

- type of dam;
- the kind and properties of the foundation rock;
- the fissuration, the infill of the fissures and its erodibility.

The upper zone of a curtain below a concrete dam on sound rock,with narrow non-erodable fissures, may be narrow and the acceptable hydraulic gradient may be high when the water loss from seepage flow is not important. Below the foundation of an embankment dam a wider injected zone is preferable, so that the exit gradient of the flow below the foundation remains low, especially if the fissures of the rock contain erodable materials. An example of a highly fissured foundation rock at the 75 m high Sklope Dam is shown in Fig. 6.6. The foundation rock is limestone of the Oligo Miocene age, very fractured, the fissures are up to a few centimeters wide and filled with erodable gauge. The curtain was injected from a grouting gallery which provides a reliable non-erodable foundation contact with the clay core. Grouting was started after more than 10 m of embankment material was already placed to provide a sufficient overburden pressure, so that the grout could be injected at a pressure higher than 10 bar. A fence of subhorizontal and inclined holes was injected from the gallery providing a more than 20 m thick grouted contact zone at the foundation plane.

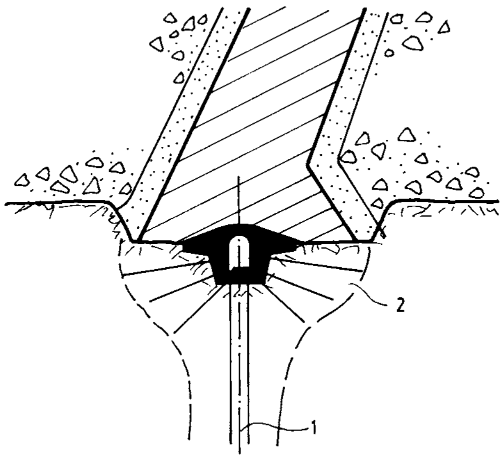


Fig. 6.6 Contact of clay core and grout curtain of the embankment Dam Sklope, 1 grout curtain with three rows of holes, 2 zone of contact grouting of foundation below core

When the rock is very fractured and permeable it is a good practice to inject three rows of holes. The two outer rows are injected first with a low grouting standard, so that the grout consumption is checked within economically feasible limits. Then the middle row can be injected uniformly to the required high standard without an excessive grout consumption.

It is an established fact that the weight of the dam compresses the foundation rock and the systems of fissures, to a degree which reduces its permeability, especially in the upper zone of moderate depth. Therefore the permeability of the injected zone of the grout curtain is reduced as the height of the dam increases. This fact was reported by Matsumoto (1985) who measured the permeability of the injected curtain of the Maekawa Dam during successively higher stages of its construction. The permeability of a few injection holes below the grouting gallery was measured in the section at the depth between 10-15 m before the filling of the embankment started and after it reached different elevations above the foundation plane. The results of these measurements are presented in Fig. 6.7. It is evident that the permeability of the grouted stages was reduced by one order of magnitude when the dam height was increased by 25-30 m. The economic consequence of this finding is remarkable: when the foundation plane is loaded by the dam weight, the injection pressure can be higher and the spacing of the holes larger, less sets of holes are needed for the closure to the required permeability standard, the grout consumption is reduced and the efficiency of grouting is increased. This important point is seldom considered in grout curtain design and construction.

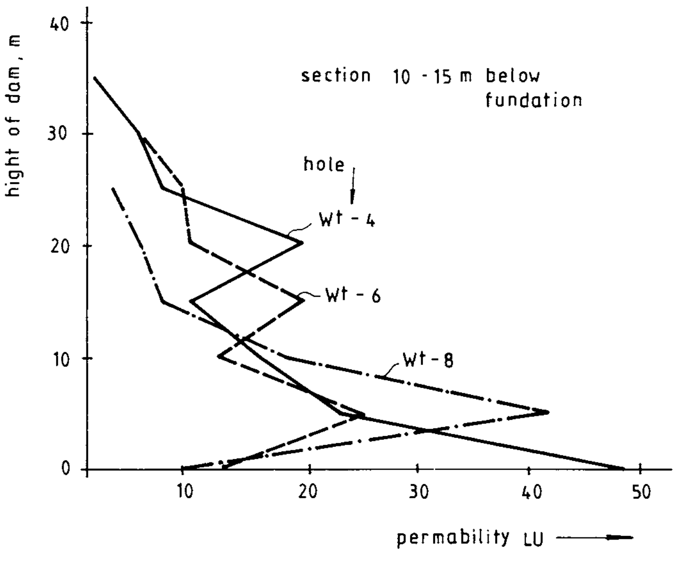


Fig. 6.7 Reduction of rock permeability with increasing embankment height at Maekawa Dam (Matsumoto, 1985)

6.32. Selection of grout compound

Although the properties of grout suspensions were discussed in detail in Section 3.2, some additional comments should be of some interest. The fissured foundation rock below dams will usually be injected with grout suspensions which may penetrate into the fissured ground with a permeability which is more than 1 LU, while formations with a permeability of less than 1 LU need not be injected at all, as shown in Fig.6.3. Therefore, there will never be a need to inject chemical compounds into fissured rock.

Suspensions injected into fissured rock formations must satisfy the following conditions:

- the ingredients of the suspension should be sufficiently fine to penetrate fine fissures;
- the injected grout should develop sufficient strength in the fissures so that the grout can not be squeezed or eroded out of the fissures by the hydrostatic pressure;
- the injected grout should develop a sufficiently low permeability.

As discussed in Section 3.2 Portland cement with a Blaine fineness more than 2,700 cm/g is generally satisfactory for injecting rock with a permeability more than 1 LU , especially when the suspension is mixed in high turbulence mixers. When rock with a permeability less than 1 LU must be injected, extrafine cement with Blaine fines up to 8,000 cm/g may be needed, but this may be achieved by a special treatment of coarse cement in the mixing plant, as described previously.

The necessary unconfined strength of the injected grout in a planar fissure shown in Fig. 6.8 can be computed from Eq.(3.9) with a safety factor F_s as:

$$q_u = h_p \gamma_w \delta F_s/L \tag{6.2}$$

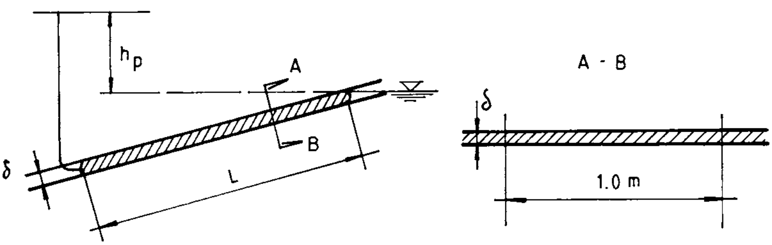


Fig. 6.8 Planar fissure drilled with grout

If a 0.1 m wide fissure filled with grout on a length of $L=1.0$ m under $h_p=100$ m head of water is assumed, the unconfined strength is $q_u = 100 \ 10 \ 0{,}1 \ F_s/1.0 = 100F_s$. It was shown in Fig. 3.19 that the unconfined strength of samples of a very lean compound, with only 25 % cement and 75 % clay developed unconfined strength $q_u=5.2$ MN/m (filtered samples) and $q_u=1.8$ MN/m (unfiltered samples). The resulting minimum safety factor against squeezing the grout out of the fissure in this extremely unfavorable example is $F_s=1.8/0.1=18$. It is evident from this extreme example that the strength of the injected grout is not in any case a parameter governing the choice of the grout compound.

Alluvial strata with grains $D_{15} > 0.4$ mm can be injected with suspensions of cement and clay. Only special fine cement (Blaine fines 8,000 g/cm^2) can be injected but the results are not very reliable. Specially treated cement-bentonite suspensions may be injected in the first set of holes equipped with sleeve grout pipes, as described in detail in Section 4.5.

The grout curtain in alluvial material always consists of many rows of sleeve grout holes. The holes are arranged in two or three sets. The first set is grouted with cement-bentonite suspensions specially treated in the mixing process, as described previously in order to obtain a very fine grained suspension, which can penetrate the ground with grain size $D_{15} > 0.4$ mm (Fig. 3.1). The number of rows and the width of the curtain is selected so that the pressure gradient across the curtain does not exceed $i=1.5$. An equidistant arrangement of injection holes, as shown in Fig. 6.9, is generally chosen. The primary holes are at the intersections of the rows ***A*** and ***B*** at primary spacing 2 a, the secondaries between the primaries on the same rows, and the tertiaries on the lines ***C*** between the lines ***A***. The primaries are first injected, then the secondaries with cement-bentonite suspension. The tertiaries are then injected with chemical solutions or resins as the case may require.

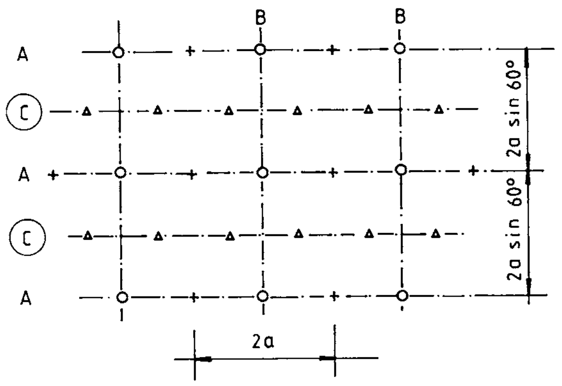


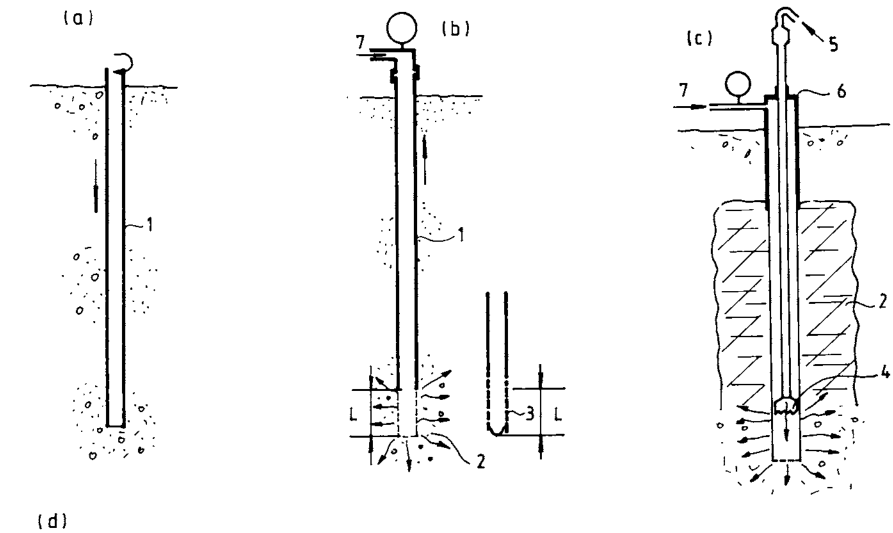
Fig. 6.9 Equidistant arrangement of injection holes for alluvial grouting

Since grout sleeve injection of the alluvial ground is very expensive it is used where reliable uniform grouting results are essential. For auxiliary purposes, such as the grout curtains for temporary structures for river diversion, foundation excavations in alluvial ground etc., the simpler method of injecting through the hole casing or the drilling rods up or down the hole can be applied. The principle is illustrated in Fig. 6.10. When grouting up the hole, the hole is drilled and cased to the bottom, then the casing is connected to the injection line and withdrawn for the length of the section L, and either a suspension or a chemical solution is injected to refusal or to the required specified volume. If the hole walls would collapse without a casing, the grout is injected through its lower perforated end to the required length L.

In the down hole method the hole is drilled and cased to the depth of about 3,0 m, then a circuit tube 6 is permanently installed and the drilling is continued in short stages which can be supported by the grout. The grout is then injected as specified and the process is continued to the required depth. In some cases it is possible to omit drilling and simply drive a casing with the lower end perforated along a length L, as shown in Fig. 6.10(d), then to grout the section and the process is continued to the wanted depth. A drawback of these through the casing grouting methods is the possibility of grout outbreak near the ground surface. Only the sleeve grouting method is recommended for reliable and uniform grouting of alluvial soil.

6.4. Technical specifications for curtain grouting

Technical specifications for curtain grouting should be flexible enough to allow the contractor to make a reasonable cost and the time delay estimate of the activities to be tendered and the owner to direct the activities so as to best meet variable conditions which, within the frame of the exploration results may greatly vary from one location to another. A rigid specification of all the activities in every detail within narrow limits may result in high unit prices because the contractor must tend to



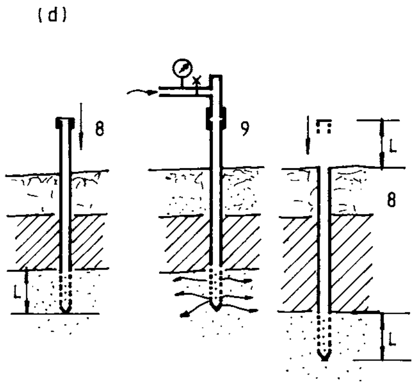


Fig. 6.10 Alluvial grouting of cased holes; (a)drilling and casing for up the hole grouting, (b) injecting with lifting of the casing, (c) drilling and injecting down the hole, (d) down the hole grouting with driven casing, 1 casing, 2 injected hole section, 3 perforated end of casing in collapsible soil, 4 drilling rod with a chopping bit, 5 flushing and injecting swivel, 6 circuit tube and fitting, 7 injection line, 8 driving phase, 9 injection phase

cover possible risks which arise when the tolerated specified limits can not be satisfied due to local variable characteristics of the ground. On the other end are the specifications which are too general and leave to many decisions either to the Owner or to the Contractor. In both cases diverging interpretations of the local condition and the ensuing conflicting views among the contract parties on the appropriate method of drilling, ways of injecting and other measures provided to achieve the specified results may lead to time delays and claims by the Contractor.

Well conceived technical specifications should contain descriptions, instructions, and detailed regulations by means of which all kinds of activities to be carried out under the contract are well and clearly defined without ambiguities, so that the contractor can have a clear picture of the logistic problems and organization details and make a reliable estimate of the cost and time delay for every particular item of the bill of quantities. Experience shows that it is favorable to split the bill of quantities into separate items for any particular kind of activity and not to combine different kinds of activities into one item. The natural ground conditions of the site should be clearly described, as well as the possibility of unknown surprises where they may be expected. The specifications should leave the Contractor and the Engineer sufficient freedom to adapt details of specified procedures to changed actual local conditions. It is convenient to provide special items in the bill of quantities for site mobilization before the start, and clear up after the completion of the project and payment for the necessary infrastructure (power, water supply, compressed air, access roads to site etc.).

Drilling and grouting activities should be split into separate items paid by unit prices according to the completed amount of work. Separate items should be provided for exploration, production drilling and for inclined holes, and holes drilled in restricted spaces (tunnels, galleries). It is advisable to plan extra funds for any setup of the drilling rigs as needed during the execution of works, coring and handling of the extracted cores, water pressure tests and measuring the hole inclination. Hookups of the injection line should also be paid for.

6.41. Digest of Technical Specifications for curtain grouting works

I. Types and volume of works

A condensed description of the geotechnical and hydrogeologic characteristics of the ground along the grout curtain profile should be given. The way and time of execution of works is described, i.e. "..drilling and grouting shall start at the valley bottom after foundation excavation and proceed to the and up the abutments following the foundation excavation.. " or ".. drilling and grouting shall start at the wing curtains and the abutments prior to foundation excavation until it reaches the foundation excavation zone, then it shall be transferred into the grouting gallery in the valley bottom where it will proceed from the completed gallery up the abutments.."

The center line and the depth of the grout curtain shown in the contract drawings and the hole spacing and depth are the result of relatively limited information from the exploratory works and the interpretation of the available geologic and hydrogeologic data. Additional exploration holes will be drilled before grouting operations start at the location of every 4^{th} or 6^{th} primary injection hole. These holes will be drilled to the bottom, continuously water-pressure tested in 5 m long sections and cased if necessary. These holes will be injected from bottom up when their turn comes in the normal sequence of injecting. The final volume and depth of grouting will be based on the contract drawings and on the additional information obtained from these exploration holes. The locations of one or several test grouting plots on which grouting starts will be established as needed. From the results of the grouting test plots, the basic hole spacing, the depth of grouting, the number of sets of injection holes, injection pressure, saturation criteria, selection of grout compounds, water ratio and other details of the curtain design, will be verified or changes made as required.

If the resulting final amount of work differs for more than 20% from the contract cost, the Contractor may claim changes in the agreed unit prices and the completion date of the grout curtain.

It is understood that the Contractor has offered the works under contract after a thorough study of the data on the ground characteristics, the natural site conditions, the possibility of local supplies, communications, accommodations and the Technical Specifications and that he has fully accepted them.

II. Drilling and grouting plant and equipment

It is useful to oblige the Contractor to provide a complete list of all the drilling and injecting plants with the characteristics and the capacity of the drilling rigs, mixers, injection pumps and of all the necessary equipment for the completion of the grouting works. The Engineer is entitled to check the list and suggest or require changes as he considers necessary in order to be sure that the specified quality of works and the completion date will be reached.

II-1 Drilling rigs and equipment

For exploratory holes, rotational drilling rigs for 101 mm Dia holes or more and to the required depth,should be available. Thin walled coring bits inlaid with carbide or diamonds in order to meet the rock characteristics should be used, and core barrels of adequate type to ensure the best coring and core recovery.

Injection holes may be drilled with percussion drilling rigs and noncoring bits. Only clean water or air must be used for flushing. The diameter of injection holes should be neither in excess of 66 mm nor less than 36 mm.

II-2 Plant and equipment for grouting

A centralized weight batching and mixing plant will be installed,from which grout is supplied by means of injection lines (single or return) to the injection holes . Exceptionally a central weight batching plant may be installed for premixing the grout compounds, which are then finalized as required at mobile mixing and injection plants near the actual injection sites in which the mixes are diluted to the proper water ratio.

All grout compounds should be stored at or in the weight batching plant in quantities sufficient for a week of continuous grouting.

The suspensions will be mixed in high turbulence mixers approved by the Engineer. From the mixers the suspension batches are delivered either directly to the injection pumps or to agitators from which the injectors are fed by means of direct or circulation injection lines.

All data on injected holes: composition of mixes, injecting pressure and flow rate of suspension must be made visible on displays in the command room of the weight batching and mixing station.

Hydraulically driven double acting piston pumps should be used for injecting suspensions. Pressure pulse equalizers should be installed between the injection pumps and the injection lines which reduce the pressure pulses to max 10% of the injection pressure. A manometer should allow visual control of the exit pressure pulses at the equalizer.

Electronic sensors for pressure and flow rate registration should be provided at the standpipe of any injected hole. The sensors should be connected to numerical displays and recording units in the grouting control room. The standpipe is provided with the necessary valves and connections for the packer, the return line if any, shutting the packer when grouting is completed, the manometer and the bleeding outlet.

For injecting very thick suspensions, and mortar if required, helical screw pumps of the needed capacity should be available. The capacity of the storage bins, hoppers and stock piles should cover twice the average weekly grouting capacity needed for the completion of the Project by the contracted completion date.

III Grouting materials

Cement

The cement quality should be in compliance with the stated local, or the required Standard. The required fines can be achieved, if the specified sort of cement is not obtainable on the market, by special processing in the mixing plant and cycloning out the grains larger than specified, provided production tests prove the feasibility of the process and the Engineer approves it.

Bentonite

Bentonite activated and pulverized of certified quality with liquid limit more than 400%, containing more than 90% particles finer than 0.08 mm, should be used. Exceptionally, mined raw bentonite mixed and activated in a slurry at the site can be approved by the Engineer. The quality of the slurry should be checked regularly.

Clay

Clay of specified plastic and liquid limits containing more than 90% particles finer than 0.08 mm should be used. It can be supplied from an approved borrow area which contains no less than 120% of the quantity required for the Project. It can be processed in the weight and batching plant, dry and pulverized, or as a slurry as approved by the Engineer. Pulverized dry clay is weight batched, if a slurry is used it is batched by volume. The quality of the pulverized clay or of the slurry should be checked regularly.

Sand

Sand used in grout suspensions and pumped mortar should meet the specified grain size distribution curves and it should be free of harmful minerals. It should be weight batched under the consideration of its moisture content.

Additives

Plasticities, air entraining agents and other additives for grout suspensions are added by weight if pulverized, by volume if liquid, all in the prescribed proportions.

Water

Water for grouting suspensions should be clean, without solid sediments or other harmful substances like sewage wastes, oils, acids, alkalies or organic substances. The Contractor should provide certificates of the water quality for the approval by the Engineer.

IV Drilling works

All holes for grouting the curtain can be drilled with rotary drilling rigs, which must be used for drilling all exploration and check holes. Injection holes can be drilled with rotary or percussion rigs with noncoring bits. Only clean water must be used for flushing during the drilling, but exceptionally flushing with air can be approved by the Engineer. Sufficient energy, flushing water and/or air supply must be made available at the working sites.

The ground water level, as well as loss of flushing water during the drilling, must be registered in all the drilled holes.

The inclination of the hole axis will be measured in the drilled holes selected by the Engineer, if the required standard is exceeded, the drilling unit price may be penalized.

The Engineer will establish the format of a protocol containing relevant data for all the drilled holes, which must be currently submitted to the Engineer.

Cores extracted from exploration holes, check holes and any of the injection holes will be stored in standardized cases for inspection.

All exploration holes will be cased if necessary, the upper end of the cased piezometer holes will be set in a concrete block 60 cm high, 30/30 cm in plan, and provided with a locked cap. A collar 50 cm long will be set in mortar at the top of every injection hole.

V Permeability testing

Permeability testing should be carried out intermittently with drilling from top to bottom. The length of the test section is normally 5.0 m, but the Engineer may instruct to test shorter sections of the hole. The section shall be thoroughly flushed after drilling with water until clear water emerges from the hole.

A mechanical, or inflatable packer is fixed on the top of the test section, as the quality of the rock may require. The mechanical packer should be at least 50 cm long. A standpipe connects the water supply line to the packer pipe.

The flow of water into the test section is measured at a constant pressure p_m (bar) during two 5 min intervals after it has stabilized. If the two readings on the flow meter differ by more than 10% one from the other a, third reading is taken and the average is recorded. In addition, the elevation of the manometer H_m, the ground water level H_g and the level of the packer H_p, the length L of the section, all in m, are noted. The hydraulic head loss p_h (bar) in the line from manometer to the packer for the relevant flow rate is determined from the gauging chart of the system. With these data the pressure p (bar) is evaluated in the test section:

$p = p_m - p_h + 0.1\ (H_m - H_g)/\ \rho_w$ if the packer is above the ground water, and
$p = p_m - p_h + 0.1\ (H_m - H_p)/\ \rho_w$ if it is below the ground water.

From the total flow, the flow Q in lit per 5 min, the flow rate $q = Q/5\ L$ (l/min m) is computed. The flow Q is measured in consecutive pressure stages of 1.5; 3.0; 5.0; 3.0; 1.5 bar. The results are presented in a p/q diagram. Where the tangent at the origin of the p/q line intersects the ordinate of p=10 bar the flow rate q_{10} is obtained, and the permeability number is $N = q_{10}$ LU. The pressure test results and the p/q diagrams are attached to the bore hole log.

VI Grouting

The scope of curtain grouting is to reduce the natural, occasionally erratically varying, permeability of the foundation rock to the required standard shown in the contract drawings. The system of injecting the holes from top to bottom or from bottom up, and the necessary details, as described in Item 6.31 should be explained so that the Contractor has a clear picture of the required organization of works, in order to prepare plans for the equipment, plant and infrastructure needed for the completion of works within the set term.

The split spacing method of injection should be described and the need to determine the final spacing on a test grouting plot should be stated. The accepted depth difference between simultaneously injected stages in adjacent holes of different sets should be stated. The criteria upon which the final depth of any set of holes will be determined from the grout consumption, or the permeability measured in the previous set of holes, as described in Section 4.1, should be stated. It is a good practice to inject any hole to the depth where the two last stages have reached the required initial permeability criterion.

The composition of the grout should match the characteristics of the joints and fissures based on the results from the grouting test plot. It is sufficient to provide the water mixing ratios of suspensions of 3:1, 2:1, 1:1 and 0.7:1, one thixotropic suspension composition and one of mortar in cases when very wide fissures or caverns should be grouted.

The injection pressure and saturation criteria are set from the results obtained on the grouting test plot. The injection pressure read on the standpipe manometer in any hole must be connected so, that the specified injection pressure is attained in the injected section of the hole.

If the saturation criteria in a stage are not satisfied after injecting 1,500 kg/m of grout compound, the injection is interrupted, the packer is pulled out and the stage is flushed with water. Injection is repeated after a specified period of 6, 12 or 24 hours. If the saturation criteria are not satisfied again the Engineer will decide:

- to reduce the saturation criteria (pressure and grout flow rate);
- to resume injection after a specified time;

- to resume injection after one hour with thixotropic grout or with mortar.

VII Payment of works

The payment of completed works should be itemized so that the financial result is convenient for the Owner. This is best achieved when the items of the bill of quantities do not contain risks that can not be reasonably estimated by the contract parties. There are two possible alternatives: A - to provide a special payment for mobilizing the site, to provide and install the necessary infrastructure and to clear up after the completion of works, and B - to include all costs of mobilization, infrastructure and clear up in the unit prices.

Drilling of exploration holes should be paid by the linear meter of hole including continuous coring. When less than 90% of the core length is obtained, the unit drilling price should be penalized. Production drilling unit prices for injection holes should be staged by the depth. A separate item should be provided for the setup of drilling rigs in exploration and production drilling.

In both alternatives, the injection can be split into the following items:

- number of packer setups for the injection of every stage, injection of any stage per linear meter, supply, storing and processing of all materials for the preparation and delivery of grout suspensions to the injection pumps per ton of cement, bentonite, clay, per kg of injected additives, per m^3 of injected sand and other fillers;
- all injection work including the supply, storing and processing of materials for the preparation and the injection of the suspension, per ton of injected compound as described in the Technical Specification.

The supply of pipes and the installation of piezometers in exploration holes, in compliance with the contract drawings and Technical Specifications, should be paid for separately.

6.5. Observation of grout curtain performance

For the observation of the performance of a grout curtain Casagrande type piezometers and/or water pressure cells should be installed along the curtain, upstream and downstream of the curtain on characteristic geologic, lithostratigraphic and hydrogeologic points. All piezometers installed during the exploration and the study phase of the project should be included in the observation network. The development of the ground water elevations as observed in the piezometers during first the impounding of the reservoir and during its operation, will provide data the elaboration of which permits to draw a picture of the hydrodynamics of the ground water flow around and through the grout curtain. A full hydrodynamic observation also includes all the existing sources and especially the appearance of new ones along the river bed downstream of the curtain. The comparison of the discharge of such sources before and after impounding at full reservoir gives a numerical measure of the efficiency of the grout curtain.

The uplift pressure is measured under the foundation of massive gravity dams in order to determine the effect of the curtain and of the installed foundation drainage on the uplift.

The elaboration of such observation data over a period of several years will establish the reliability and the efficiency of the injected curtain and disclose defective regions which might require any remedial works in order to maintain the safety of the dam foundation.

Experience with many injected grout curtains in very different natural grounds have shown that they are permanent under natural conditions if the basic rules of good design and injection practice are observed. The efficiency of the curtain can be impaired under some unfavorable circumstances, such as:

- leaching of the cement from the grout by the action of aggressive water;
- solution of the rock by aggressive ground water;
- use of injected grout which is not resistant to hydraulic erosion;

The danger is greater if the permeability standard to which the curtain was injected is not adequate for the natural conditions. Most cements may be leached by percolation of soft water. In this case it is convenient to reduce the cement content of the grouted compound to the minimum required for gaining sufficient strength, and the rest is replaced by clay. Uniform tight injection is the best guarantee for durability of the curtain in such conditions.

Curtains in soluble rocks should be sufficiently wide so that the pressure gradient H/d through the grouted zone is less than 3.0. They must be injected to a very low permeability standard, with a small hole spacing so that any potential leakage path is intercepted and plugged by the grout.

The resistance of the injected compound to erosion is in a direct relation to its unconfined strength, but injected clay, from which water is squeezed and filtered out, gains already during the injection and saturation stages sufficient cohesive strength to withstand erosion. A high cement content in the grout is required to gain the needed strength of the injected grout when very wide fissures are present. As shown in Item 6.32, very low strength is needed to safely plug even relatively wide fissures which can be achieved by grout containing a low proportion of cement.

The influence of blasting on the integrity of the grouted curtain is often discussed on many construction sites. If the distance of the blast from the curtain is such that no destruction of the rock can occur, only elastic waves reach the region of the curtain and these could not be expected to cause any damage. Verfel (1983) has presented two cases where the influence of blasts in the proximity of the curtain was measured. On the Moravka Dam in Czechoslovakia in 1964 it was demonstrated that the permeability of the injected ground increased when the blast was closer than 3 m to the grouted zone. On the Šance Dam, the rock on the foundation surface was blasted with charges of 40 to 80 kg of explosives at a time. After concreting the foundation, it was shown by water pressure tests that the permeability of the grouted curtain was not increased.

6.6 Examples of grout curtains

The experience on completed grout curtains can contribute much to the solution of complex design and construction problems on new curtains in similar geologic and geotectonic conditions. It is therefore of interest to present conditions, the methods of exploration and design solutions of completed grout curtains, as well as the characteristic features such as the hole depth, the density, the grout take and the literature available for a detailed study of the case records. A few characteristic achievements will be presented, arranged into three groups according to the geologic formations on dams in Yugoslavia and several other countries:

- curtains in Karst regions;
- curtains in other geologic formations;
- curtains in alluvial deposits;

followed by a review of the dimensions and characteristic features of a number of grout curtains.

6.61 Grout curtains in Karst regions

Grout curtain of the Peruča Dam

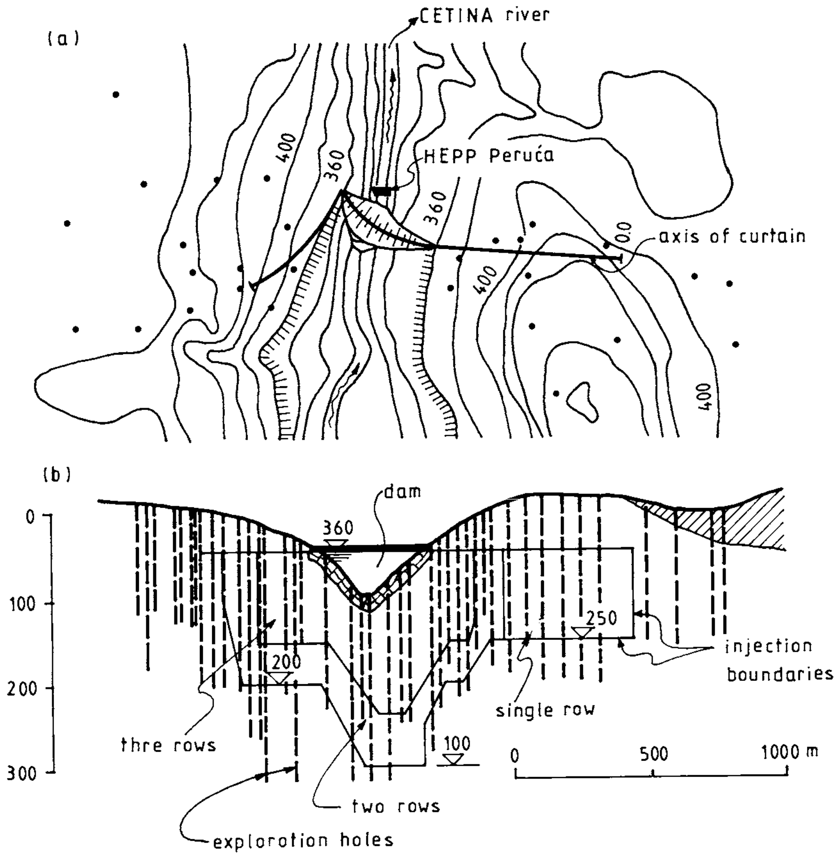


Fig. 6.11 Peruča Dam, (a) plan of the dam and grout curtain, (b) section of the dam and grout curtain and exploration holes

The Peruča Dam on the river Cetina (Yugoslavia) is a 60 m high embankment dam with a central clay core founded on very fractured and highly karstified Cretaceous limestone beds. It forms a reservoir of some 500 millions m^3. Considering the high energy potential of the Cetina River on a drop of about 360 m to the level of the Adriatic Sea, and the value of the stored water in a basin formed of very pervious karstified limestone, it was important to reduce possible seepage losses to a minimum by injecting an extensive and deep grout curtain under the dam and on both wings. Detailed geologic studies, supported by some 20,000 m of exploration drilling with water pressure testing, have demonstrated that the limestone beds on both banks of the basin were underlain by impervious rock formations and that seepage losses would occur only through the pervious limestone beds under the dam and through the abutments. The plan and longitudinal section of the dam and grout curtain are shown in Fig. 6.11 where the location and depth the of numerous exploration holes is also presented.

Three grouting test plots were injected in order to determine the technical parameters of the curtain, as shown in Fig. 6.12.

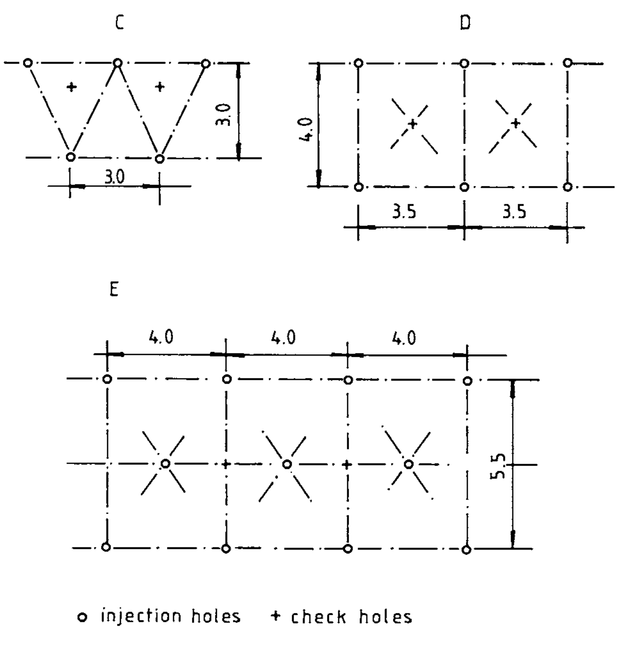


Fig. 6.12 Arrangement of injection and check holes in test grouting plots C, D and E at Peruča Dam

Two rows of injection holes were grouted in test plots C and D, and two check holes were pressure tested in each test plot. Both test plots were injected with stabilized cement suspensions containing 5 % to 8 % bentonite. In test plot E the injection holes were arranged in three rows and two check holes were water pressure tested. Suspensions of 25 % cement and 75 % local plastic clay stabilized with some bentonite were injected. The characteristic data for all three test plots are shown in Table 6.2, from which the big advantage of using suspensions containing a large portion of clay over pure cement suspensions is evident. In test plot E, which exhibited the highest primary permeability of the rock, it was reduced to 3% of the original value in the grouted zone, while in test plots C and D it was 8% and 11% respectively of the original lower value. All parameters presented in Table 6.2. are more favorable for the test plot E, which was grouted with a cement/clay compound.

TABLE 6.2.

Grout test plot results in karstified limestone at Peruča Dam foundation

Description	Units	Grout test plot		
		C	D	E
Total length of grouted holes	m	360	400	560
Depth of test plot	m	72	66	50
Specific length of holes	m/m^3	0.67	0.36	0.21
Average permeability of all holes	LU	17	13	38
Grout composition	compounds	C:B:W	C:B:W	C:CL:B:W
Specific grout take	kg/m	440	38	45
Same	kg/m^3	294	135	137
Cement consumption	kg/m^3	270	130	34
Specific grout take per unit	kg/m^3 LU	17.4	10.4	3.4
permeability	kg/m LU	26.2	30.4	16.9
Average permeability in check holes	LU	1.34	1.38	1.22
Ratio between permeabilities of ungrouted and grouted rock	-	0.08	0.11	0.03

The properties of the cement/clay grout compound for this project were verified in a very comprehensive laboratory investigation program, in which the strength, erosion resistance, volume stability and durability of the injected grout was tested and documented. Based on these results it was decided to design and implement the curtain shown in Fig. 6.11. The length of the curtain - river, abutment sections and wing curtains - amounts to 1,600 m, the greatest depth below the river level is 200 m. The total length of drilling is 180,000 m, out of which 141,000 m were injected. The grout consumption totals 49,000 t of cement, clay, bentonite and sand. Thixotropic suspensions and mixes with sand were used to reach saturation of hole sections with excessive grout take, very large fissures and caverns, and sections from which communication to sources or to the surface was established. Grouting works on this large project lasted three years (1953-1956), and two injection companies from Zagreb were engaged for grouting.

The results from water pressure tests on a completed section of the grout curtain are presented in Fig. 6.13 which shows the permeability before grouting of all grout hole stages in the upstream row of holes, of the middle row of holes after grouting the external rows, and in check holes after grouting the middle row.

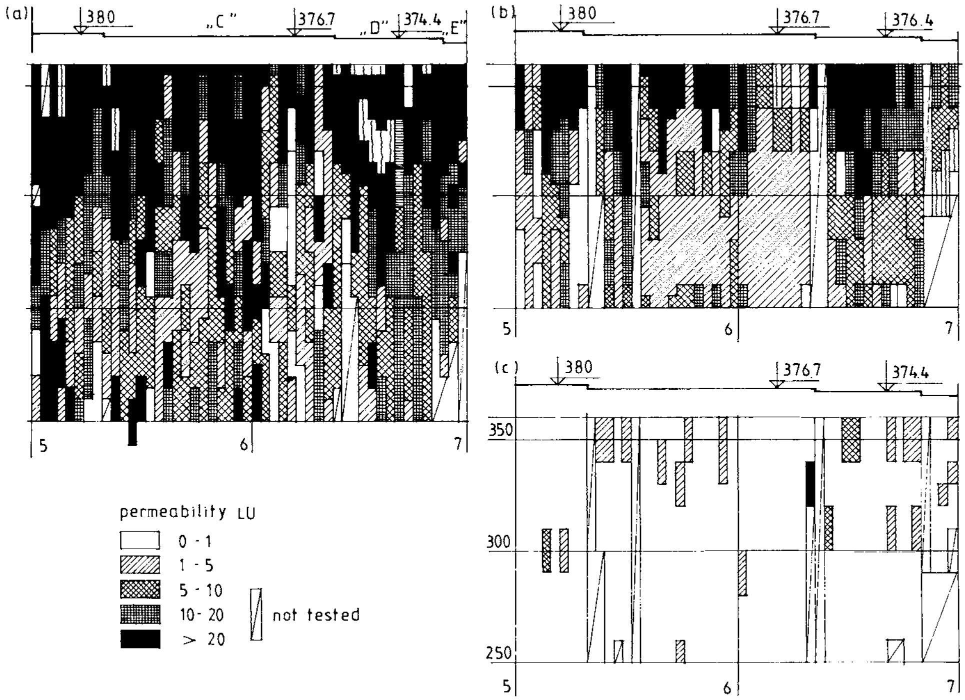


Fig. 6.13 Results from water pressure tests in section 5 - 7 of the Peruča dam grout curtain, (a) upstream row before grouting, (b) middle row before grouting, (c) middle row in check holes after grouting

In Fig. 6.14 the grout take in the upstream row is presented. The injection pressure was up to 40 bar in the exterior rows, up to 60 bar in the middle row to the depth of 100 m, and 65 bar to the depth of 200 m. Details are shown in Fig. 4.22. When the grout take in a grouted section exceeded 1,000 kg/m, grouting was stopped and later on continued with a thixotropic compound, in some cases with sand added. Grouting in the river section was done from the grouting gallery constructed in the clay core foundation trench.

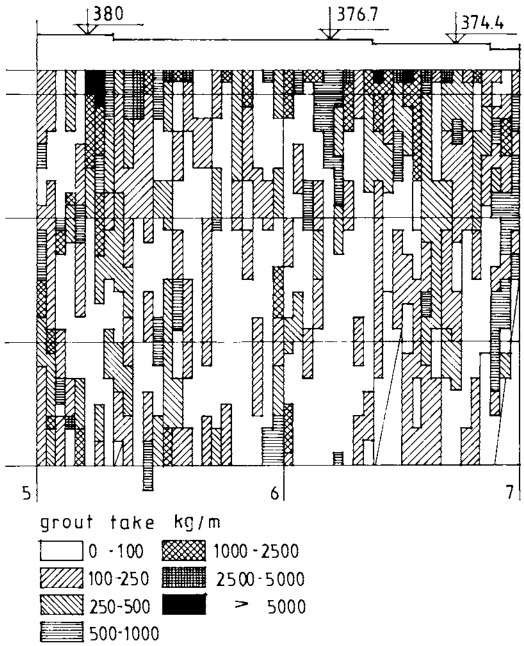


Fig. 6.14 Specific grout take in the exterior row of holes of section 5-7 of the Peruča Dam grout curtain

The effect of the grout curtain on the seepage flow through the curtain and around the wings is controlled by a number of piezometer holes upstream and downstream of the curtain. Also, the discharge of springs along the Cetina River, which existed downstream before the dam construction was measured. From the observations during a period of many years, it is estimated that the seepage loss from the reservoir at full storage level is about 1,000 l/s, which is less than 1.5% of the average river flow (Pavlin, 1961). This discharge has remained constant over the years, which evidently proves that the injected cement/clay compound is fully stable and durable. At that time the work on the grout curtain of the Peruča Dam, carried out in extremely unfavorable karstified foundation rock served as a full scale field laboratory and a school in which this kind of grouting was perfected. It became a model for many other dams which were constructed later on in the Dinaric karst of Yugoslavia. It was a major success for Yugoslav scientists, engineers and grouting specialists.

Grout curtain of the Sklope Dam

The unpredictability of karst limestone formations manifested itself fully during the injection of the grout curtain for the Sklope Dam. It is a 75 m high embankment dam with a slightly inclined clay core and a reservoir of 140 million m^3 in volume. The dam foundation consists of stratified and broken limestone beds of the Oligo-Miocen formation.The dam site is in a narrow canyon formed

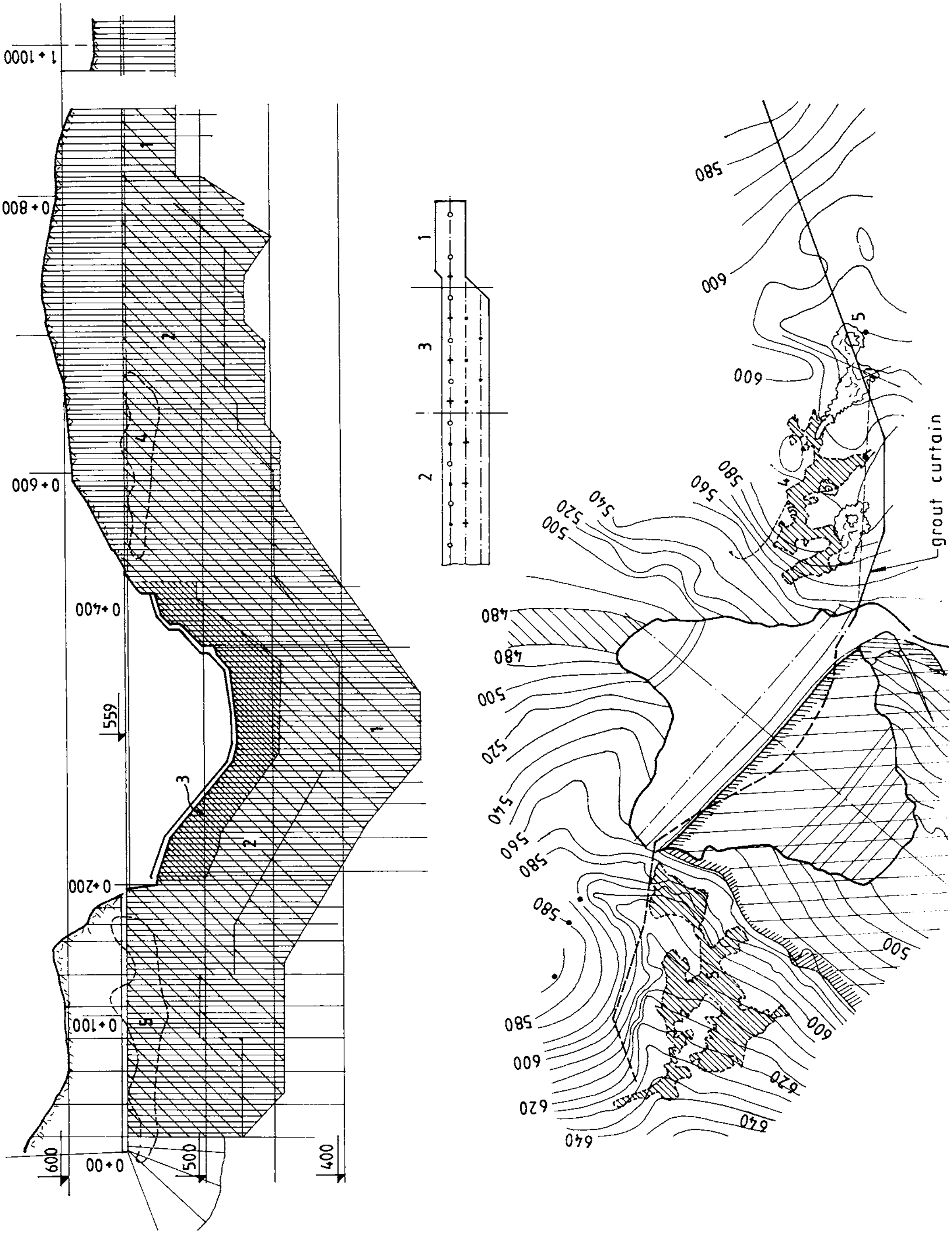


Fig. 6.15 Plan and longitudinal section of the grout curtain of Sklope Dam, disposition of holes; 1 single row curtain, 2 double row curtain, 3 three row curtain, 4 cave in the right abutment, 5 cave in left the abutment

by the erosion of a barrier which,in an undefined geologic time,dammed a lake. The limestone barrier was corroded and eroded by percolating water until it was fully destroyed. The rock in the steep abutments was fractured and cavernous, and generally very permeable.

Detailed geologic studies, based on extensive exploration drilling, were carried out in order to define the most convenient location for the dam and the 1,100 m long, and up to 150 m deep, grout curtain. The plan and the longitudinal section of the dam and the grout curtain are shown in Fig. 6.15,where sections of the curtain with one, two and three rows of injected holes are indicated. Great care was taken to thoroughly inject the contact of the core foundation of the dam core where the rock was very fractured. An injection gallery was constructed, from which a fence of subhorizontal injection holes was injected, as shown in Fig. 6.6.

When grouting started at the right dam abutment, the first surprise was encountered. The first injection hole struck into a 20 m deep void and further exploration led to the discovery of a large cave, shown in Fig. 6.15, which had a hidden access from the canyon wall. The axis of the curtain was moved upstream to bypass the upstream end of the cave, which remained downstream of the grout curtain.

In order to economize with idle drilling,the left wing curtain was grouted from a tunnel. When grouting without any difficulty was already close to the end of the curtain, the next surprise came along. In a deep section of an injection hole 3,000 t of cement was injected without reaching saturation. Only cement was injected, because the design engineer was not convinced in the durability of cement/clay compounds. When a detailed exploration was undertaken, a very large cave, as shown in Fig. 6.15 was discovered. It was connected through a crevice to the injection tunnel. A detailed geologic survey provided the shape and the extent of the cave, as shown in the figure. The axis of the grout curtain cuts the cave close to its end.When a speleologist team inspected the cave, a large volume of injected soft cement slurry was discovered at the far end of the cave.

In order to prevent large seepage losses, when the cave is flooded by the impounded water a concrete wall was constructed which sealed the end of the cave, and the very pervious stages of the fanning holes at the end of the curtain were injected with mortar pumped to full saturation with a helical screw pump .

Investigations after impounding the reservoir have shown that seepage losses at reservoir full amount to about 1,000 l/s. This can be regarded as a very good result when the natural high permeability of the rock is considered.

Grout curtains for the Buško Blato reservoir

In the karst plain Buško Blato, at the elevation of 700 m above the sea level, which was yearly flooded during the wet season, a reservoir was created by building a 3,000 m long embankment dam which impounds water to an elevation of 716.4 m. The plain was flooded every spring until some time in summer by the Ričina River at the eastern side of the plane and some smaller springs and estavelles at its south-eastern boundary. The river sinks through ponors at the south-western and western boundaries of the plain. The capacity of the ponors is limited, so that the spring flood water is retained on the plain. The water from the ponors emerges from springs along the Cetina River on the lower karst plane. The region of the Buško Blato plain consists of limestone of the Cretaceous age overlain at the bottom of the plain by tertiary sediments of limestone conglomerates and marl. The formations are folded in the NW-SE direction and cut by minor and greater faults, the most important of which is on the line of the Liskovača and Prozdrikoza ponors and Podgradina (Fig. 6.16). Quaternary 10-40 m tick sediments consisting of clay and sandy clay cover the bottom of the plane.

Exploration drillings have shown that the rocks around the plain are highly fractured with many karst features and very permeable, the depth of karstification reaching 70-100 m below the level of the plain. Piezometer observations have shown that the ground water level is near the surface of the plain in the eastern part, and about 30 m deep at the southwestern part during the wet season and near the base of karstification during the dry season. The maximum capacity of all ponors at the flood level amounts to about 35 m^3/s out of which about 20 m^3/s is concentrated at the Liskovača and Stara Mlinica ponors. The average yearly inflow into the plain is 11 m^3/s.

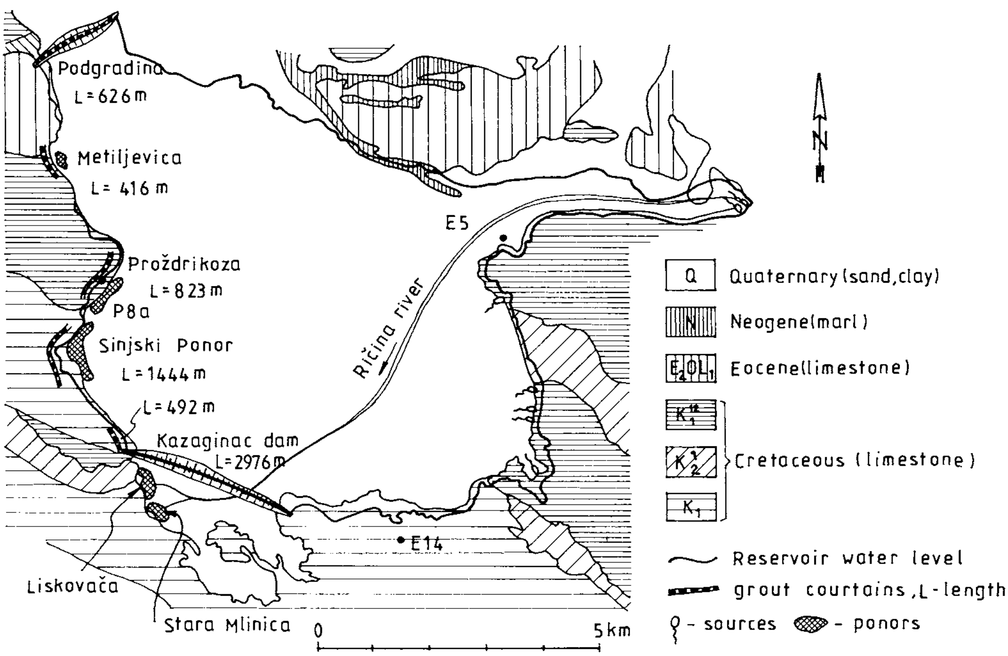


Fig. 6.16 Situation and geologic formations of the Buško Blato area

Extensive studies were carried out in order to establish the best means of reducing seepage losses from the reservoir. The results of the studies indicated that the zones of the ponors Metiljevica, Prozdrikoza and Sinjski ponor could be sealed with grout curtains to the injected depth of about 10 m below the karstified zone. The Liskovača and Stara Mlinica ponors had to be isolated from the reservoir by the 3,000 m long Kazaginac Dam. Since the northwestern end of the plain is below the maximum storage level, the 600 m long Podgradina Dam was built there. The length and the main design parameters of the grout curtain of the Buško Blato reservoir are shown in Table 6.3.

TABLE 6.3

Grout curtains at the perimeter and under the dams of the Buško Blato reservoir and planned quantities

Grout curtain	length m	injected holes m	injected compound t	specific consumption t/m
Kazaginac	3,136	94,800	33,200	0.350
Sinjski Ponor	1,470	18,500	13,000	0.703
Prozdrikoza	640	19,400	3,900	0.201
Metiljevica	416	7,000	1,800	0.257
Podgradina	544	5,700	600	0.105
Total	6,206	145,400	52,500	0.361

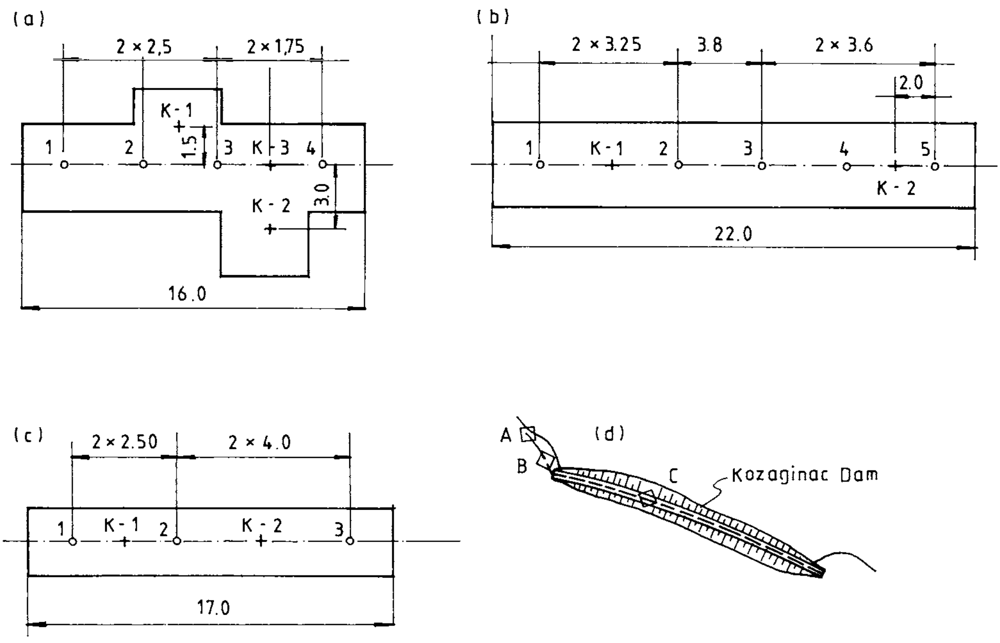


Fig. 6.17 Buško Blato grouting test plots A, B and C, (a) plot A, (b) plot B, (c) plot C, (d) location of test plots

Three grouting test plots A, B and C were injected in order to establish the spacing of holes and the best grout compound composition, the grouting procedure and saturation criteria. The plan and the hole arrangement of injection and check holes are shown in Fig. 6.17. Test plots A and B were situated on the big fault zone near the Liskovača ponor representing the conditions of the grout curtain on the western boundary of the plain, and the test plot C was on the axis of the Kazaginac Dam where the permeability of the rock is lower.

The composition of the grout suspensions compounds was:

- Type G 0.25C:0.75CL+2.5% sodium carbonate on the clay portion,
- Type E 0.25C:0.70CL:0.05B and 0.25C:0.55CL:0.20B both with 2.5% sodium carbonate of clay and bentonite weight. Injection pressure was from 5 to 30 bar depending on the depth of the section. An overview of the grout take in the test plots is given in Table 6.4.

TABLE 6.4

Results of grouting test plots

Description	Grouting test plot					
	A		B		C	
Depth, m	4-89	89-160	4-89	89-160	20-70	70-140
Average permeability, LU	185	9	165	32	-	-
Grout take, kg/m	750	38	1290	44	408	137
Permeability in check holes, LU	34	0.2	13	1.5	0.5	0.2
Grout take in check holes, kg/m	283	0	-	-	-	-

From the study of the grouting test plot results the following parameters for the grout curtains were given:

- split spacing one single line of primary holes at 16 m, secondaries at 8 m and tertiaries at 4 m spacing, exceptionally quaternary holes at 2 m;
- the grout curtain was divided in 64 m long sectors;
- every fourth primary at 64 m spacing was drilled first as an exploratory hole, cored and water pressure tested;
- then every second primary at 16 m spacing was drilled and water pressure tested;
- the rest of the primaries, and all other holes were drilled and grouted to the required depth without water pressure testing.

The depth of the curtain was defined from the results of the cored and water pressure tested holes for every sector of the curtain. From the data obtained in the grouting test plots, the amount of work for the grout curtains around the Buško Blato reservoir shown in Table 6.3 was anticipated.

The central batching and injection station was near the right abutment of the Kazaginac Dam. From there, injection holes to a distance of 400 m were directly grouted. The suspensions for grouting farther distant sectors were premixed in the central batching station and pumped to auxiliary agitators at the injection sites, from which the final suspension was mixed and pumped into the injection holes.

Large caverns were discovered in the Sinjski Ponor grout curtain. Adits were excavated into the caverns and they were plugged with concrete walls in the axis of the curtain.

The amount of work effectively carried out on all grout curtains is shown in Table 6.5.

TABLE 6.5

Extent of work carried out on Buško Blato grout curtains

Grout curtain	length	injected				
		holes		compound		
	m	m	m/m^2	t	t/m	t/m^2
Kazaginac	3,468	89,604	0.320	25,604	0.281	0.091
Sinjski Ponor	1,444	31,621	0.250	17,602	0.556	0.139
Prozdrikoza	823	28,711	0.506	7,100	0.247	0.125
Metiljevica	416	7,062	0.267	3,253	0.460	0.123
Podgradina	626	7,498	0.154	486	0.065	0.010
Total	6,777	164,496		54,045		

In addition some 40,000 m of idle drilling in overburden was carried out. By comparison with the planned amount of work in Table 6.3, it is seen that the length of the curtains was 9% and the length of production drilling 17% more than planned; the grout compound consumed only 2% more than planned.

On the longitudinal section of the Prozdrikoza grout curtain in Fig. 6.18, the specific grout take within the shown intervals is also presented. It is characteristic that the take in many places was high above the elevation 675, and generally less that 100 kg/m below that elevation.

The average grout consumption per sets of G,R, primary to tertiary holes is presented in Fig. 6.19. The grout take was highest in the G and R holes, and in the primaries it dropped to 1/3 of the G holes.

The effect of the curtains on the percolation from the reservoir was studied for a long time. Ground water levels were measured in piezometers in front of the grout curtain on the western rim of the reservoir, P7 at Sinjski Ponor and P 8a at the Prozdrikoza curtains, as well as at the southern and

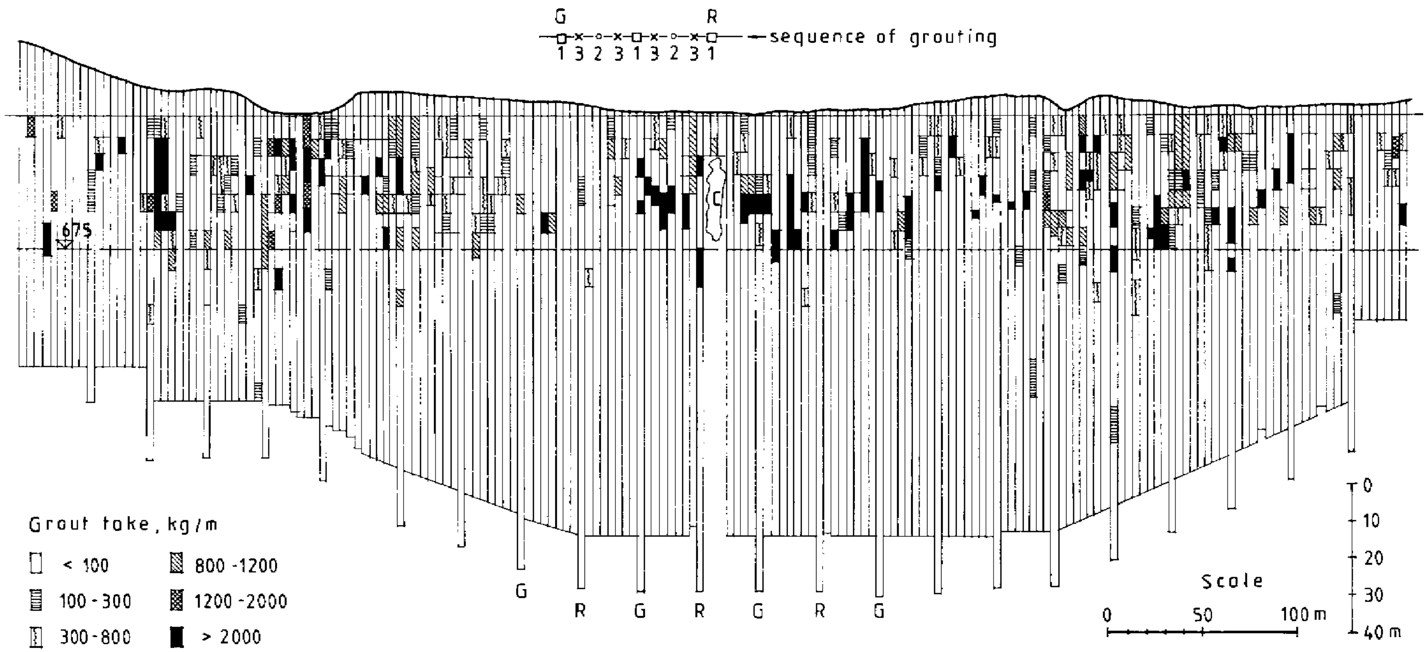


Fig. 6.18 Longitudinal section of the Prodrikoza grout curtain and the grout take in sections of injected holes, G cored, R water pressure tested holes, 1,2,3 primary, secondary, tertiary sets of holes

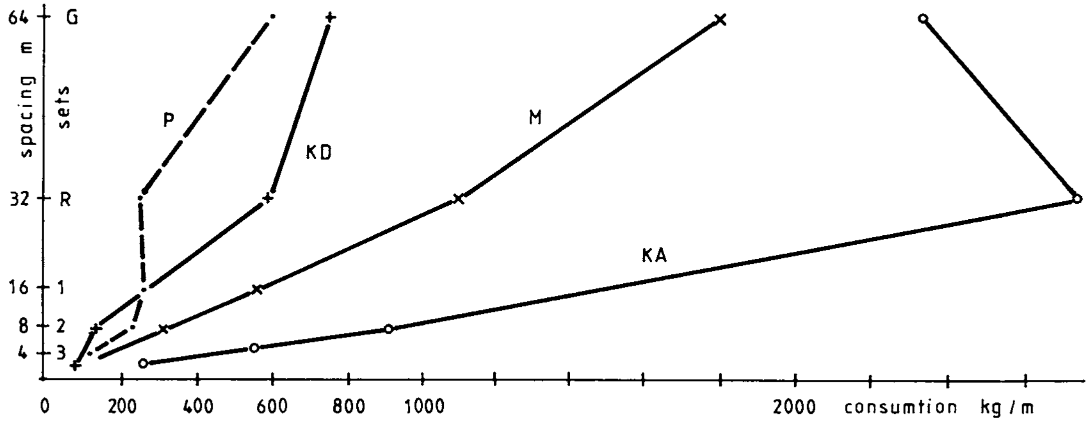


Fig. 6.19 Grout consumption in kg/m by sets of holes of the Buško Blato grout curtains, P Podgradina, KD Kazaginac Dam, KA Kazaginac abutments, M Metiljevica

eastern sides of the reservoir in the piezometers E14, E7, and E5. The results from readings in piezometers E5, E14 and P8a are shown in Fig. 6.20. It is evident that the registered ground water levels after the completion of the curtains are much higher than before. The lowest ground water levels

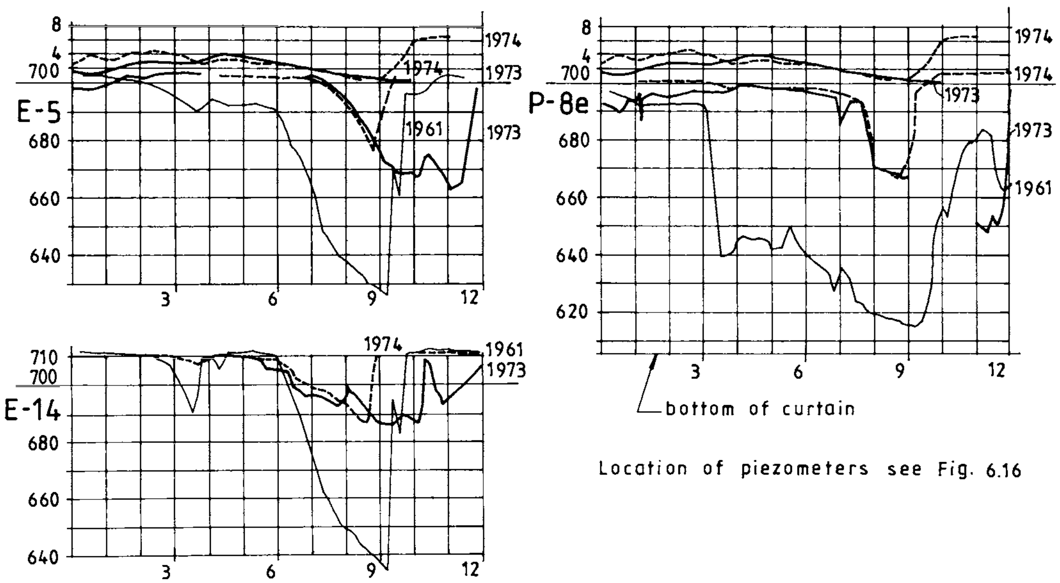


Fig. 6.20 Ground water levels in piezometers E5, E14 and P8a, and the corresponding reservoir levels in 1961 before, and in 1973 and 1974 after injecting the grout curtains and impounding the lake (levels between 702 and 708)

at the end of summer are more than 50 m higher in all observed piezometers. A percolation loss between 2 and 6 m^3/s is obtained from the balance of inflow and consumption of the stored water. From the observation and measurements of the discharge of sources on the lower karst horizon in the Cetina plain, it was determined that the discharge was reduced during the high water periods and slightly increased during the low water periods.

Left reservoir rim curtain at Keban Dam (Turkey)

The Euphrates River was dammed by constructing the 100 m high embankment and the concrete Keban Dam near Keban Village, for the electric power production. The dam is constructed on a ridge through which the Euphrates further on cut its course. It consists of metamorphic marble and limestone with beds of dolomite and calcshist of Paleozoic age known as the Keban Formation. The calcareous beds are fine grained, hard and slightly to heavily fractured and jointed, as the consequence of intensive tectonic activity. A few longitudinal and some perpendicular faults intersect the formations on the line of the dam axis. Along the rock defects, hydrothermal water easily circulated from the depth up to the surface, and a chemical solution occurred in the calcareous and dolomitic rocks, successively increasing its permeability. There is evidence indicating that upflowing hydrothermal water following some volcanic phases, was mainly responsible for the formation of extensive solution channels, openings consisting of even very large and deep caverns in the limestone and dolomite under the dam and on the left abutment upstream from the concrete gravity spillway and wing section of the dam. The dissolved calcium carbonate was deposited on the surface as travertine deposits, a few of which are located on the left abutment upstream from the gravity dam wing. The Keban Formation is overlain by the Upper Shist containing mica, sericite, talc and graphite. It is impermeable and it covers the permeable limestone in some parts of the reservoir and downstream of the dam axis. Some volcanic syenite dikes crop out on the right and left abutments downstream of the dam axis. The tectonics of the site is very complex. Strong tectonics have been active in several geologic periods, the last of them took place probably in the Tertiary. The results are many faults of large extent which cross the valley below the dam and a few faults of lower order perpendicular and oblique to its axis. These features make the Keban Dam foundation very complicated, especially regarding percolation losses and foundation stability.

The final extent of the geologic, tectonic and hydrogeologic setup, as well as their meaning for the dam foundation, were disclosed gradually during the dam construction, as the results from additional foundation works became available.

Extensive foundation treatment was necessary in such difficult natural conditions. The design provided for a deep cutoff consisting of an impervious curtain injected from five galleries, which is shown in the longitudinal section in Fig. 6.22. The total length of excavated galleries amounts to 11 km. During the excavation of the galleries some large fissures, open or filled with gauge, were discovered. They required a special treatment. The gauge was excavated and replaced with concrete, in some locations concrete diaphragms replaced grouting as shown in the longitudinal section of the cutoff in Fig. 6.22.

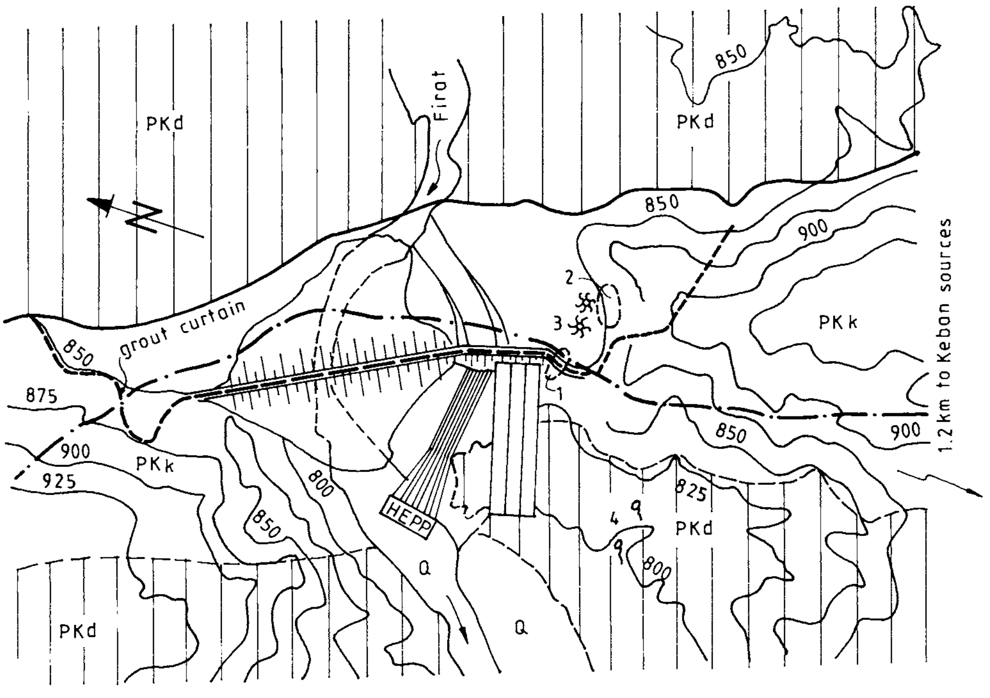


Fig. 6.21 Geological map of Keban Dam, Q Quaternary, P-Kk Paleozoic limestone, P-Kd Upper schist, 1 Crab Cavity, 2 Petek Cavity, 3 vortexes after impounding, 4 temporary sources

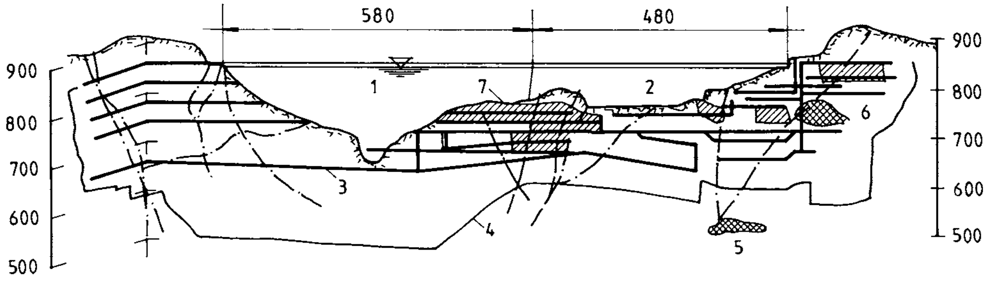


Fig. 6.22 Longitudinal section of impervious curtain in the Keban Dam foundation, 1 embankment dam, 2 concrete gravity section, 3 galleries and shafts, 4 boundaries of grout curtain, 5 Crab Cavity, 6 Petek Cavity, 7 concrete diaphragms

It was divided into three sections: the right wing section, the central curtain and the left wing section. Later on when the very permeable nature of the left abutment became evident, the 900 m long left reservoir rim curtain was implemented. Its longitudinal section is shown in Fig.6.23. This curtain cuts three fault zones, two of which required a very large grout take. In the first fault zone the grout take in the deepest injected hole sections was still very large, as shown in the figure.

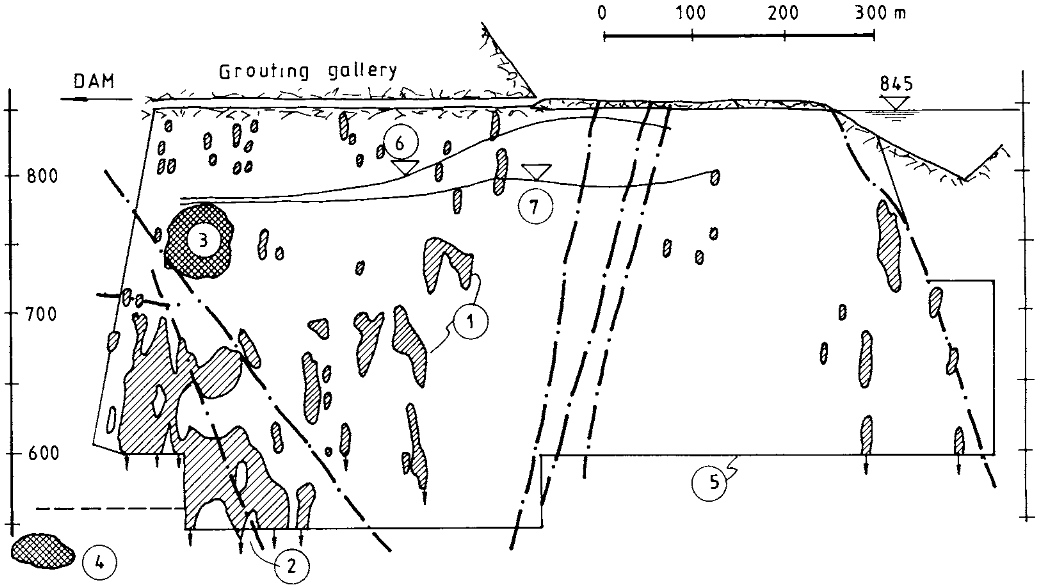


Fig. 6.23 Longitudinal section of the left rim grout curtain, 1 zones with grout take more than 2.5 t/m, 2 zones where the last injected hole stage took more than 2.5 t/m, 3 projection of Petek Cavity on the plane of curtain, 4 same for Crab Cavity, 5 lower boundary of the curtain, 6 ground water levels upstream, 7 downstream of the grout curtain

Uspstream and downstream of the curtain axis a number of piezometers were installed in which the ground water level near the bottom of the curtain was observed. Fig. 6.23 shows that along the section which did not reach the low permeability rock at the bottom, the ground water level in the upstream and downstream piezometers is low, 80 m lower than the lake level, while in the region of the reservoir rim, where the curtain is sufficiently deep, the upstream piezometer levels approach the reservoir level, and the head difference at the bottom of the curtain is more than 50 m.

As the dam construction approached its end in 1971, a large empty cave called the Crab Cavity was discovered below the gravity dam section shown in Figs. 6.21, 6.22 and 6.23. Its bottom was at elevation 500, deep under the grout curtain. It was 39 m high, its volume 105,000 m^3 filled with water which was not in direct contact with the river. The cave was filled through a large hole with 64,000 m^3 of tremie concrete.

Another large cave called the Petek Cavity, also shown in the mentioned figures, was discovered in 1975 when the reservoir level reached elevation 826.5 for the first time. Some vortexes appeared near the southern shore of the lake upstream of the left abutment gravity dam. Some of them were intensified in the spring of 1976 when the lake reached elevation 838.5. At this elevation the lake water eroded the ingress into what was then discovered to be a very large empty cave. Speleologic investigations carried out after the reservoir lake was lowered, indicated that the bottom of the cave was at the elevation 730, and that its volume was about 600,000 m^3. At the same time it was found that the discharge of the Keban Creek sources, about 2 km south of the dam, rapidly increased to 26 m^3/s, as shown in Fig. 6.24.

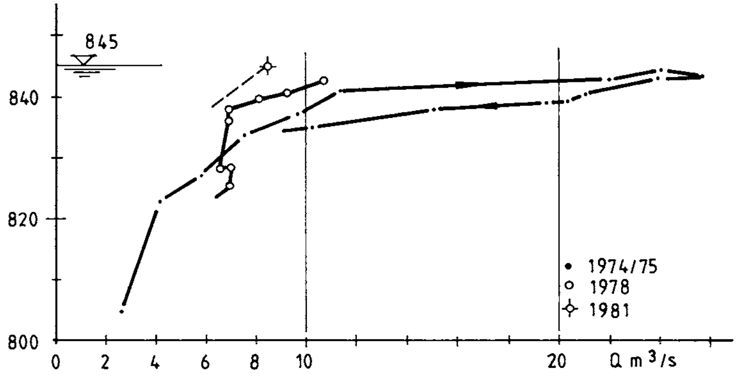


Fig. 6.24 Discharge from the Keban Creek sources as a function of the lake level

The Petek Cavity was completely filled with a gravelly material. The ground surface above it and at the reservoir shore on the whole zone of the vortexes was covered with concrete slabs in order to prevent further erosion of the flow paths by percolating water, as shown in Fig. 6.25.

Fig. 6.25 Left shore of the Keban reservoir lined with concrete slabs

The effect of these measures was a substantial reduction of the discharge of the Keban Creek sources, as shown in Fig. 6.24, from the maximum of 26 m^3/s in 1976 to about 9 m^3/s in 1980 at the full reservoir level. The discharge of the group of sources downstream of the dam axis (4 in Fig. 6.21) which in 1976 amounted to 1.15 m^3/s, was reduced after the treatment of the Petek Cavity to 0.20 m^3/s at the full reservoir level in 1980.

The total amount of work carried out on this very large and complex curtain is presented in Table 6.6.

TABLE 6.6.

Amount of work on the Keban Dam cutoff curtain

Cutoff section	length m	drill holes m	cement t	concrete m^3	cement kg/m
River valley	2,310	262,000	66,200	178,000	252
Left rim curtain	900	76,000	69,000	-	910
Crab Cavity	-	-	64,000 +	-	-
Petek Cavity	-	1,380	2,900	650,000*	2,100

Notes: + m^3 of concrete;
* m^3 of gravel fill.

It should be mentioned that a few tenths of m^3/s seepage losses from the Keban reservoir are negligible compared to the large mean river discharge and the huge volume of the reservoir.

Grout curtain of the Kao Laem Dam

The rockfill concrete faced Kao Laem Dam, 130 m high and about 1 km long, was built on the Quae Noi river about 300 km East of Bangkok in Thailand. The exploration works on this geologically very complicated site lasted six years, whereas the dam and the grouted cutoff were built from 1979 to 1984.

The area around the dam site consists of rocks of Permian age, Ordovician is on the left abutment and the valley section up to the main fault at the right abutment, where again rocks of Permian age crop out. Rocks of Ordovician age consist of schists and sandstones, and locally limestones with karstification phenomena are also present. Permian rocks consist of massive to thinly bedded very karstified limestones with large caverns and voids which are partly or completely filled with clay and sandy material. Karstified corrosion fissures reach down to 200 m below the dam foundation level.

The rocks are very fissured with many karst features especially at the reverse fault "Three Pagodas" which is probably of Pleistocene age.

Very extensive remedial measures were necessary to tighten the foundation rock considering the important height of the dam and the vast karstification of the foundation rock, and to provide a lasting protection against erosion of many wide and narrow fractures.

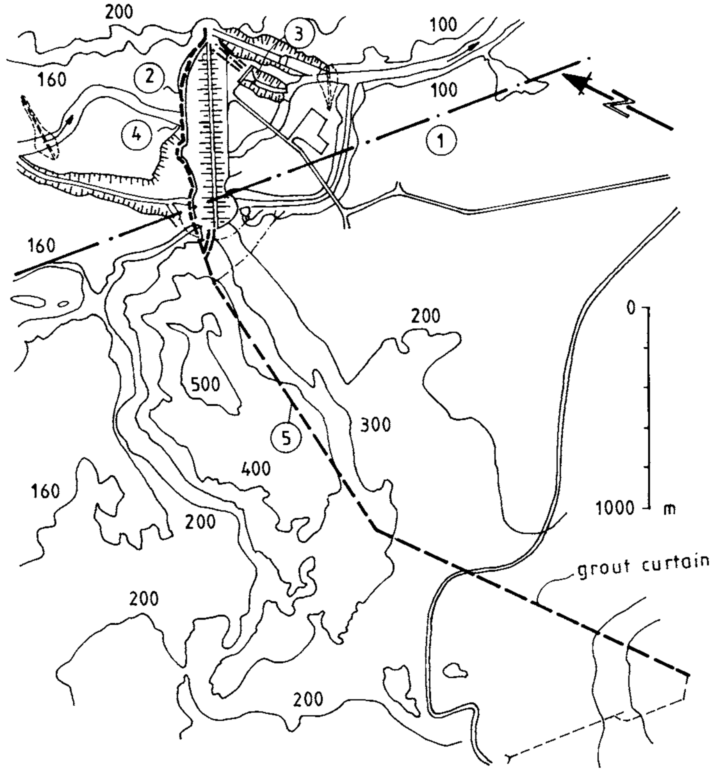


Fig. 6.26 Geology of the Kao Laem Dam site, 1 "Three Pagodas" fault, 2 rockfill dam, 3 spillway and HE powerhouse, 4 grout curtain in the valley section, 5 axis of grout curtain and galleries in the right abutment

The measures undertaken in the cutoff consist of:

- diaphragm walls of intersecting concrete piles 76 cm, at 62 cm distance in all karstified zones;
- grouting with mortar after previous sluicing with water under 10 bar pressure from two rows of holes at 1,0 m spacing;
- construction of concrete walls mined from the galleries in the zone of the main fault or excavated from the ground surface at the right abutment;
- grout curtain injected from the foundation plane under the dam concrete face and from galleries deep into the right abutment.

The locations, dimensions and some details of this project are shown in Fig. 6.27. Some fissures in the fractured rock zones in the right abutment were filled with clay and silty material in an amount

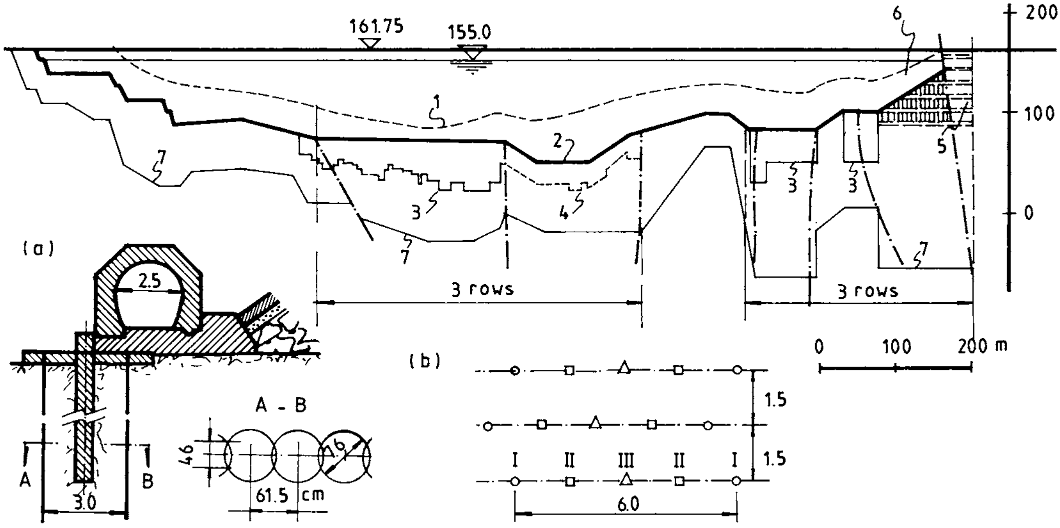


Fig. 6.27 Longitudinal section along foundation of concrete membrane of dam face, 1 ground surface, 2 bottom of foundation trench,3 diaphragm wall of intersecting concrete piles, 4 mortar injected into 165 mm holes previously flushed clean, 5 grouting galleries in right abutment, 6 mined wall between galleries, 7 bottom of grout curtain, (a) section through the perimetral gallery and diaphragm wall of intersecting piles, (b) arrangement of injection holes in curtain with three rows of holes

which was not suitable for grouting and not so important that excavation and construction of a concrete wall would be necessary. For such cases an original procedure was devised: Holes of 165 mm Dia spaced at 330 mm were drilled between two galleries. The holes were flushed through a special flushing head with water at 10 bar pressure, and the effect was controlled through a television camera. Then the holes were filled with plastic mortar and previbrated to fill all the voids. The method was applied up to 15,000 m of the cutoff, which is marked in Fig. 6.28.

The right wing curtain was injected from the excavation surface in the zone of the dam and from six galleries in the right abutment with a total length of 21 km. In the dam zone three rows of holes where injected to half the depth of the curtain. The external holes were injected first, the spacing of the primary holes was 6 m, and the injected sections of the holes were 10 to 20 m long. The injection pressure was 3-6 bar in the upper section, and 30 bar in the fourth and the following sections. Cement suspensions stabilized with 2% bentonite were used, the mixing ratio was 1C:2W, 1C:1W and 1C:0.7W. The last mix with 3% sodium silicate was added when the reach of injection had to be limited. In more karstified zones 100 kg of sand was added to 200 l of the thickest mix. The saturation criterion was 20 l of the thickest suspension during 15 min. The closure criterion for the last set of holes was fixed partly by the permeability measurements but dominantly by the cement take:

- to 30 m depth	40 kg/m or 5 LU,	-100-150 m depth	160 kg/m
- 30-100 m depth	80 kg/m or 2 - 10 LU,	-150 m depth	800 kg/m.

Up to 180 m down the hole was injected in sections, then the holes were drilled to the bottom of the curtain and injected in sections up the hole.

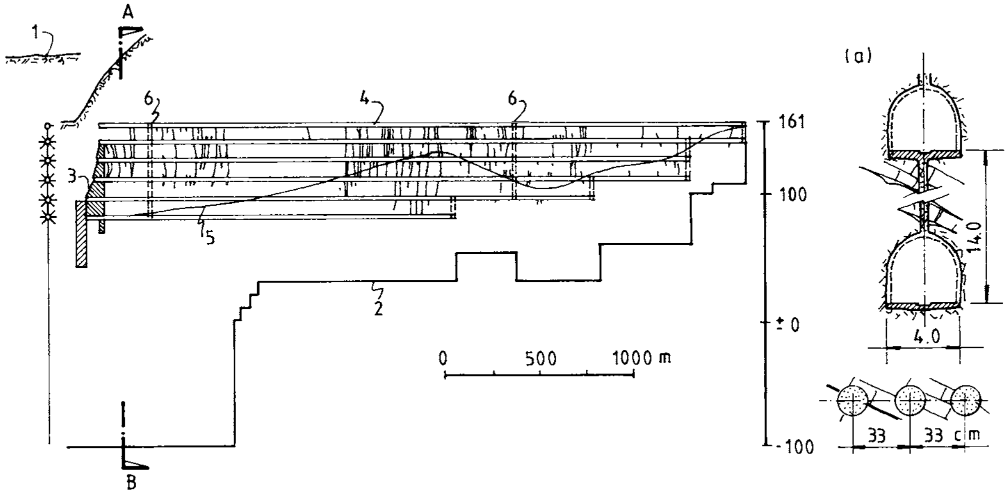


Fig. 6.28 Longitudinal section through the grout curtain in the right abutment, 1 section A-B through the injection galleries A-F, 2 lower boundary of the grout curtain, 3 mined concrete diaphragm wall, 4 zones injected with mortar through 165 mm Dia holes, 5 ground water level in dry season, 6 ventilation shafts, (a) detail of gallery and zone injected with mortar

In the zone around the right wing curtain 41 observation holes for ground water level recording, and four measuring veirs in the galleries for flow measurement were installed.

The amount of works on this very large grouting Project is shown in Table 6.7. In addition 39,200 m^3 of concrete was used in the diaphragms and walls in the river section and in the right wing curtain.

TABLE 6.7.

Kao Laem grout curtain, total amount of work

Grout curtain	drilling m	surface m^2	drilling m/m^2	cemen t	specific cement take kg/m	kg/m²
River section	179,000	68,300	2.62	14,000	78	200
Right wing	380,000	512,000	0.74	50,000	130	100
Total	559,000	580,300	-	64,000	-	-

Grout curtain of the El Cahon Arch Dam

Very interesting solutions were developed and experiences were gained on the injection of the grout curtain below the 232 m high El Cahon Arch Dam in Honduras (Flores et al., 1985). The foundation rock of the dam consists of limestones of the Cretaceous age which were very karstified before the region was covered by lava from the volcanic activity, which lasted from the Oligocene - Miocene age to the Quaternary. The lava was later on completely eroded in the canyon and the limestones were uncovered. The karstification of the limestones was determined only when injection of the curtain started. It was found that the intensity of karstification of the limestones did not decrease with the depth. As it was the consequence of hydrothermal processes during the volcanic activity, some thermal springs were discovered also in the excavation for the underground power station foundations. Because of these conditions, the original design of a hanging vertical grout curtain had to be changed. The curtain was inclined upstream and bound into the little permeable volcanic rocks in the valley bottom, and extended to the volcanic rocks in the abutments, so that the curtain was completely closed into tight rock, as shown in Fig. 6.29.

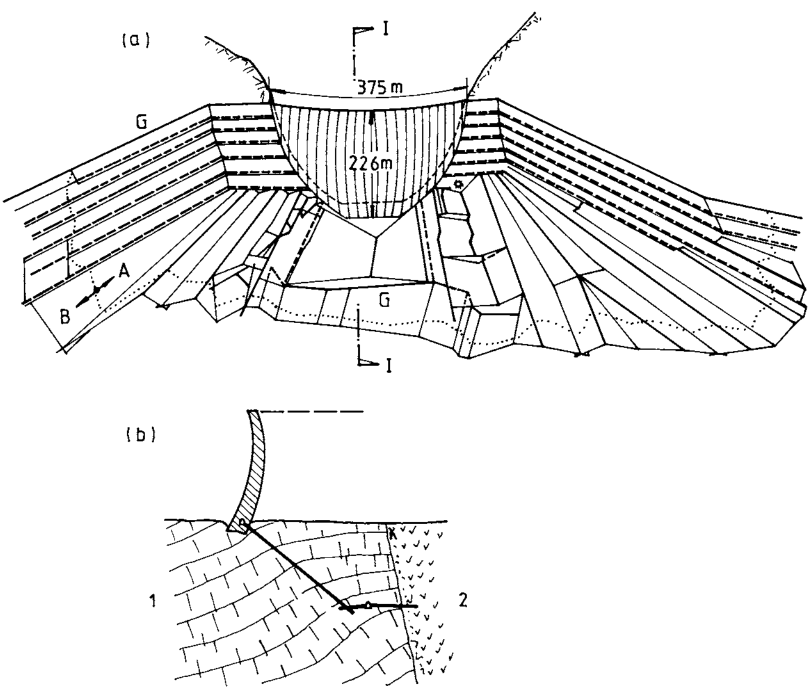


Fig. 6.29 View from upstream on the grout curtain of El Cahon Dam, A zone of limestones, B zone of volcanities, (b) sketch of the cross section I-I, 1 limestone, 2 volcanic rock

The karst features, which were encountered during the excavation of the grouting galleries, were treated differently according to their size:

- smaller voids were connected by pipes through the lining into the gallery for later injection;
- the material filling larger caverns was excavated and the spaces were filled with concrete.

Split spacing with three sets of injection holes was used for grouting, the primaries at 10 m o.c. When the grout take in the tertiaries exceeded 200 kg/m, quaternaries and, if needed quinaries at 0.63 m spacing, were drilled and injected. The grout consumption in the sets of holes is shown in Fig. 6.30 for the river section and for the abutments, separately for limestone and for volcanities.

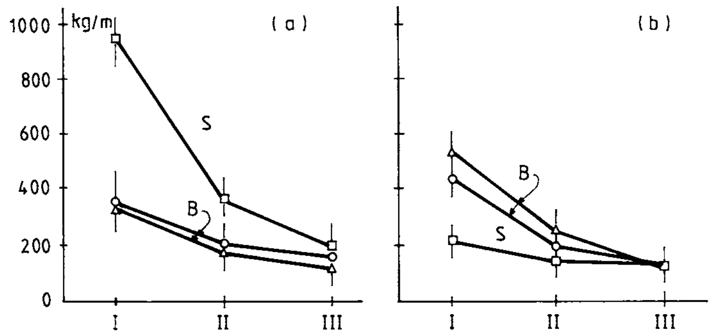


Fig. 6.30 Average grout consumption in sets of holes, (a) in limestone, (b) in volcanities, S-valley section, B-abutment section of the curtain, I primary, II secondary, III tertiary holes

It is evident that the basic spacing between the holes was well selected, because the cement take was substantially reduced in the secondary holes, that the use of a thick starting suspension with 1C:2W was well selected, and that injection with a high pressure was successful. From the presentation in Fig.6.31 it is seen that 80% of the grouted stages in the tertiary holes took less than 200 kg/m of cement, in the quaternaries 95% of the stages satisfied this criterion.

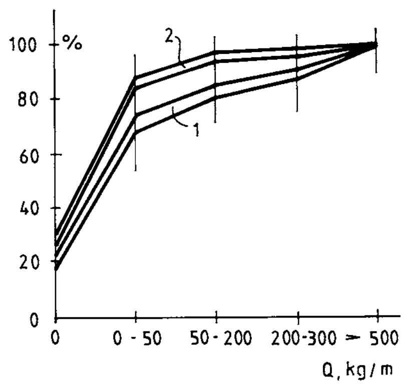


Fig. 6.31 Percentage of grouted stages with grout within the indicated intervals, 1 Tertiary, 2 Quaternary holes

Grouting was mainly uphole, except where the rock was too fractured, and the injection stages were 5 m long. The saturation criteria are shown in the flow chart in Fig.6.32.

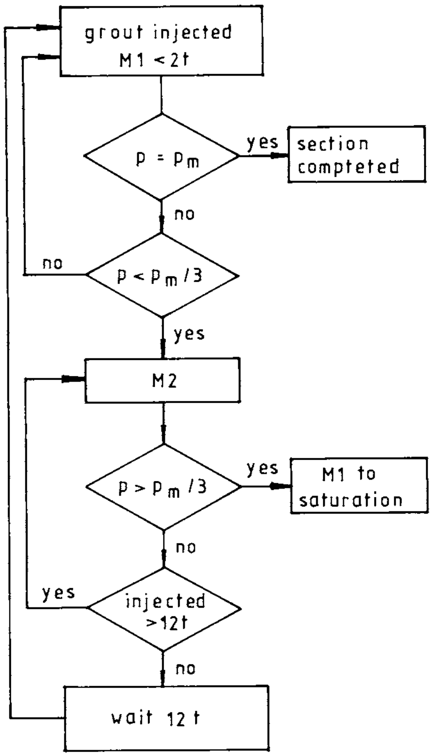


Fig.6.32 Flow chart for saturation criteria of 5 m long grouted stages, M type of suspension, p_m saturation pressure of the injected stage, in very fractured rock pressure reduced to p/6 if not saturated when grout take more than 2 t (Flores et.al., 1985)

The injection pressure was selected according to the thickness of rock cover S above the gallery and the maximum hydrostatic head H as follows:

$D < 2H$, $\quad p = 0{,}5$ bar/m in primaries and secondaries,

$p = 0{,}25$ bar/m plus 25 bar in tertiaries,

$p = 50$ bar in additional holes in limestone,

$p = 30$ bar in additional holes in volcanities,

$D > 2H$ $\quad p = 0.5$ bar/m of rock cover but max $p = 50$ bar in limestone and 30 bar in volcanities

Only thick suspensions were injected, type I: 1C:0.02B:0,6W and type II: 1C:0.02B:0.15S:1W. The Marsh cone visosity was 4050 sec, and the sedimentation requirement after 3 hours was 1,5% - 2,5%. The reasons for using such dense mixes and a high pressure are the following:

- the limestone rock is permeable in the fault zones and karst features which can be permeated with thick suspensions;

- the hardened grout must achieve high strength in order to resist hydrostatic pressure up to 30 bar;
- very fine grained cement of 4,600 cm/g was used and large cement particles, which block the penetration in finer fissures, were not present.

Inclined check holes were drilled through completed sections of the curtain which were accepted when the following criteria were satisfied in all 5 m long test sections:

- permeability less than 2 LU;
- cement consumption less than 50 kg/m.

One inclined check hole was drilled at every 50 m of the grouting gallery. The results are presented in Fig.6.33, from which it is evident that 91 % of the tested sections of check holes have satisfied the required criteria.

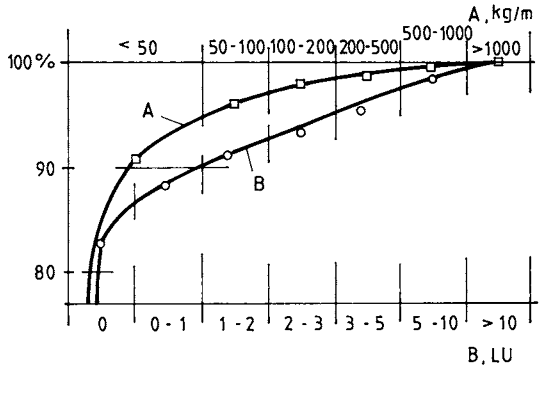


Fig.6.33 Average of check hole tests satisfying the cement take *A* and the permeability criteria ***B*** of the El Cahon grout curtain

For the whole grout curtain, 11,5 km of galleries were excavated, 530,000 m²of curtain were injected 485,000 m of holes were drilled, 100,000 t of cement injected with an average take of 200 kg/m of hole resp. and 170 kg/m²of curtain.

Conclusion on grout curtains in karstified rock

The examples of grout curtains in karstified rock show a wide range of rock properties encountered on the sites, which depend on the details of the variable geologic and hydrogeologic characteristics of the different sites, depending on the history of the geotectonic development of the sites and the physical properties of the limestone rocks. These characteristics govern to some extent the choice of grouting methods, the selection of materials,and the depth and extent of the curtain.

The experience of the leading exploration and design teams is dominant for the selection of grout compounds, arrangement of grout holes, and saturation criteria. Different approaches have led to satisfactory final results. The examples of the Peruča and Sklope dams illustrate that very deep holes can be grouted up to 200 m from the ground surface. Cement consumption was reduced to a minimum by using a high proportion of local clay for the preparation of grout compounds, and the clay and gauge filling in some parts of the fissures. Voids and cavities can be permanently stabilized by the injected grout without expensive cleaning and replacing by concrete, and thus the filling can be consolidated and incorporated into the curtain. In the examples of Keban and Kao Laem dams a substantial length of galleries were excavated in the center line of the curtain, so that the length of the injected holes was reduced to less than 20 m, and in the case of the Keban Dam a low injection pressure was specified. Many of the larger fissures were excavated from the galleries and treated with mortar, or plugged by placing concrete or diaphragm walls. In some cases such direct methods of treating very pervious and cavernous zones may be justified, but the less expensive methods of grouting should be tested before they are discarded in the design.

Constructing a grout curtain in karstified rock is a very difficult undertaking full of possible unexpected and disappointing discoveries during the development of project. Competent design and site engineers must be aware of the need to use flexible interpretations, and to adapt the course of work to any unexpected local conditions.

6.62 Grout curtains in diverse rocks

Although it is still a complex undertaking, grouting of other types of rock is much less complicated and unexpected site conditions seldom arise provided the exploration program is carefully prepared and carried out. Two examples will illustrate the greater predictability of the site conditions.

Grout curtain through masonry of the Aswan Dam on the Nile

The case record of grouting the masonry of the Aswan Dam on the Nile is interesting, because a detailed and careful record of the effect of grouting on the uplift and the percolation through the dam masonry provides a real picture of the effect achieved by grouting.

The masonry Aswan Dam is 34 m high and 1982 m long. It was constructed in 1900 as the first dam for the regulation of the Nile flow for more efficient irrigation of the fertile agricultural land in the Nile Valley. Later on it was raised to 39 m,and eventually to 48 m. It is built of granite blocks and flagstones set in cement mortar, which was not sufficiently resistant to chemical corrosion and leaching by the very soft Nile water. The mortar lost strength and became permeable, so that water percolation through the masonry progressively increased during the years. The downstream surface of the dam became wet, water seeped out of the masonry leaving sediments of calcium carbonate leached out of the cement mortar.

After careful studies and test grouting, it was decided to reduce the permeability of the masonry by injecting a grout curtain at the upstream part of the dam section, as shown in Fig. 6.34. The work started in 1960 and it was completed late in 1961.

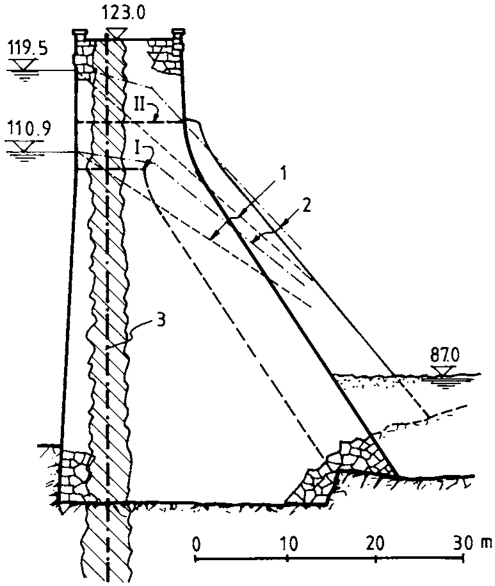


Fig. 6.34 Typical cross section of Aswan Dam and the position of the grout curtain, I and II dam construction phases, 1 linear distribution of uplift pressure in the foundation plane, 2 uplift pressure measured in piezometers, 3 center line of grout curtain

The design of the grout curtain included the following features:

- 2,000 m of exploration drilling with the installation of piezometers along the dam axis,
- 49,000 m of holes injected with Portland cement containing 20% of Kieselguhr from Fayoum,
- 100 m of redrilling for permeability checks.

All holes were carefully drilled and cored, cores containing mortar were analyzed in the laboratory, and the results are shown in Table 6.8. The average permeability of the masonry was about 35 times greater than the one of the mortar, so that clear water percolated through seams between granite blocks and the mortar, which were successively leached by the soft Nile water, and not through the bulk of the mortar. The permeability of the masonry and the granite rock in its foundation is presented in Fig. 6.35 (a) for a sector of the dam. The permeability of the masonry was evaluated from the flow rate and pressure of the injected suspension of 1C:3W during the first minutes of grouting.

TABLE 6.8.

Properties of mortar in Aswan dam

Construction phase	Description of samples	Properties of mortar		
		$CaCO_3$ cont. %	Porosity %	Permeability x 10^{-6}cm/s
Basic dam section	Compact to permeable	2 - 9	18 - 23	2 - 3
First heightening	Crumbly segregated partly weathered	5 - 7.5	17 - 19	0.9 - 5
Second heightening	Compact, hard	7 - 9	22 - 26	1 - 2

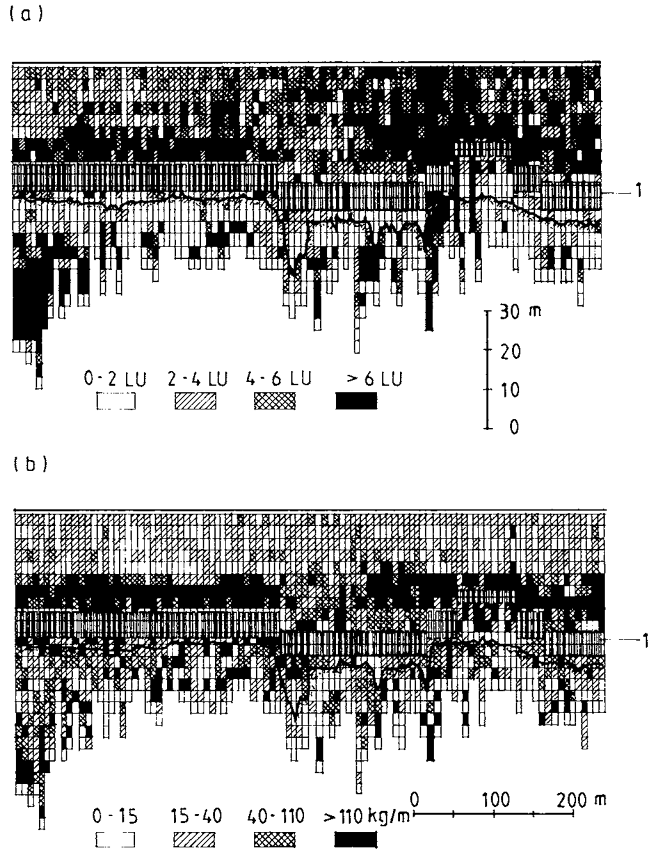


Fig. 6.35 Longitudinal section of dam sector 6 - 15, (a) permeability of masonry measured in injected 3 m long holes, (b) injected cement in the 3 m sections of the holes, 1 bottom sluices

The average results are shown in Table 6.9. The average permeability of sectors along the dam length ranges between 2.5 LU and 8.9 LU. In some sectors the permeability of the foundation rock was more than 6 LU even to the depth of 50 m, despite the several meters thick silty Nile sediment deposited in front of the dam. Evidently, the silt particles are too small to get trapped in the fissures of the granite.

TABLE 6.9.

Average permeability of Aswan Dam masonry and foundation rock

Description	Permeability	
	LU	$x10^{-6}$ cm/s
Foundation rock	5.0	75
First phase of construction	6.1	91
First heightening	4.7	70
Second heightening	3.8	57

The characteristics of the Portland cement, Kieselguhr and slag cement used for grouting are shown in Table 6.10. Initially, 83% Portland cement with 17% Kieselguhr was used for grouting, later on it was decided to use only slag cement from the Cairo Cement Factory.

TABLE 6.10.

Composition of cements and Kieselguhr used for grouting

Description	Portland cement	Slag cement	Kieselguhr
Specific surface cm/g	2,880	3,100	-
SiO_2 content %	21.2	24.3	37.6
$Al_2O_3+Fe_2O_3$ content %	8.8	10.0	15.0
CaO content %	63.4	59.6	22.6
Maximum grain size mm	-	-	0.03
Residue %	6.4	6.1	24.8

The mixing ratio of the suspensions was from 1C:3W to 1C:0.6W prepared in high turbulence mixers from which it directly was supplied to the injection pump which had a manometer and a pressure recorder. The movable mixing and pumping unit is shown in Fig. 6.36.

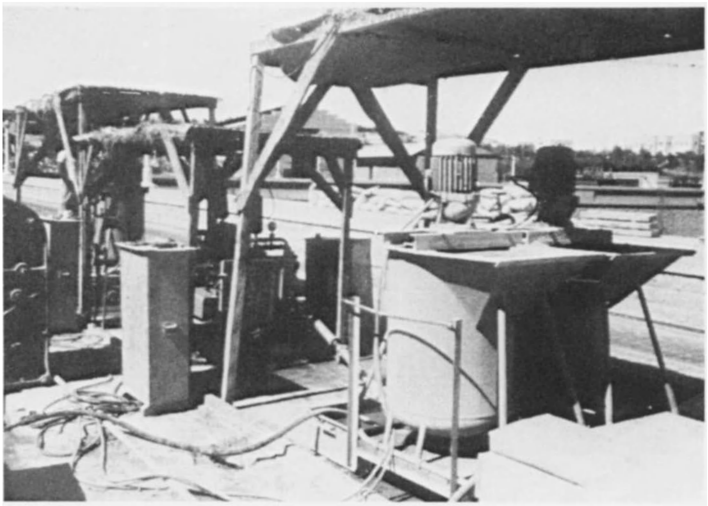


Fig. 6.36 Movable mixing and injection unit on the dam crest

The injection pressure was defined according to the depth of holes as follows:

- 15 to 20 bar in the rock foundation;
- 10 to 15 bar in masonry up to elevation 108.50;
- 5 bar to the final crest elevation;
- 3 bar at elevations of the first and the second phase dam crest.

The saturation pressure was reduced if outbreaks of grout occurred, or if saturation was not obtained after 500 kg/m cement was injected. The saturation criterion was fixed as less or equal to 10 kg cement injected during 30 min at a specified pressure.

Injection holes were drilled to the bottom, flushed with water under 5 bar pressure, then injected from bottom up in 3 m long sections. Split spacing was adopted in three sets, primary holes at 7.0m, secondary holes at 3.5 m and tertiary holes at 1.75 m spaced centers.

The grouting results of 7.0 m long dam sections were elaborated statistically. The results for every 3.0 m long section of injected holes for dam sectors 6 - 15 are presented in Fig. 6.35. The zone of high permeability also had a large cement take up to elevation 108.5. The cement take above this elevation seldom exceeded 40 kg/m, because it was grouted at a low pressure. The effect of systematic split spacing the distance of injected holes on the consumption is evident from the data in Table 6.11.

TABLE 6.11

Average grout consumption in subsequent sets of holes

Set of holes	grout take kg/m	ratio
Primary holes	92	1.00
Secondary holes	58	0.63
Tertiary holes	36	0.39

The result of grouting was evident visually when zones of seepage on the downstream dam face became dry or only slightly wet after grouting was completed. The Engineer expected however, that the water level in the piezometers downstream of the curtain would drop to the river water level downstream of the dam. Since this was not the case, it was argued that grouting was not successful. In order to demonstrate the possible effect of the curtain on the piezometer readings in the foundation plane of the dam, the theoretical flow nets of percolation through a typical dam section, with and without the grout curtain were computed. It was assumed that the grouted zone was 3,0 m thick and that its permeability was reduced by one order of magnitude. The results of the computation are shown in Fig. 6.37. The potential head in the piezometer before grouting would be $r_u = 4.2/6 = 0.70$ H, and after grouting the curtain it would be reduced to $r_g = 2.5/6 = 0.42$ H. Thus, the theoretical head efficiency of the curtain is $E_h = 1 - r_g/r_u = 0.4$.

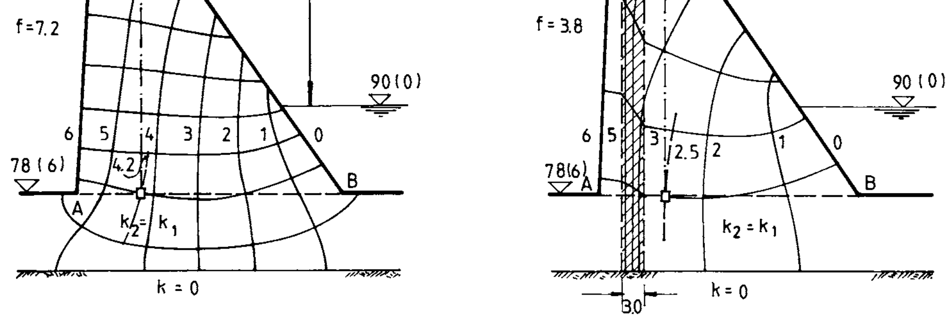


Fig. 6.37 Theoretical flow nets through a homogeneous section of Aswan Dam, (a) before grouting the curtain, (b) with grout curtain

The seepage flow can be computed from the flow nets by the expression $Q = H\,k\,f/n$, where f/n is the ratio of flow channels to head drops of the flow net. The ratio of percolation flow without and with a grout curtain is $Q_u/Q_g = f/n = 7.2/3.8 = 0.53$. The effect of the curtain on percolation was clearly visible on the downstream dam face, which was dry almost everywhere after grouting. The statistical elaboration of the pore pressure ratios r_u from all piezometer readings before and after grouting is shown in Fig. 6.38,which clearly indicates that the grout curtain through the dam masonry has a pronounced effect on the pore pressure in the foundation plane.

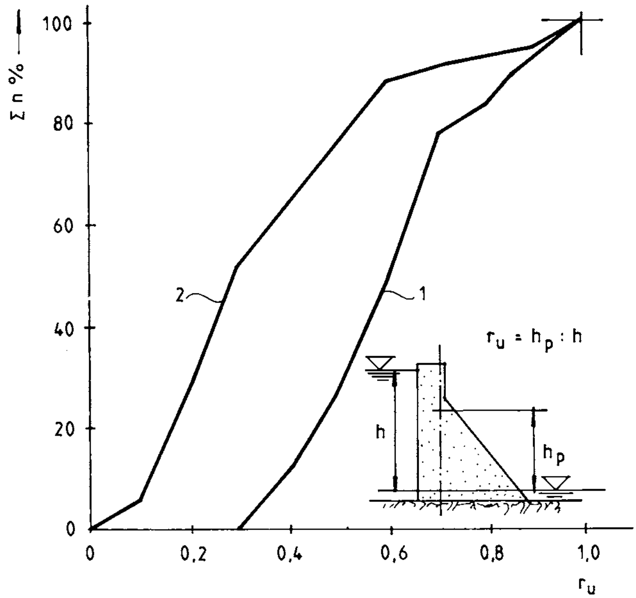


Fig. 6.38 Incidence of pressure head ratio r_u in piezometers, 1 before grouting, 2 with a grout curtain through the dam masonry

It can be concluded that the grout curtain in the Aswan Dam masonry and in the foundation had a beneficial effect in reducing percolation rate and leaching, and it increased the safety by efficiently lowering the uplift in the foundation plane.

Aslantas Dam grout curtain in Flysh

The embankment dam Aslantas was constructed on the Ceihan River North of Adana in Turkey. It is a 96 m high embankment dam, the reservoir has a volume of 1,746 million m^3 for the irrigation of the fertile Adana plain. The region around the dam consists of Flysh beds of Eocene age, covered on the surface by a slab of lava, which was eroded by the river, so that the dam is founded only on Flysh beds. The Flysh beds consist of thick siltstone beds of low permeability interbedded with thinner layers of brittle jointed sandstone, which are very pervious. The permeability of the formation is generally low, even in the fault zones. The geologic section is shown in Fig. 6.39.

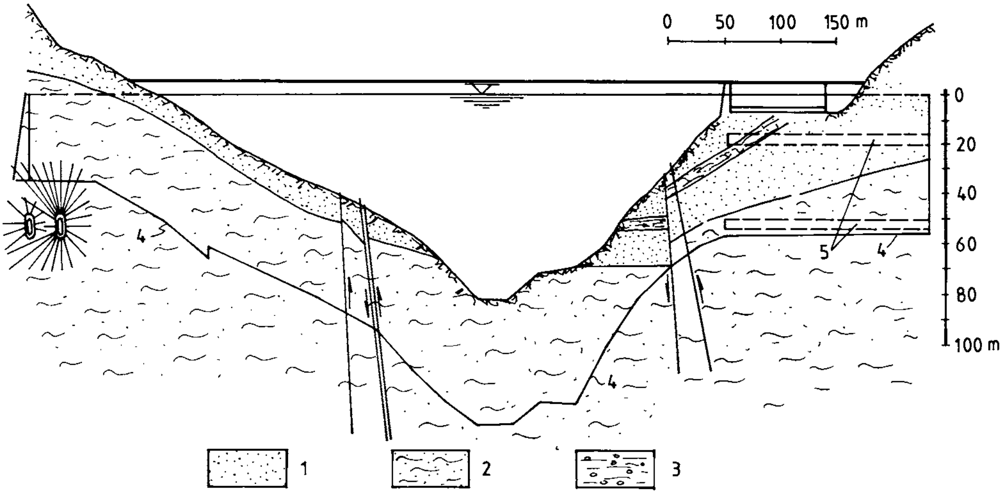


Fig. 6.39 Longitudinal section along the grout curtain axis of Aslantas Dam, 1 siltstone, 2 siltstone interlayered with up to 40 cm thick sandstone beds, 3 conglomerate, 4 boundaries of grout curtain, 5 grouting and injection galleries

The design of the grout curtain was based on the results of very detailed explorations and investigations, and on a grouting test plot. The curtain consists of a single row of injected holes, which were drilled from the surface of the excavated foundation trench for the core of the dam. The surface of the trench was provided with a 75 cm thick concrete slab, as shown in Fig. 6.40.

Before injecting the main holes of the curtain, two rows of consolidation grout holes 7.5 m and 15 m deep, 2.25 and 0.75 m upstream and downstream of the center line of the curtain were grouted. Two sets of holes were grouted, primaries with 6.0 m spaced centers, and secondaries between them. In some locations check holes at 1.5 m spacing were added. The curtain is 40 to 50 m deep from the foundation plane. The ground water contains a large amount of sulfates, so that sulfate resistant cement from a local factory, with 2 - 3% bentonite for stability, was used for the grout suspensions. The mixing ratio of the suspensions was 1C:3W, 1C:2W and 1C:1W. The injection pressure was relatively low, because the rather large fissures in the sandstone beds had to be injected only, avoiding hydraulic fracturing of the soft siltstone beds. The injection pressures were drilled to the bottom and then grouted uphole in 5 m long stages. Injection of all the holes was started with the 1C:3W suspension, then the thicker mix 1C:2W was injected until the grout take reached between 150 kg/m to 200 kg/m, then the 1C:1W mix was injected if needed. The saturation criteria required that not more that 30 l of the thinnest mix is injected during the last 30 min at the specified pressure. The curtain was considered to be closed when the cement take in secondary holes was less than 25 kg/m. Otherwise check holes were added. The measured permeability had to be less than 3 LU. The extent of work done for this curtain is as follows:

Fig. 6.40 Aslantas Dam, concrete slab in the foundation trench and grouting standpipe

Surface of the whole curtain	40,600 m
Total length of holes	28,400 m
Specific drilling length	0.70 m/m^2
Total cement injected	1,400 t
Specific grout take	39 kg/m (34 kg/m^2)

The results of a detailed statistical elaboration of the water pressure tests and the cement take in the middle row of the injected holes is shown in Fig. 6.41. It is evident that the permeability of the formations was generally low, and that the injection was very efficient.

6.63 Grout curtains in alluvial soils

The first large alluvial grouting for a cutoff curtain under an embankment dam was the one for the 130 m high Serre Ponçon Dam on the river Durance in France, where the sleeve grouting method (tube à manchettes) was applied for the first time on a large scale in a 100 m deep gravel deposit.

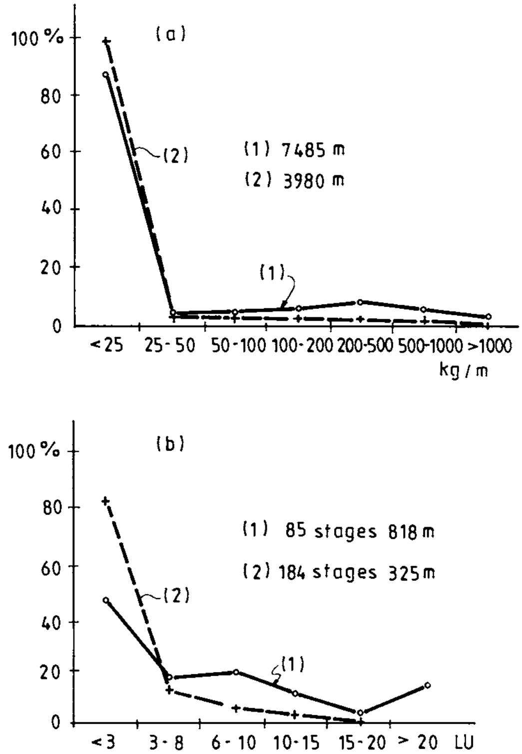


Fig. 6.41 Aslantas Dam, statistical elaboration of grouting results, (a) percent of stages by cement take, 1 average of all holes, 2 average secondary holes, (b) permeability, 1 in exploration holes, 2 in last set of injected holes

This method, which is very expensive and requires very experienced grouting personnel was not so often used since then because the diaphragm wall, first developed as a wall of intersected piles and then as a continuous wall of reinforced concrete, plain concrete or clay concrete which is often preferred to grouting, proved to be cheaper and the results can be better controlled. Nevertheless, it is interesting to present details of some executed grout curtains in alluvial beds.

Sadd el Aali Dam grout curtain in the Nile alluvium

The Sadd el Aali Dam on the Nile was constructed about 4 km upstream of the Aswan Dam. It rises the Nile level 100 m above the valley bottom. The Nasser Lake upstream the dam is 400 km long and reaches into Sudan at Wadi Halfa. The alluvial foundation is about 125 m thick and consists of sand and sandy silt beds about 45 m thick, sandy clay and clayey sandstone beds overlaying the granite base rock. The dam foundation has three lines of defense against erosion by seepage water, as shown in Fig. 6.42:

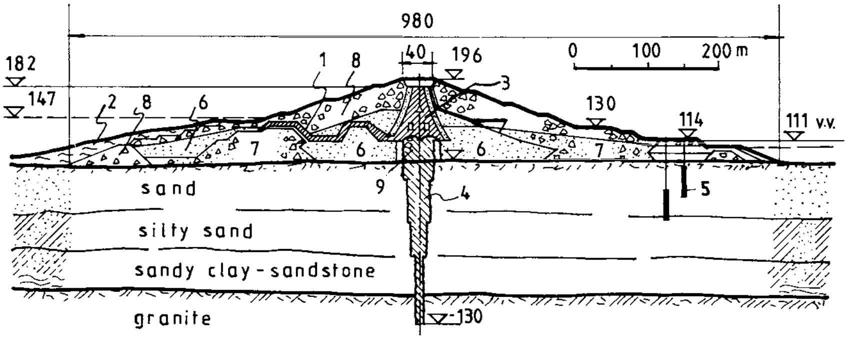


Fig. 6.42 Cross section of the Sadd el Aali Dam on the Nile, 1 upstream clay blanket, 2 blanket of silty river sediment, 3 clay core, 4 grout curtain, 5 two lines of drainage wells, 6 hydraulically placed sand, 7 rockfill hydraulically sluiced with sand, 8 rockfill, 9 hydraulically placed medium and coarse sand

- an upstream blanket of compacted clay (1) and of natural silty river deposit (2);
- a grout curtain (4) injected through hydraulically placed selected coarse and medium sand and through the river alluvium down to the granite rock base;
- two lines of drainage wells, (5) 75 and 50 m deep, below the river bed under the downstream toe of the dam.

Each of these defense lines would be sufficient for safety against erosion by seepage water, but it was considered necessary to triple the safety arrangements against erosion, because of the vital importance related to the safety of this large structure for the economy and life in Egypt. A catastrophe of the Sadd el Aali Dam would mean the end of civilization in the Nile Valley from the dam to the Mediterranean.

The grout curtain through the alluvial beds consists of two sets of equidistant holes with center spacings of 5.0 and 2.5 m respectively. The upper zone of the curtain through the hydraulically placed sand layer and the alluvial sand bed is 53 m deep and 40 m wide, through the silty sand the width is 30 m and through the sandy clay and sandstone to and in the granite it is 5.0 m wide. The total depth of the curtain is 245 m, out of which about 210 m was grouted through placed sand and alluvium. The area of the curtain is 60,000 m^2 and the volume of the injected soil was about 1,670,000 m^3. The length of the injected holes in the alluvium was 388,000 m, and injection pipes with 580,000 rubber sleeves were installed in these holes. Grout suspensions consisted of cement and clay, stabilized with sodium phosphate, in the ratios of 1C:0.6CL:1.3W, 1C:3CL:7.2W and 1C:4CL:10W, clay suspensions stabilized with sodium silicate in the ratios 1C:4CL:10W, clay suspensions stabilized with sodium silicate in the ratios 1CL:1.4W, bentonite suspensions with sodium silicate in the ratio 1B:7W, and finally sodium silicate in the mixing ratio 1SI:1.2W with aluminate reactive, the gel time of which was 30 - 60 min.

In the central batching station, suspensions of stabilized clay and bentonite were prepared in high turbulence mixers and stored in agitators with a volume of 300 m^3, and the silicate solution was stored in 300 m^3 reservoirs. From the central station, the suspensions and the silicate solution were pumped to fully automated grouting stations up to 3 km away, where cement, water and chemicals were added in order to prepare the needed grout suspensions which were mixed and fed to the injection pumps. The maximum monthly output of the installation was:

- 15,700 m of drilling in the alluvium;
- 20,700 sleeves installed;
- 35,000 m^3 of suspensions mixed;
- 98,000 m^3 of soil injected.

A total of 640,000 m^3 of suspensions and solutions were injected, out of which 27% were cement - clay, 48% clay, 12% bentonite and 13% sodium silicate solutions. The permeability checks proved that the initial permeability of the beds was reduced to 1/200 - 1/250 of the original value. The whole Project was completed within 4 years.

Asprokremmos Dam alluvial grouting around concrete diaphragm wall

The Asprokremmos Dam on the Xeropotamos river on Cyprus is a 60 m high embankment of gravel with a central clay core, founded on 30 m thick very permeable alluvial gravel beds. The cutoff through the alluvial gravel is a reinforced concrete diaphragm wall 80 cm thick, 180 m long and up to 28 m deep, and its surface is 3,182 m. It had to be keyed about 1.0 m deep into the underlying impermeable marly beds.

When the wall was completed, it was discovered that some joints between the panels contained a sandy material which could have been eroded under the pressure of the impounded water. It was also found that the panels were not properly keyed into the bedrock. Out of 37 exploration holes drilled through the panels, in seven holes only the bottom of the diaphragm was properly in contact with the bedrock. The rest of the holes had a gap filled with a gravelly sandy material from 10 cm to more than 40 cm thick. In order to prevent the possible erosion of the alluvial material from the gap, which had a total area of 17 m, it was decided to grout the alluvial material in front and partly downstream of the wall with cement-bentonite suspensions and with silicate solutions, as shown in Fig. 6.43. In addition to zone A, the initially foreseen contact grouting below the clay core foundation, zones B and C were injected to prevent the possible erosion of the not properly constructed joints of the wall between the panels, and zones C and D to prevent erosion from under the bottom of the panels which were not properly keyed into the bedrock. Zone D was provided also to strengthen the resistance of the alluvial material in order to provide sufficient horizontal support to the bottom of the panels which were not keyed into rock.

The tube à manchette grouting method was used to inject the alluvium. The PVC manchette pipes of 38 mm ID were installed into 96 mm bore holes. They had perforations covered by rubber sleeves at every 50 cm, as shown in Fig. 4.27. The sleeve grout consisted of 1C:2B:2W. The total length of the sleeve pipes amounted to 36,000 m.

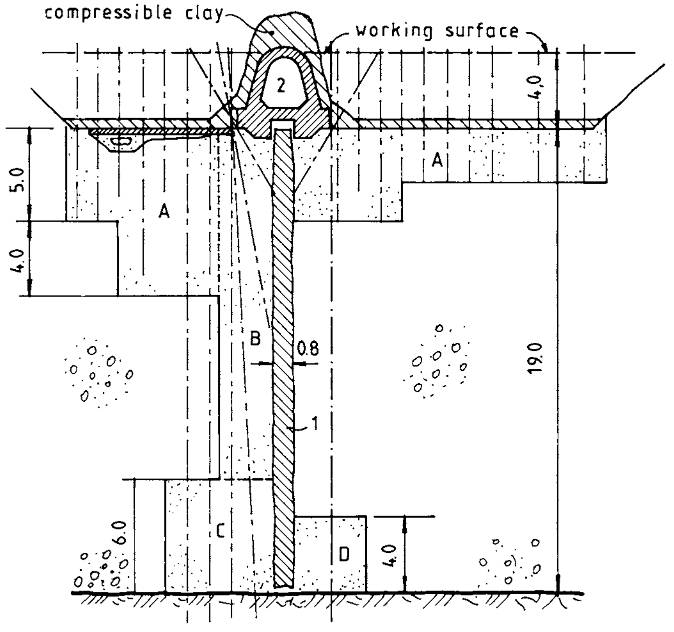


Fig. 6.43 Cross section through the Asprokremmos Dam foundation, 1 reinforced concrete diaphragm wall, 2 control gallery, A, B and C zones injected with cement-bentonite and silicate, D zone injected with cement suspensions

Zones A, B and C were injected from holes spaced 1.40 m. First all sleeves in the external row were injected with a suspension of 1C:0.05B:2W, which gives 2.32 m^3 of suspension per ton of cement. Then, every second sleeve of the inner holes was injected down the hole with a pressure of 0.45 bar per meter to a limit of 700 l per manchette. If the specified injection pressure was not achieved at this volume of injected grout, the neighboring upper and lower manchettes were also injected with the same suspension. Injection was stopped when the flow was less than 8 l/min. The remaining manchettes were injected with a silicate solution composed of:

- 522 parts by weight of sodium silicate M 75;
- 12 parts of sodium aluminate;
- 604 parts of water.

The gel time was 30 to 60 minutes the injection pressure was 0.75 bar per meter of overburden, and the injected quantity was limited to 1,250 liters per manchette. When the saturation pressure was not reached with this quantity, the neighboring upper and lower manchettes were immediately injected . Injection was stopped when the flow was less than 6 liters per minute.

In zone D, cement suspensions of 1C:1W were used, and it was specified to inject 2-3 t/m of cement. The holes centers were spaced at 2.5 m in a single line 2 m downstream from the face of the diaphragm wall. The permeability of the injected zone was not specified. A total of 60 holes were injected on a length of 150 m. The pressure required to inject the specified cement quantity was up to 50 bar, which obviously caused hydraulic fracturing of the formation. From some sleeves the grout reached as far as the ground surface from a 26 m depth. A total of 400 t of cement was injected on 240 m of sleeve pipes, amounting to 1.7 t/m.

In Table 6.12, the extent of work in the zones A, B and C is shown.

TABLE 6.12

Grout compound consumption at Asprokremmos Dam alluvium grouting

Zone	Volume m^3	Cement m^3	Suspension m^3/m^3	Chemical grout t m^3	Chemical grout t m^3/m^3	total m^3/m^3
A	13,800	1,063	0.077	1,926	0.140	0.217
B	4,515	754	0.167	148	0.033	0.183
C	5,500	594	0.108	166	0.030	0.198
Total	23,715	2,411	0.102	2,240	0.094	0.196

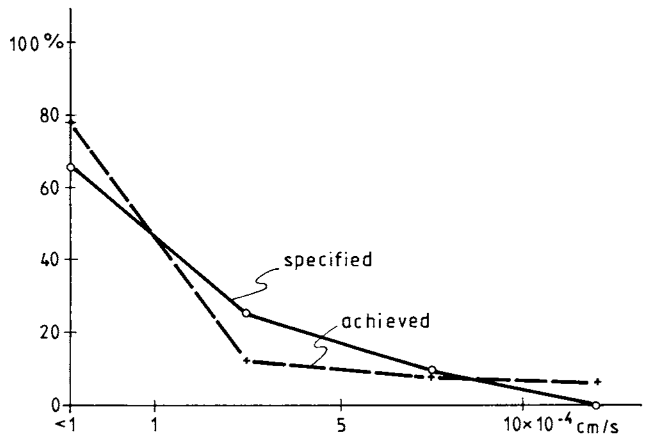


Fig. 6.44 Target specification of permeability ranges, and average results from permeability checks

The target specification of permeability of the injected ground is shown in Fig. 6.44, along with the achieved permeabilities. The average values of permeability, shown in Table 6.13, were measured through the manchettes of the injection pipe and through specially drilled check holes, which were also provided with sleeves.

From Fig. 6.44 and Table 6.13, it is seen that permeability of the injected alluvium varied by one order of magnitude in the range of $1.10^{-4} < k < 10.10^{-4}$ cm/s, and better results than the target specification were achieved.

TABLE 6.13

Summary of permeability checks of grouted alluvium

Zone	No.of tested sleeves	% of holes	Range of permeability 10^{-4} cm/s							
			<1		1 - 5		5 - 10		>10	
			No.	%	No.	%	No.	%	No.	%
A	280	5.5	211	75.4	34	12.1	24	8.6	11	4.0
B	82	21.0	65	79.2	11	13.4	6	7.4	-	-
C	100	12.4	84	84.0	10	10.0	55	5.0	1	1.0

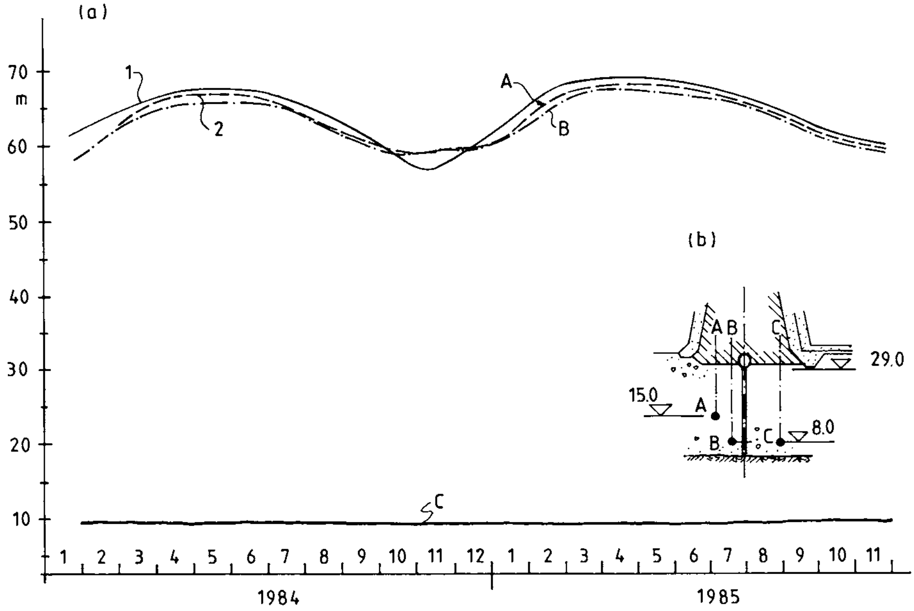


Fig. 6.45 Asprokremmos Dam, (a) registered ground water levels, 1 reservoir level, 2 piezometer levels A, B, (b) location of foundation piezometers A, B and C

The statistical elaboration of the grouting results has shown that better average results were obtained upstream of the wall, where deeper manchettes were injected at a higher pressure, which also accounts for the better averages in zone C.

It is interesting to note that despite the rather intense grouting with 1.5 m of holes per m^3 of ground, with a consumption of 100 l/m^3 of cement suspension (44 kg/m^3 cement) and of 94 l/m^3 silicate solution, the average permeability after grouting was more than $k = 1.10^{-4}$ cm/s (about 8 LU) and the specific cost was about 125 US$ per m^3 of injected ground.

The results from ground water observation in piezometers upstream and downstream of the diaphragm wall after filling the reservoir are shown in Fig. 6.45 for a maximum head difference of 59 m. The pressure in the upstream piezometers closely followed the development of the reservoir level, while the downstream pressure followed the downstream water level, indicating a tight cutoff.

Grouting for prevention of sea water intrusion in brackish sources

A large amount of rainfall in the Dinaric Karst drains through the permeable karstified limestone to submarine sources, or near the sea level along the Adriatic Coast, and it is an important contribution to the water supply of the dry coastal area and the islands. Many of these sources are brackish due to the intrusion of sea water, especially during high tide cycles. In some cases this can be prevented by injecting an appropriately located grout curtain.

An example of a successful intervention of this kind is the group of sources in the Zrnovica Bay near Novi Vinodolski, which feed the water supply system of the coastal region of Croatia. The sources spring from Cretaceous limestone near the sea level and partly below it. When the flow is low and the demand high, brackish water with up to 2.000 ppm sodium chloride seeps in and makes the water unsuitable for drinking (Pavlin, 1978). Coloring tests proved that rainfall from the higher karst horizon in Gorski Kotar (about 700 m above the sea level) drains into the sources at Zrnovnica near Novi Vinodolski, and Zminci near Bakarac, some 30 km to the North West. The sources are located at the ends of a deep impermeable barrier of Eocene Flysh, which obstructs percolation from the permeable limestone toward the sea. Coloring tests at the back of the Zrnovnica sources have detected the groundwater flow as shown in Fig. 6.46. Detailed investigations and explorations which were carried out have shown that infiltration of sea water into the sources can be prevented by a grout curtain in front of them (Fig. 6.46).

The southeastern end of the grout curtain is where the ground water level is permanently above the tide sea level in front of the Cardak water supply pumping station. At the northwestern end it cuts the ground water courses to the nearest sources. In the second stage of the Project it was supposed to be extended to cut the sources at the entrance into the bay, in order to increase the supply capacity of the Cardak pumping station.

Five exploration drill holes which were 50 m deep in the line of the first stage of the grout curtain have shown that the permeability of the limestone only at some locations was more than 2-3 LU, although it reached up to 20 LU at the northwestern end.

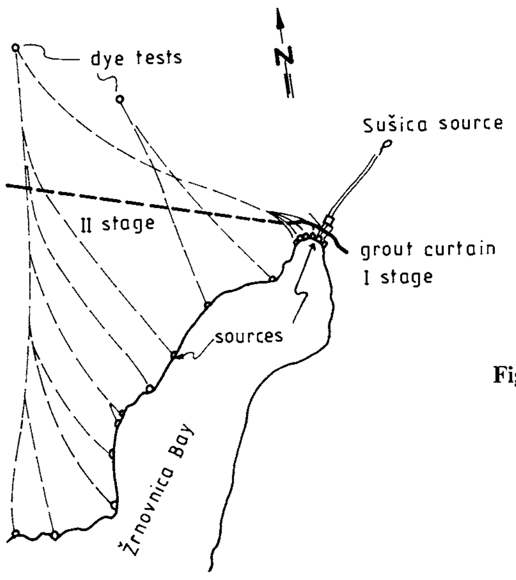


Fig. 6.46 Situation of the sources and the water supply station in the Zrnovnica Bay

The alluvial deposit in the bed of the creek was the most permeable. The grout curtain injected to prevent the intrusion of sea water into the fresh water is shown in Fig. 6.47. It is 150 m long, 30 m deep, and a single row of holes was injected into rock at a 5.0 m spacing.

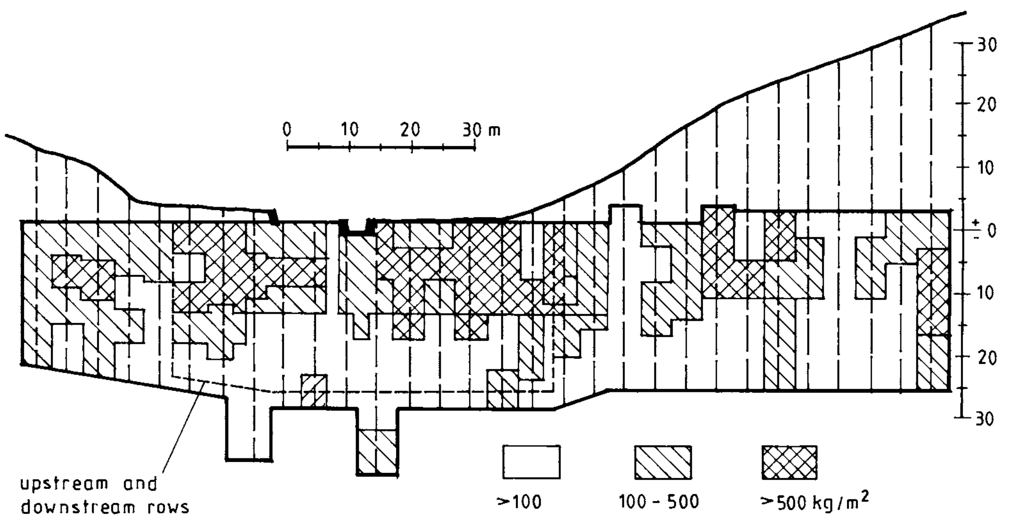


Fig. 6.47 Longitudinal section of the grout curtain in front of the sources in Zrnovnica Bay

One upstream and one downstream rows were added in the alluvial deposit, in some locations at a 1.0 m spacing. The total length of the 53 injected holes amounts to 1,260 m ; 1,000 t of cement, 53 t of bentonite and 113 t of sand were injected, and the average specific compound consumption was 925 kg/m of injected hole. In Fig. 6.47 the specific consumption per m of curtain surface is marked along the curtain surface. The injection pressure was 12 bar in the gravel, and up to 40 bar in the deeper sections in rock. The gravel was injected through the drill rods downstage, and the rock in 5 m long sections was also injected downstage.

When the grout curtain was completed, the ground water level rose in front of the curtain, the discharge of the sources was reduced, the capacity of the sources was substantially increased, and the chlorine content did never rise above 30 ppm. Thus, full success was achieved.

6.7 Parameters of various grout curtains

A review of the parameters of some completed grout curtains in various rocks may be useful for preliminary estimates of the hole density and the consumption of grout compound for the design of grout curtains in different foundation conditions. Data available for grout curtains in alluvial ground are assembled in Table 6.14. In Table 6.15 data for curtains injected in different rocks are assembled. It is interesting to note that the density of drilling for curtains in limestone and dolomite rocks is mostly in the range of 0.5 m per m and the grout take is around 200 kg/m. The grout take is much higher in karstified limestone, the average being between 200 and 600 kg/m, and in other rocks the consumption is much smaller. It is also interesting to note that the drilling density in the oldest grout curtains is much larger than the average.

TABLE 6.14

Parameters for alluvial grout curtains

Dam	Soil	Permeability		Injected	Injected soil	Injected	
		before	after	holes	volume	holes	comp.
		cm/s	cm/s	m	m^3	m/m^3	m^3/m^3
Serre Ponçon, France	a,b	$3x10^{-3}$	$3x10^{-5}$	17,000	100,000	0.17	-
Sylvenstein,	a	10^{-1}-10^{-2}	10^{-4}	10,000	73,000	0.14	-
FR Germany	b	10^{-3}-10^{-4}	10^{-4}	-	-	-	-
Mattmark,	b	10^{-1} - 10^{-2}	$3x10^{-5}$	71,000	460,000	0.15	-
Switzerland	c	10^{-2}-10^{-3}	-	-	-	-	-
Sadd el Aali, Egypt	a,b	10^{-2}	$4x10^{-5}$	328,000	1,670,000	0.20	0.45
Asprokremmos, Cyprus	a,b	10^{-2}-10^{-3}	10^{-4}	18,500	23,715	0.78	0.20

Notes: a gravel,
b sand,
c glacial till.

TABLE 6.15

Parameters for some grout curtains

Dam, height m	Type	Geologic rock formation	Depth m	Injected holes km	m/m^2	Consumption mix	kg/m^2
I. Dinaric Karst, Yugoslavia							
Liverovici, 49	CA	Triassic limestone	140	29.6	0.66	C:B:S	80
Krupac, wings zone 1		Cretaceous lime-	43	4.3	0.42	C:S	528
zone 2		stone, dolomite	30	2.4	0.57	C:S	320
zone 3			25	4.4	0.47	C:S	127
Široka Ulica, wing	-	Cretaceous limest.	75	18.2	0.35	0.3C:0.7CL	202
Peruča, 65, right	RF	Cretaceous limest.	200	68.0	0.58	0.5C:0.5CL	176
left		Cretaceous limest.	200	72.7	0.58	0.25C:0.75CL	248
Sklope, 78	RF	Cretaceous limest.	120	55.7	0.59	0.6C:0.4CL	290
Rama, 100	CFRD	Cretaceous limest. and dolomite	200	32.6	0.38	0.95C:0.05B	42
Spilje, 110	RF	Triassic & Upper Cretaceous limest.	145	90.5	0.48	1.0C	277
Kazaginac Dam, 20	RF	Cretaceous limest.	126	74.8	0.31	0.3C:0.7CL	48
Kazaginac Dam, wing	-	same	80	14.8	0.34	0.3C:0.7CL	304
Sinjski Ponor	-	same	100	31.6	0.25	same	139
Prozdri Koza	-	same	80	28.7	0.51	same	125
Metiljevica	-	same	67	7.1	0.27	same	123
Gorica, 30	CG	Upper Cretaceous limestone	70	8.7	0.46	0.4C:0.6CL	122
Grančarevo, 123	CA	Jurassic limestone	129	17.7	0.28	0.33C:0.67CL	49
II. Karstified limestone in other countries							
Camarassa, Spain	CG	Dolomitic limest.	92	50.0	0.16	1.0C	620
Santa Giustina, Italy	CA	Deteriorated dolomite	150	105.0	2.20	1.0C	80
Salto, Italy	CA	Deteriorated limestone	104	14.0	0.65	1.0C	160
Castillon, France	CA						
right		Bedded limestone	102	5.1	0.21	1.0C	157
left				2.9	0.29	1.0C	131
Genissiat, France	CG	Cretaceous sound limestone	104	4.7	0.30	1.0C	29
Sautet, France	CA	same	130	1.6	0.33	1.0C	155
Charmine, France		Karstified limest.		5.0	0.11	1.0C	265
Chaudanne, France		Fissured limestone		4.0	0.15	1.0C	40

Al Kansera, Algiers		Porous limestone		15.0	0.25		117
Oued Fodda,Algiers	CG	Fissured limestone	89	9.5	0.43	1.0C	300
Qaraoun, Lebanon	CFRD	Limestone stratif.		92.0	0.58		130
Dokan, Iraq	CA	Karstified dolom.	111	117.0	0.42	1.0C	181
III. Other kinds of foundation rock in Yugoslavia							
Moste, 54	CG	Triassic limestone breccia	49	26.6		1.0C	36 *
Medvode, 25	CG	Dolomite,cavernous	45	8.3		1.0C	67 *
Bajer, 15	CG	Paleozoic sandstone	20	1.7	0.71	1.0C	82 *
Lokvarka, 49	RF	Paleozoic shist & dolomite	20	5.6	1.20	1.0C	192 *
Kokin Brod, 80	RFT	Tertiary calcite-rous shist	25	19.8	1.15	0.65C:0.35CL	194
Globočica, 93	RFT	Tertiary stratif. limestone	74	20.4		0.45C:0.57CL	226 *
Aswan Dam,46 Egypt	MG	Granite masonry	30	49.0	0.57	1.0C	38
Keban, left rim curtain, Turkey	RF	Paleozoic lime-stone	300	76.0	0.34	0.95C:0.05B	305 910 *
Kao Laem, Thailand	CFRD						
under dam		Ordovician sand-stone and shist		179.0	2.62	0.98C:0.02B	200
right wing		Permian limestone karstified		380.0	0.74	0.98C:0.02B	100
Sidi Yacoub, 95 m, Algiers	RF	Cretaceous clay shist and silt-stone		28.0	0.66	0.97C:0.03B	37* 25
Aslantas,96,Turkey	RF	Eocene marl and sandstone		28.4	0.70	0.98C:0.02B	49* 34

Notes:

CA	concrete arch dam
CG	concrete gravity dam
RF	rock fill with core
CFRD	concrete faced rockfill dam
AFRD	asphalt faced rockfill dam
*	consumption in kg/m of hole

7. CONTACT AND CONSOLIDATION GROUTING

7.1. Scope and application of consolidation and contact grouting

Contact grouting is the process of filling the joint on the contact between the structure and its foundation. This can be the interface of a concrete gravity or arch dam foundation, or the contact joint between the concrete lining of a tunnel and the surrounding rock. The purpose of this kind of grouting is to achieve a tight contact between the concrete structure and the surrounding rock, so that they can act as a structural entity. Contact grouting involves only the joint and a rock zone 0.5 to 1.0 m thick.

Consolidation grouting involves a rock zone of greater thickness than contact grouting below the foundation of large concrete structures, or around the lining of tunnels or large excavated underground spaces. Its purpose is to reestablish or to improve and homogenize the deformation properties of the rock volume below the foundation surface where the rock was loosened by excavation blasting, in the zone of highest foundation stresses and highest deformability. The depth of consolidation grouting depends on the rock properties, the size of the structure and the load imposed by the structure.

Consolidation grouting is also carried out around the linings of tunnels in a zone where the rock is loosened by blasting and by the stress relaxation following the excavation around the tunnel. Part of the load of hydrostatic pressure on the lining of pressure tunnels can be transferred to the rock surrounding the tunnel if its deformability is low, thus reducing the tensile stresses in the lining, its dimensions and reinforcement. The deformability of rock around the tunnels perimeter loosened by blasting for excavation is increased additionally in a zone of stress relaxation by the redistribution of gravitational stresses in a volume of rock equal to about five times its diameter. These effects are cancelled or reduced by efficient consolidation grouting. In such conditions consolidation grouting contributes to improve the load transfer from the lining into the surrounding rock volume so that the cost of lining is reduced.

7.2. Contact grouting

The order of magnitude of the width of fissures and contact joints which have to be injected in contact grouting ranges between some tenths of millimeters up to 10 cm and more. Therefore, thick cement suspensions can be injected, preferably stable mixes which do not lose water and segregate when setting and thus do not leave areas filled with water only. For filling only smaller joints, suspensions of 1C:0.02B:0.8W are satisfactory, for filling larger voids the use of sand in suspensions of 1C:2S:0.02B:2.4W are needed. Where non shrinking grout is required (as in contact of steel liner with concrete) 1.5 g of aluminium powder can be added to 50 kg of cement.

The injection holes are drilled 0.5 to 1.0 m deep into rock in a pattern of equidistant holes at 1.5 to 3.0 m spacing, as shown in Fig. 6.9. The correct spacing should be determined in a grouting test plot. For grouting the contact joint of concrete tunnel linings the same equidistant hole pattern is convenient, as shown in Fig. 7.1. It is convenient to install the injection pipes on the forms before concreting the tunnel lining. After the forms are stripped, the injection holes are drilled through the pipes to the required depth with percussion drills. The injection pipes can be conveniently directed toward the deeper excavated spaces at the top of the arch which aids complete filling of the voids. It is important to install injection pipes as described when the excavation steel ribs or when the lining is reinforced; in unsupported excavations with plain concrete lining the injection holes can be drilled through the lining.

Contact grouting is done in one operation from one end of the area to the other . A group of holes is connected to the injection pump and grouted until saturation is achieved as specified in the design. As soon as connection with the neighboring holes is established, these are connected to the injection pumps and the operation proceeds in the desired direction until all the holes have been grouted. Saturated holes are closed by a valve before the injection line is disconnected in order to keep the injected grout under pressure until it has set. The same procedure is used for contact grouting of tunnel linings, in which case injection proceeds in one direction from one ring of holes to another, always starting from the bottom to the top of the arch, as shown in Fig. 7.1. It is convenient to organize the work as a continuous 24 hour operation.

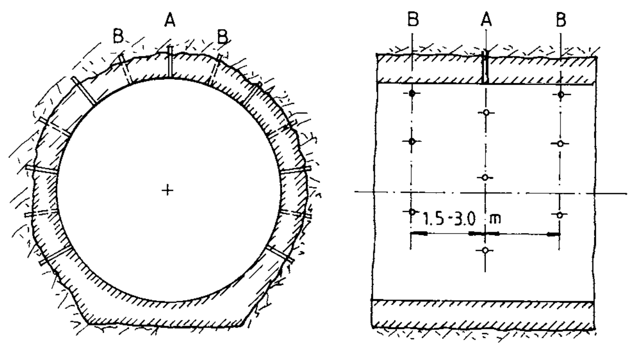


Fig. 7.1 Rings of injection holes for contact grouting of the tunnel lining

The injection pressure for contact grouting depends on the characteristics of the rock and of the overburden load in foundations, as well as on the structural strength of the lining in tunnels. The injection pressure should be tried in a grouting test plot when the Project starts. A pressure between

2 and 10 bar is satisfactory, the higher pressure should be preferred because it better consolidates the grout and greater strength of the injected compound is achieved. Too high an injection pressure may cause failure of the lining, as shown in Fig. 7.2.

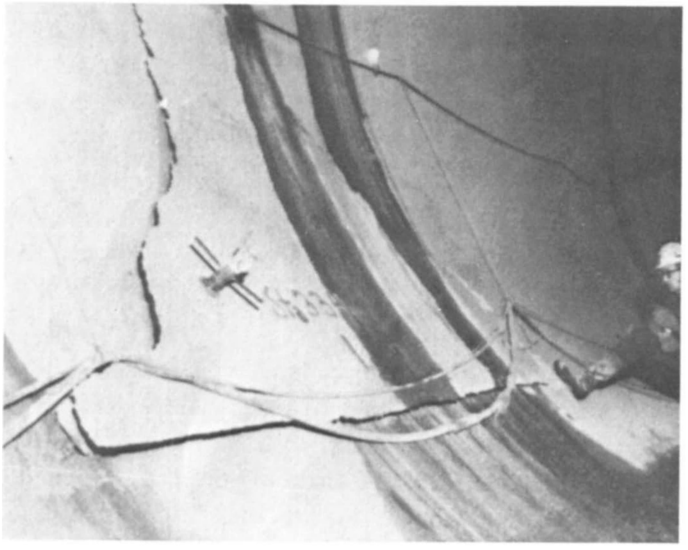


Fig. 7.2 Failure of the tunnel lining due to a high injection pressure (Johnson, 1982)

Sufficient mixing and pumping capacity is essential for a smooth development of such grouting operations, as well as an uninterrupted supply of all materials to the injection site. In tunnels of large diameter all batching, mixing and pumping equipment may be installed on movable units near the working site. In tunnels a of small diameter the batching and mixing plant may be at the portal from which the suspensions are pumped through a line to the injection site, where the injection pumps are fed. In some cases the suspension was pumped to a distance of up to 1 km, and when the site was farther, an auxiliary pump was installed which supplied the grout farther. Agitating containers were used in some cases to transport the mixed suspensions from the portal to the injection site. The required capacity of such installations may be up to 700 t of cement per 24 hours.

Three movable platforms are required on the working site, one for the drilling team and the installation of the injection pipes, the second for the injection pump and the third for the team which closes the injection valves and moves the injection lines.

Contact grouting is mostly regarded as routine work, but it should be well defined in the design and specifications, and also scrupulously supervised in every detail. The consequences of the failure to carefully fill the voids around the tunnel ling will be described in Item 7.43.

7.3. Consolidation grouting

Since the main purpose of consolidation grouting is to reduce the rock deformability, all existing fissures, joints and voids must be filled with high strength incompressible grout. In many cases it is essential that fissures filled with any sort of gauge should be cleaned of any filling material. When consolidation grouting is carried out below structural foundations the injection pressure is limited by the low overburden pressure. Neat cement suspensions are therefore required with additives for increasing fluidity and penetrability at a low water content of the suspension. Additives used for structural and mass concrete are adequate for this purpose. The equipment described in Chapter 5 should be used for the preparation of suspensions for consolidation grouting.

Injection holes in equidistant pattern are drilled as mentioned for contact grouting, generally the same holes are drilled deeper. It is convenient to inject the holes in sections from top to bottom in order to form a sealed upper layer which prevents surface leakage when deeper sections are injected with a higher pressure. The injection procedure and saturation criteria are the same as described in Chapter 4.

Injection pressure depends on the systems of joints and fissures of the rock, preferably as high a pressure as possible is used so that the injected suspension is well consolidated. On the other hand the pressure is limited by the overburden load, the possibility of surface leaks and lifting the excavation surface by hydraulic fissuring. For the control of surface lifting, bench marks for geodetic survey are installed, but also other alarm systems may be installed with light or sound signals. The injection pressure must be reduced when lifting of the surface starts.

Thick grout suspensions and relatively low injection pressures can be adopted for the injection of the upper zone of rock loosened by the excavation blasting. Suspensions of high penetrability and high injection pressures are required for successful consolidation of undisturbed deeper rock strata.

Joints and fissures filled with loose material must be washed clean before grouting, because the loose fill can not be penetrated by the cement suspension and the effect on deformability would be only partial. The loose fill must be washed clean before grouting, which can be done efficiently only by intermittently injecting every second hole in a series of holes with air and water under pressure. First, one series of holes is injected until clear water escapes from the adjacent ones, then the sequence of washing is reversed. High turbulence is caused in the fissures in the process which efficiently extrudes the loose fill. The procedure can be repeated a few times if necessary. When the washing of one 3-5 m long section of the holes in one series is completed, every second hole is connected to the injection line and injected until the grout gradually begins to appear in the other holes, which are then connected to the grouting injection lines and injected. Injection of every single hole is stopped when the specified saturation criteria (pressure and flow limit) are attained. The process is repeated in the next deeper section of holes with washing and injecting until the required depth is grouted in all the holes. Then the next set of holes is treated in the same way until the whole volume of the rock as specified in the design is consolidated.

Consolidation grouting the rock around tunnel linings should reestablish the original rock deformability in the zone loosened by blasting and in the stress relaxation zone which is about five radii deep from the lining. Deeper consolidation grouting might be necessary if the original rock deformability should be improved in order to engage the lining in the transmission of the hydrostatic pressure, but this is a different problem which may be solved by prestressing the lining, as will be discussed in Chapter 8.

The system of grouting is the same as described previously for consolidation grouting the foundation of structures. Usually the holes used for contact grouting are drilled in sections between 3.0 and 5.0 m long and grouted continuously as described for contact grouting.

Another area of application of consolidation grouting is weak rock in front of the excavation face in order to facilitate the excavation and reduce the pressure on support ribs and lagging. Holes in the direction of the tunnel axis are drilled around the perimeter of the excavation from the excavation face and they are grouted with compounds which depend on the kind of rock: either cement suspensions in crushed rock or combinations of cement clay and chemical solutions in gravelly or sandy soils. The treated length is up to 50 m in stages down the holes which are intermittently drilled and injected. When the soil has gained sufficient strength, excavation is resumed and the procedure is repeated as needed. This method is applied also to reduce the permeability of the material and inflow of water into the working area until the final lining is in place, and to stabilize zones of loose sand where conditions of hydraulic instability arise during the excavation.

7.4. Case records of consolidation grouting

7.41. Rock improvement in dam foundations

Test consolidation grouting was carried out in some cases during the exploration of dam foundations in order to determine the possible effect which may be achieved by consolidation grouting rock of inadequate properties for the dam foundation.

The **Sklope Dam** was first designed as a concrete arch structure for which an extensive exploration program was carried out. Among other investigations the deformability moduli of the foundation rock was tested in exploration adits by means of flat jacks. The disposition of one test is shown in Fig. 7.3. The slit for the flat jack was cut by drilling and chiseling so that the surrounding rock was disturbed as little as possible. The injection holes were drilled at spacings of 1.0 m to 75 cm in a row and these were spaced between 1.1 and 2.0 m from each other, symmetrically on both sides of the flat jack. The dynamic deformation moduli before consolidation grouting was 7,000 MN/m^2, after consolidation grouting values of E_{din} = 20,000 MN/m^2 were obtained, which is a threefold increase. The average dynamic moduli of the rock on the abutments was in the range of E_{din} = 15,000 - 20,000 MN/m^2, in the valley floor values of E_{din} = 30,000 MN/m^2 were measured. Since the geotechnical characteristics of the foundation were not favorable, the arch dam design was abandoned and an embankment dam was constructed (Mrvoš, 1963).

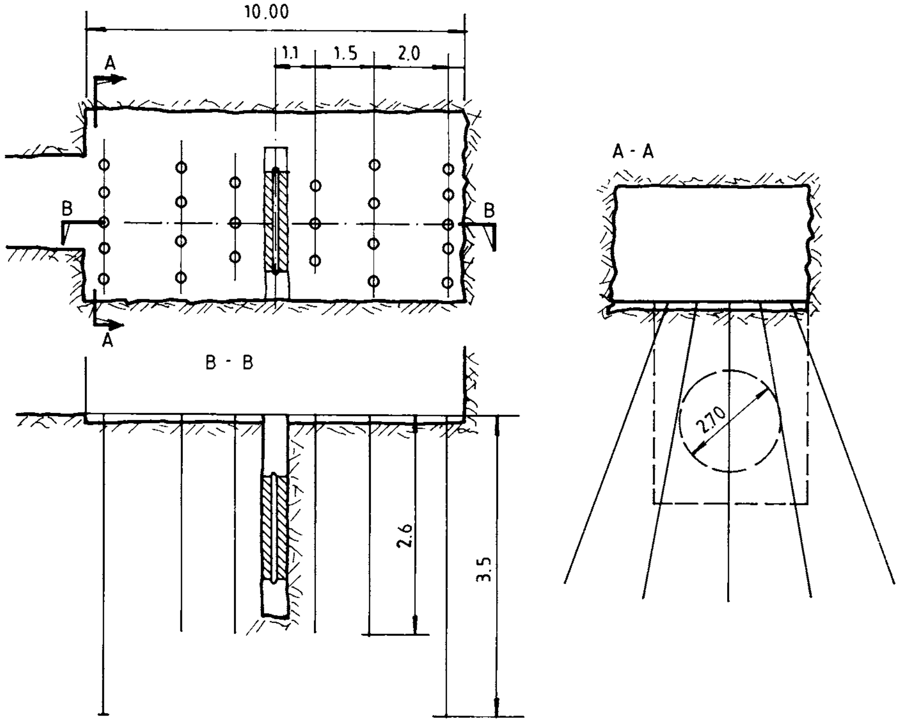


Fig. 7.3 Flat jack for testing rock deformability and disposition of holes for consolidation grouting

The disposition of the flat jack and the injection holes for consolidation grouting at the **Roge Dam** foundation is shown in Fig. 7.4.

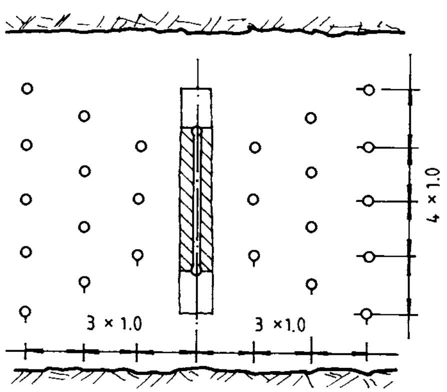


Fig.7.4 Disposition of flat jack and injection holes for consolidation grouting the Roge Dam foundation rock

Cement suspensions 1C:8W to 1C:1W were injected with a pressure of 12 bar after previous washing of the fill from the fissures with water under pressure and a flow of 0.5 l/m min. The whole group of holes was injected simultaneously . The results obtained are shown in Table 7.1.

TABLE 7.1

Roge Dam consolidation test grouting results

Moduli MN/m^2	before grouting	after grouting
of deformation	2,000 - 4,000	5,300 - 7,600
of elasticity	6,000 - 8,000	8,300 - 12,200

The grout consumption amounted to 50-85 kg/m of hole which is about 50 kg/m^3 of rock.

The results of this test show that an increase of moduli in the range of 50% of the original values was obtained by consolidation grouting.

The ***Grančarevo Arch Dam***, 123 m high is founded on slightly sloping limestone beds of Jurassic age (Lias). The bedding planes contain sandy and clayey well compacted material from a few millimeters to 30 cm thick. The deformation characteristics were measured at four locations with flat jack pressure tests. The deformation characteristics of the rock along the whole section through the foundation rock were interpolated from these values from seismic measurements of the propagation velocity of seismic waves and the resulting dynamic deformation moduli (Stoji, Torbarov, 1958, Stoji, 1968).

Consolidation test grouting was performed around two of these flat jacks. The injection holes were flushed clean by injection of water and compressed air, then cement suspensions were injected. The results are shown in Table 7.2.

TABLE 7.2

Consolidation grouting results, Grančarevo Dam flat jacks

Flat jack	Elastic moduli MN/m^2 before consolidation	Elastic moduli MN/m^2 after consolidation	Dynamic moduli MN/m^2 before consolidation	Dynamic moduli MN/m^2 after consolidation
A	15,200	16,750	25,500	43,000
B	7,300	14,650	36,500	65,000

The whole cross section of the foundation rock below the dam was investigated by the seismic method to the depth of 30 m below the foundation level before the foundation was excavated and after the excavation. The results are presented in Fig. 7.5. It is evident from the figure that the deformability was increased after the excavation,and that consolidation grouting increased the defor-

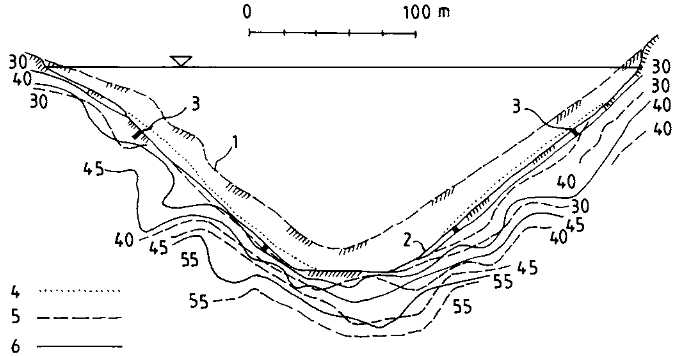


Fig. 7.5 Lines of equal dynamic moduli in the Grančarevo Dam foundation, 1 ground surface before excavation, 2 excavated foundation surface, 3 flat jack test locations, 4 Ed = 20,000 MN/m contour before excavation, 5 E_d lines after excavation, 6 E_d lines after consolidation grouting, in 10^{-3} MN/m

mation moduli substantially. The increase is more pronounced in zones of an originally low modulus. It is logical that consolidation grouting could not rise the moduli above the one of undisturbed rock. A statistical elaboration of the measured moduli is presented in Fig. 7.6 as the frequency of values before and after consolidation grouting of the foundation to 30 m depth.

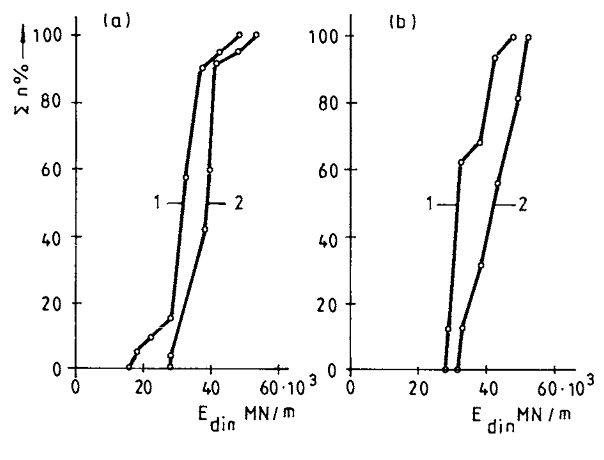


Fig. 7.6 Summary frequency of dynamic moduli in the foundation of the Grančarevo Dam, (a) left abutment, (b) right abutment, 1 before consolidation grouting, 2 after consolidation grouting

In Fig. 7.7 the average of all measured values of the dynamic moduli at different depths are shown before and after consolidation grouting. It is interesting to note that the range of values measured before is greater than after consolidation grouting. The average values of the moduli were increased by 5,000 to 10,000 MN/m and the increase resulting form consolidation is greater where the initial moduli are low.

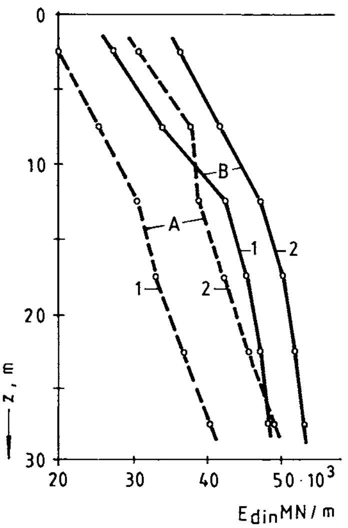


Fig. 7.7 Average values of dynamic rock moduli versus depth, (A) left abutment, (B) right abutment, 1 before, 2 after consolidation

It should be taken into account that the improvement of deformation characteristics of the foundation rock is a very expensive operation, and it must be carefully studied before the decision to apply it is made. For consolidation grouting of the foundation rock at the Grančarevo Dam a total of 1,500 holes with a length of 27,000 m were drilled, and 3,500 t of cement were injected, and the work lasted three years.

7.42. Contact and consolidation grouting around tunnel linings

Pressure tunnel for Hydroelectric Pover Plant Zakučac

Two pressure tunnels were constructed for the HEPP Zakučac one in the first, and the other in the second phase of the power plant implementation. The length of the tunnels is 9,570 m, out of which 8,450 m from the inlet is in the karstified and very fissured Cretaceous limestone, and the 1,120 m long rest to the end of the tunnel is in practically impermeable marl and sandstone formations of the Eocene age.

The first tunnel was excavated by blasting, the concrete lining was placed with a concrete gun. The second tunnel was excavated with a rotational mole, the concrete lining was placed by means of concrete pumps. The lining of both tunnels was contact and consolidation grouted with cement suspensions stabilized with some bentonite. The suspensions were mixed at the portals and delivered to the working sites through pipes where the injection pumps were supplied. In Table 7.3 the results of contact and consolidation grouting of both tunnels are assembled. It is interesting to note that the

TABLE 7.3

Contact and consolidation grouting of Zakučac tunnel, I.D. = 6.5 m

Item	Average, whole tunnel		Average, section in limestone	
	Tunnel 1	Tunnel 2	Tunnel 1	Tunnel 2
Length, m	9,570	9,570	2,900	2,900
Number of holes	25,781	25,079	7,911	6,291
Cement consumed, to	5,726	3,818	1,146	1,889
Specific cement consumption, kg/m	598	399	395	651
Cement, kg/hole	222	152	145	300

Note: Tunnel 1 excavated by blasting, tunnel 2 excavated with rotational mole

cement consumption is higher in the very karstified section of tunnel 2 excavated with a mole, which is the consequence of the very nonhomogeneous character of the karstified limestone. The cement consumption in the section of tunnel 2 through the Eocene marl was separately worked out and it amounts to 169 kg/m of the tunnel.

Collapse of a section of Tunnel 2 in Zakučac

The consequence of unsatisfactory contact grouting around the tunnel lining became evident after four years of operation in tunnel 2 of HEPP Zakučac in a section through the Eocene marl. The tunnel was excavated with a rotational mole, 7.5 m in diameter, the marl was hard and did not need temporary support. The surface of the marl was covered with a 5 cm layer of gunnite reinforced with steel mesh immediately behind the excavation face in order to protect it from weathering before the lining was constructed after the completion of the excavation of the tunnel. The contact between the pumped concrete lining and the rock was grouted with a rather thin cement suspension composed of 1C:0.13B:4W with a pressure of 5 bar. Four holes were injected per section 2.5 m distant from each other. Check holes were drilled and injected with a density of 7.8 m/hole at the top of the arch. The cement consumption was 8.2 kg/m of the lining. From the results of the check holes it was concluded that the contact grouting was satisfactory.

After four years of satisfactory operation of tunnel No. 2 it was discovered that turbid water emerged from the tailrace tunnel and slight knocks were temporarily heard in the turbines. The tunnel was emptied and inspected and it was discovered that in the section through the marl the lining had collapsed on the top of the arch on a length of 8 m and a width of 6 m. Above the opening in the top of the arch water had eroded a cavity about 20 by 35 m in plan and 28 m high. The lining was cracked and damaged up to 32 m downstream and 10 m upstream of the collapsed hole. The total volume eroded from the cavern was about 12,000 m^3, transported along the bottom of the tunnel. A sketch with sections of the cavern are presented in Fig. 7.8 and a view of the collapsed roof in Fig. 7.9.

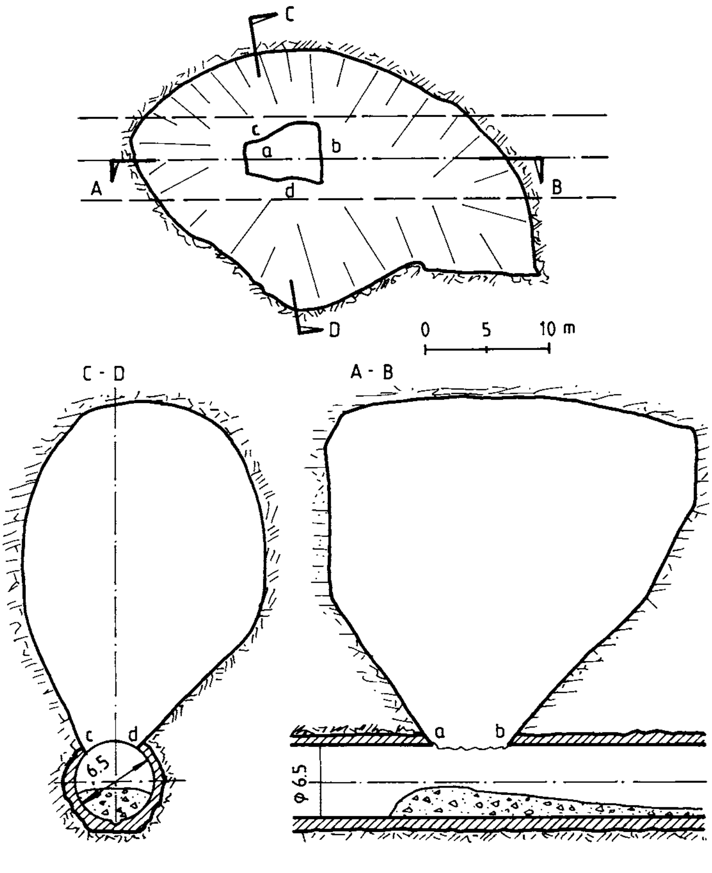


Fig. 7.8 Section of tunnel No. 2 and the eroded cavern above the top of the collapsed lining

From the hydrogeology of the region it was known that the ground water level was near the ground surface and the lining of the tunnel supported a hydrostatic pressure much higher than the inside water pressure. A seismic refraction survey was carried out after excavation was completed in order to detect any larger voids near the tunnel interface, which did not uncover any anomalies.

The investigation of the possible causes of the collapse have led to the conclusion that:

- during the excavation of the tunnel no traces of any swelling of the marl were detected;
- the outside hydrostatic pressure is much higher than the hydrostatic pressure of the water in the tunnel.

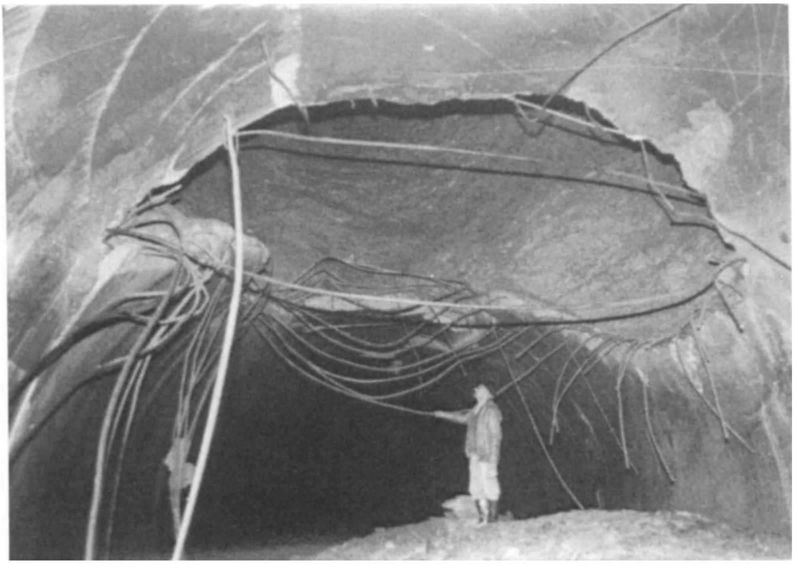


Fig. 7.9 View of the roof collapse in the Zakučac tunnel

Under such conditions the lining of the tunnel, constructed as it was designed, should have remained intact even under the most unfavorable ratio between the outside and inside hydrostatic pressure. It was obvious that the collapse could only be the consequence of some structural error in the top of the arch of the concrete lining. There it is essential that contact grouting fills all voids which normally remain due to settling of the placed fresh concrete. Where the lining at such places does not have the design thickness and voids between rock and lining remain after contact grouting, the outside hydrostatic pressure may damage the lining and crack open some holes. The stream of water in the tunnel causes high turbulences which may start eroding the marl and gradually a large hole may be eroded until the rock collapses at the top and destroys some portion of the lining.

A detailed survey of the stretch of the tunnel, where the collapse happened confirmed that the described scenario of the collapse mechanism is realistic. At one location a 10 by 30 cm large hole was discovered in the top of the lining where the concrete was only 5 cm thick, at another location about 1 m of the top of the lining was already missing and a small cavity already eroded to a depth of 1.5 m. No traces of the contact grout were found at both locations. An ultrasonic survey was carried out in order to investigate the hole stretch of the tunnel and additional 12 locations were detected where the lining was thin and smaller cavities not filled by the contact grout existed. It is probable that the thin grout that settled after the injection at low pressure of 5 bar, contributed to the faulty contact grouting.

It is interesting to mention that during construction neither the grouting contractor or the supervisory personnel of the Engineer reported to have detected any larger voids between the concrete lining and the rock interface during the drilling of the injection holes.

The consequences of probably insufficient care during concreting the upper part of the arch of the lining, and later the contact grouting, resulted in an interruption of operation of the tunnel of many months before the faulty section could be reconstructed.

Collapse of the Awali tunnel excavation

During the excavation of a 17 km long power tunnel for the Jezzine HEPP through the Lebanon range, at about half the length of the tunnel, where excavation commenced from the downstream end, a layer of sand under high hydrostatic pressure was struck, which invaded the face and filled about 3.200 m of the excavated tunnel. The first part of the tunnel was excavated in Jurassic and Cretaceous rocks. At the site of the collapse a water bearing syncline in sandstone rock was being excavated, and it degraded to fine sand, under a hydrostatic pressure of 70 bar. All attempts to remove the sand from the tunnel resulted in new sand intrusion from the tunnel face. It was necessary to drill long drainage holes along the material in order to stabilize the invaded sand before it was possible to reach the previous excavation face. A detailed exploration carried out after the collapse revealed that the section of the deteriorated sandstone in the syncline was 190 m long. It was decided to excavate it in five stages, each 40 m long. An array of 50 holes was drilled around the perimeter of the tunnel section and it was injected with cement/clay suspensions and a silicate solution. After the grout gained sufficient strength the section was excavated to a safe distance from the end of the previously injected array of holes, and the process was repeated until sound sandstone rock

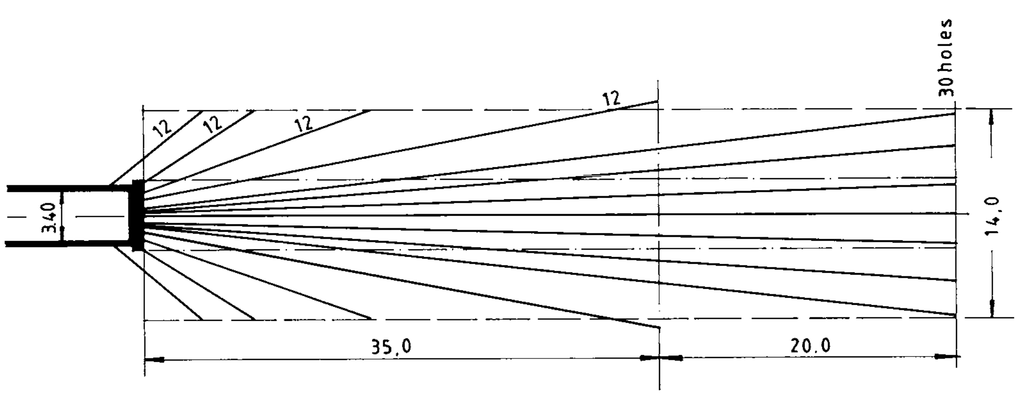


Fig. 7.10 Consolidation grouting for the excavation of the Awali tunnel through water bearing mobile sand strata, longitudinal section through the injected stage, 12 - number of holes in a ring

was reached. A sketch of the operation is shown in Fig. 7.10. A total of 10,000 m of holes was drilled, 90,000 m of redrilling through the injected compound and 4,550 m of drainage holes were drilled. The amount of grouting compounds was 10,600 m^3 of silicates, 3,200 t of cement and 90 t of clay. No special difficulties were encountered during the operations.

8. PRESTRESSING OF ROCK AND LINING OF PRESSURE TUNNELS BY GROUTING

Rock anchors and tendons one end of which is fixed in a bore hole by grouting are used for prestressing rock volumes and various concrete structures. When the injected compound has gained the necessary strength, the steel anchor is stressed and it transmits a force into the mass which increases the ambient stresses and increases its strength. Such strengthened rock mass can replace supporting walls of deep rock cuts.

Another field of prestressing by grouting is the lining of pressure tunnels in order to prevent the development of tensile stresses and fissures in the lining through which water losses of important magnitude would occur.

8.1. Rock anchors

The are two kinds of rock anchors. One kind consists of reinforcement steel bars placed in drilled holes, and the whole length is imbedded in cement mortar. When the mortar has hardened the bars prevent tensile deformations of the rock, as a reinforcement bar does in reinforced concrete structures.

The other kind of rock anchors is also placed in drilled holes but their inner end is only set in mortar by grouting and fixed to the surrounding rock. When the mortar has developed strength, the bars are tensioned and their free end is fixed to the rock surface, transmitting a compressive force to the surrounding volume of rock, thus creating a prestressed structure.

Several applications of rock anchors for reinforcement and prestressing of rock volumes and structures are presented in Fig. 8.1.

In the case of a supporting wall, the difference between a simple anchor and a prestressed one is illustrated in Fig. 8.2.

In order to activate force A in the anchor which is needed for equilibrium of the active wedge *abc* behind the wall, a deformation L_2 of the anchor is needed and a corresponding rotation of the wall will occur. If the anchor is pretensioned, the needed anchor force is activated by fixing the elongated anchor to the wall, and no rotation of the wall will occur.

Rock anchors are often used for temporary support of excavations before the final support is constructed. In such cases passive anchors are very often used. They are embedded in mortar without prestressing,and known as the "Perfo" anchor. In a 34 mm diameter hole drilled by pneumatic hammer a longitudinally slit perforated pipe filled with plastic cement/sand mortar is inserted, as shown in Fig. 8.3.

The reinforcing bar, 25 mm in diameter, is then pushed in by means of a pneumatic hammer. The penetrating bar displaces the mortar through the perforations and tightly fills the gap between the perforated pipe and the walls of the hole with mortar, thus fixing the reinforcing bar in the hole. After the mortar has developed strength, the extruding end of the bar is tightened to the surface of the rock by means of a steel plate and nut.

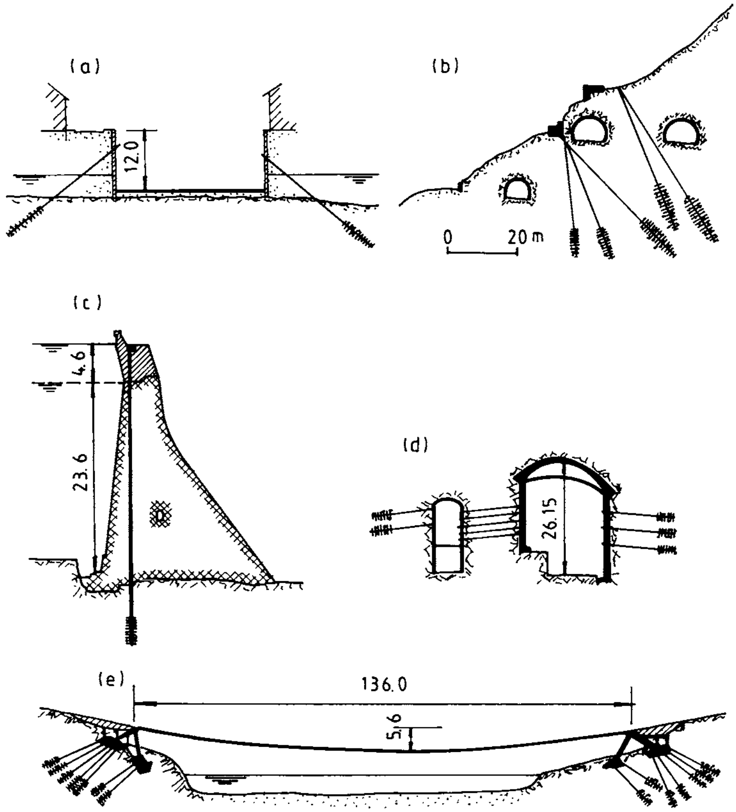


Fig. 8.1 Several examples of rock anchoring, (a) retaining walls of a deep excavation in urban environment, (b) improving the stability of a rock slope with motorway tunnels, (c) heightened concrete dam stabilized with tensioned anchors, (d) prestressing rock in underground excavations, (e) tension anchors for piers of a pedonal suspension bridge

The usual length of such anchors is between 2 and 6 m, and lengths of 10 m have been successfully installed with some caution. Some small prestress of the anchored length can be attained with "Perfo" anchors if the mortar at the far end of the slit pipes is mixed with a high early strength additive and the rest of the length with a strength retarder. The anchor is tensioned with the nut as soon as the mortar with the early strength additive has developed sufficient strength.

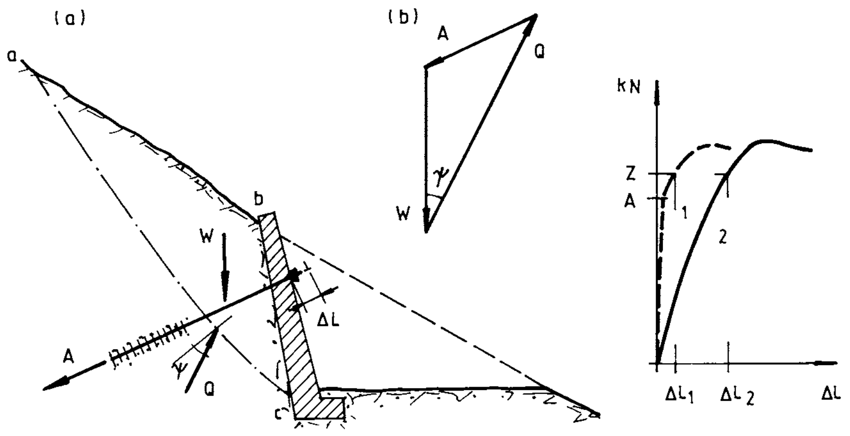


Fig. 8.2 Wall supporting a slope cut, (a) cross section of a wall and anchor, (b) polygon of forces for stability of wedge *abc*, (c) deformation of anchor, 1 prestressed, 2 passive anchor

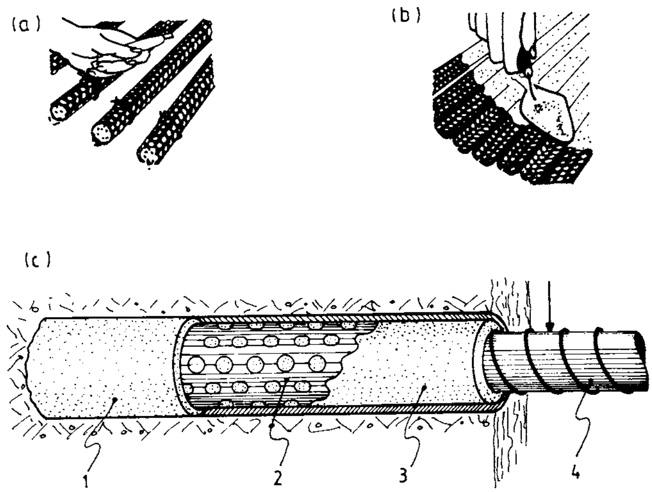


Fig. 8.3 Construction phases of "Perfo" rock anchors, (a) filling halves of slitted perforated pipes with mortar, (b) fastened halves filled with mortar ready to be inserted the in hole, (c) view of "Perfo" anchor in place.

Other solutions are available for long prestressed anchors of large capacity. A typical prestressed rock anchor is shown in Fig. 8.4. The anchor installed in the drilled hole of an adequate diameter, consists of a bundle of single reinforcing bars of 6 to 12 mm in diameter depending on the capacity of the anchor. Their number depends on the capacity and may range between 7 and 100 for anchor forces of 400 to 7,000 kN. At the exterior end of the anchor the bars are assembled in the stressing head, and at the inside end in a steel plate which transmits a part of the force directly to the injected mortar.

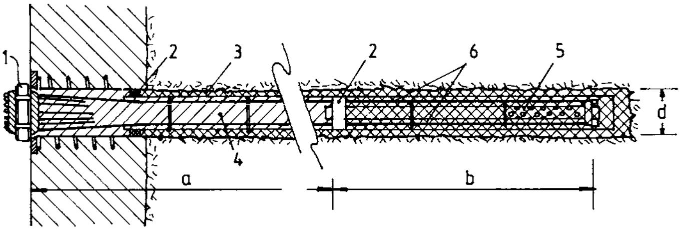


Fig. 8.4 Typical prestressing anchor, a free length of anchor, b grouted fixed length of anchor, d hole diameter 66-150 mm, tensile force 360-5,200 kN, number of steel bars 7 mm ϕ7-102, 1 stressing head and nut, 2 packer seal, 3 plastic hose, 4 cement grout or plastic filling, 5 injection pipe, 6 injected grout

The hole is grouted after drilling so that the rock volume around the anchor is consolidated, redrilled and then the anchor is introduced. A packer plug at the upper end of the fixed length of the anchor makes it possible to pressure grout the fixed section of the tendon. When the injected grout has developed sufficient strength, the anchor is stressed by a special hydraulic jack and fixed by a nut on the stressing head. A reinforced concrete block is constructed at the exterior end of the hole. It supports the stressing head of the anchor and distributes the force on to the rock surface. Special care must be devoted to protecting the reinforcing bars against corrosion. If the rock around the tendon anchor is not subjected to tangential deformations, the free length of the bars may also be filled with grout which provides good corrosion protection against moisture. Otherwise the free length of the bars is protected against moisture and corrosion by special coatings, and wrapping with plastic bands. Experience has shown that the grouted end of the tendon bars may corrode because tensile fissures may open in the cement grout from the tensile strain of the tensioned bars. There are special patented types of protection of the fixed length of the tendons which are sometimes used, but the problem of lasting corrosion protection of this type of rock anchors is still not ideally solved.

In another type of rock anchors the problem of corrosion of the fixed length is circumvented. The reinforcing bars are insulated against moisture by plastic sleeves from end to end. At the bottom they are fixed to a tube corrugated at the outside, as shown in Fig. 8.5. The tube and the bars are inserted into the hole and the fixed section with the tube is injected with cement grout. The hole on the free length of the anchor can be injected with mortar or plastic compounds for better insulation, or left empty as the case may require. In this case the anchor force is transmitted to the mortar and to the rock surrounding the hole walls by means of the tube which is compressed and does not cause tension cracks in the surrounding mortar, permanently protecting the tube from corrosion by ground water. There is a variety of other patented types of anchors permanently protected against corrosion on the market.

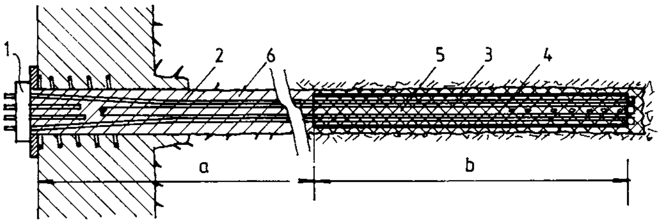


Fig. 8.5 Prestressing a rock anchor with compressed fixed end, a free length, b fixed length of anchor, 1 stressing head and nut, 2 steel bars insulated by plastic pipes, 3 outside corrugated steel pipe, 4 injection pipe, 5 injected mortar, 6 free length filled with mortar or plastic compounds

Much simpler anchors may be implemented for anchoring and prestressing temporary structures like sheet pile walls and retaining structures for deep excavations etc. in loose ground, as shown in Fig. 8.6.

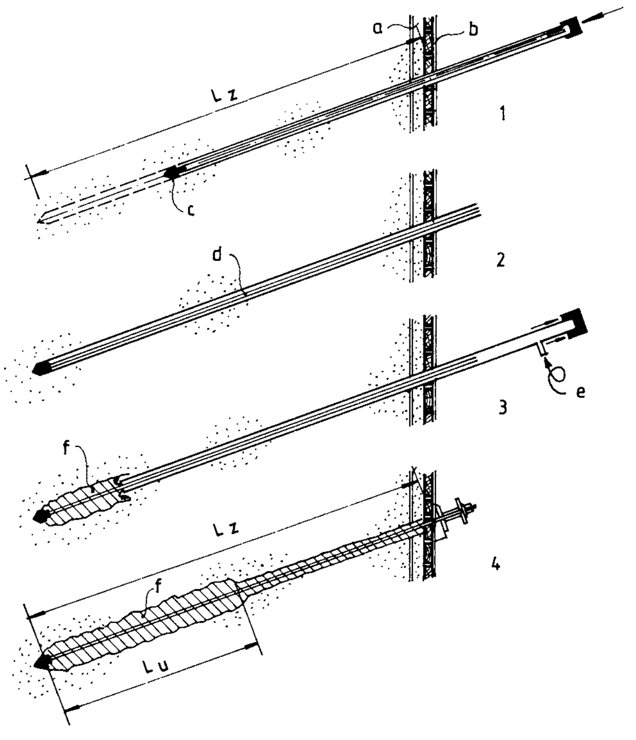


Fig. 8.6 Four construction phases of a temporary soil anchor, 1 driving casing with the anchor rod, 3 grouting and pulling out the casing, 4 stressing an anchor bar with a nut at the exterior end,(a)H-piles, (b) lagging, (d) anchor rod, (f) injected mortar

A driving point is fixed at the end of the anchor bar and placed in a casing tube. The whole is driven by an appropriate pneumatic hammer to the required length into the soil. Then the casing is connected to an injection line and slowly driven out, simultaneously injecting cement grout under high pressure until the fixed length is completed. The free length is then injected under low pressure to fill the hole. When the grouted mortar has developed sufficient strength, the anchor is fixed to the wall with a plate and a nut.

Another solution was implemented for the foundation of a building in New York City (Anonymous, 1965). The foundation was gradually excavated with the support of driven H-piles and wooden lagging. As the excavation advanced,sloping H-piles were driven at three levels, and their ends were fixed

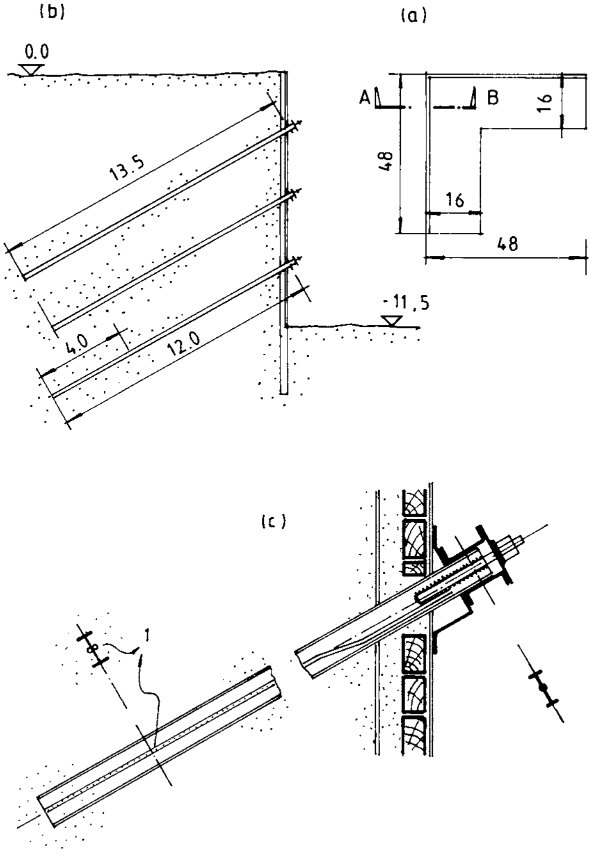


Fig. 8.7 Temporary H-pile supporting wall for the foundation excavation of a building in New York, (a) plan of building, (b) section A-B of H-pile wall, (c) details of H-pile anchor, 1 injection pipes welded to stem of H-pile

by grouting cement suspension through two injection pipes perforated at the end section, as shown in Figure 8.7. When the grout gained sufficient strength the beam was fixed and slightly tensioned by a nut to the soldier beams. The force of such an anchor reached up to 450 kN.

Measurements on such types of anchors have shown that forces, as shown in Table 8.1, can be developed and used in design of temporary structures.

TABLE 8.1

Design forces of soil anchors, fixed length 4.0 m

Type of soil	Consistency	Type of anchor	Design load kN
Fill	-	-	max 200 (1)
Cohesive	$I_k \geq 1.0$	permanent	200
		temporary	250
	$I_k < 1.0$	permanent	max 200 (1)
		temporary	200
Granular	-	permanent	200-300
		temporary	400

(1) or more if proved by pullout tests

A rough approximation of the bearing force of an anchor is obtained as a product of the interface area between the steel rod and the injected mortar with the adhesive strength of the mortar is about 400-600 kN/m^2. It results in a linear ratio of force to fixed length of the anchor. Many pullout tests have demonstrated that the linear relation is not valid and that the bearing force does not increase if the fixed length of the anchor exceeds 6.0 m. The reason for this will be shown by a detailed consideration of the mechanism of transmission of the anchor force from the bar to the surrounding mortar. The adhesion bond strength between the mortar and the steel interface is typically strain softening, as shown in Fig. 8.8 (a). As the bar is stressed longitudinally its diameter contracts,and the adhesive stress is further reduced to a low residual value τ_r when the strain δ_f exceeds a critical value. From the ratio τ_x/δ the bond stress distribution along the anchor bar can be computed, as sketched in Fig. 8.8 (b) for a different intensity of anchor force z from the first loading to the final pullout or failure of the anchor steel. From the bond stress distribution along the bar, the distribution of total anchor force along the bar is computed, as shown in Fig. 8.8 (c) and (d) in relation to the total elongation ΔL. It comes out that the active length of bar L_a depends on the shape of the stress strain ratio τ/δ and the elastic properties of the bar. A fixed length of the bar $L_f > L_a$ only slightly increases its pullout resistance. When the maximum force is reached, the resistance of the anchor decreases to a residual value which is a linear function of the residual bond stress and the fixed length L_u of the anchor. The force is reduced to zero when the fixed length is completely pulled out of the bonded length.

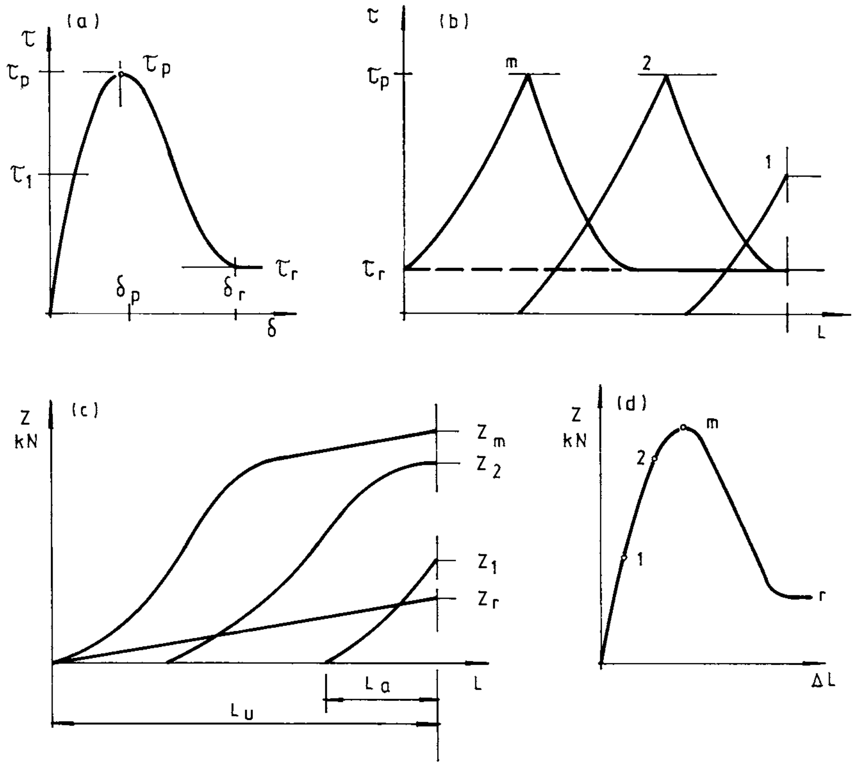


Fig. 8.8 Development of force transfer from the reinforcing bar to the surrounding mortar, (a) adhesive strengt τ vs strain δ, (b) development of bond stresses τ_p along the loaded anchor bar, (c) distribution of force F in the anchor bar, (d) force F in bar vs, elongation ΔL

The correctness of this force transmission mechanism was confirmed by pullout tests in which the strains and stresses in the anchor bar along the fixed length were measured. The test disposition and the results of tests carried out by Ostermayer (1977) are shown in Fig. 8.9. The anchor was in a sandy material, the hole diameter was 15.5 cm, and the anchor bar was fixed on a length of 3 m by injecting cement grout under pressure. Form the stresses measured along the bar by five stress gauges, the force in the bar and the contact stress on the sand to grout interface were computed. The distribution clearly agrees with theoretical distributions shown in Fig. 8.8. For practical purposes, it can be assumed that a fixed length of rock anchors in excess of about 6 m increases the cost and does not benefit either the anchor load or its safety.

The foundation stability on the left abutment of the Grančarevo arch dam had to be improved by the application of a large number of large capacity prestressed anchors. A part of the arch dam and the slope with the array of tendons are shown in Fig. 8.10. The dam was founded on relatively thinly bedded Jurassic limestone, the bedding planes were coated with sandy and clayey material, and the shear strength along the bedding was comparatively low (Stojić, 1965). In the critical part of the slope,the orientation of the bedding planes, joints and fissures were unfavorable,so that the stability

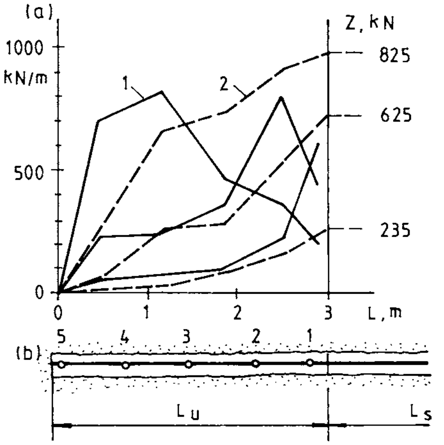


Fig. 8.9 Pullout test on an instrumented ground anchor, (a) anchor with stress gauges 1-5, (b) stress and force distribution along the anchor, 1 mean contact stress, 2 force along the fixed length of anchor

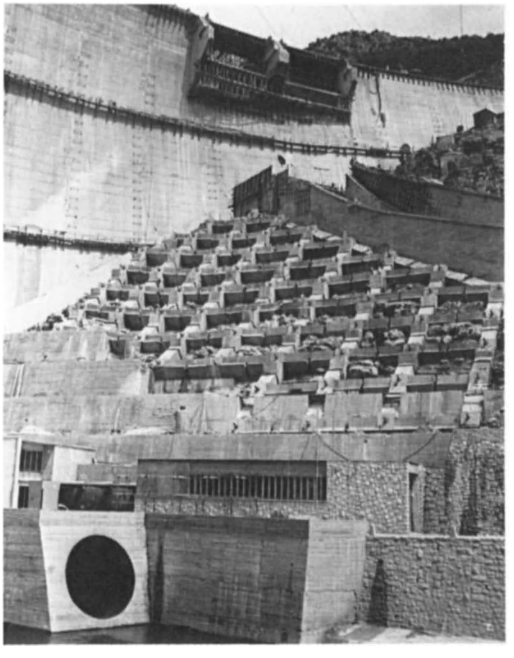


Fig. 8.10 View of the left abutment of the Grančarevo Dam with the field of large capacity rock anchors

and the bearing capacity of the dam foundation had to be improved. Smaller slides occurred during the excavation of the foundation, which were stabilized by the construction of massive retaining walls anchored with 280 "Perfo" rock bolts of 32 mm in diameter, each 10 to 25 m long.

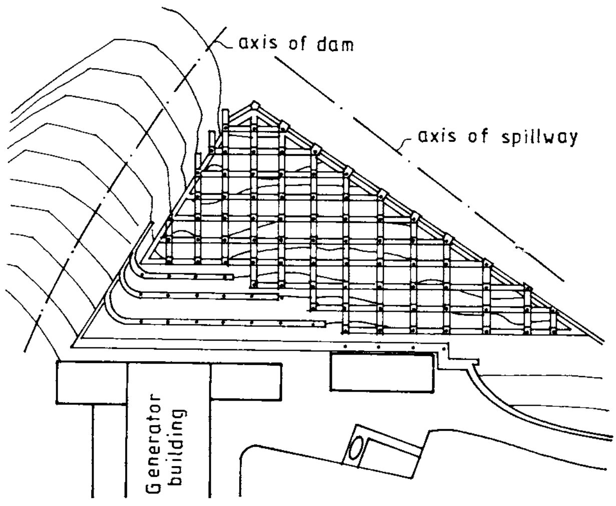


Fig. 8.11 Plan of reinforced concrete raft and rock anchors for stabilizing the left abutment of the Grančarevo Dam foundation

After a detailed investigation and study of the problem,it was decided to implement a field of prestressed rock anchors on the slope in the unstable part of the foundation. A total of 90 prestressed rock anchors of 2,000 kN capacity, 40 to 50 m long with a 5.0 m long, fixed length were installed. A raft of reinforced concrete beams in which the prestressed anchors were fixed was constructed on the slope, as shown in Figs. 8.11 and 8.12. The anchors were inserted in 133 mm diameter sloping holes, the fixed length of the anchors was 5.0 m, and it was injected with cement suspensions and additives for stability and prevention of shrinkage. The free length of the anchors was not grouted in order to prevent the development of tangential forces in the case of small slope movements. The free length of the anchors was thoroughly isolated from moisture and provided with cathodic protection against corrosion. Twenty anchors were provided with stress measurement gauges in order to have a permanent control of the forces and to be able to tension the anchors again if needed.

This extensive and very expensive application of rock anchors has permanently increased the foundation safety, but it requires a lasting supervision by experts so that any possible surprise is detected on time.

A similar rock prestressing on a much larger scale was constructed at the abutments of the unfortunate Vaiont Dam (Sirćo, 1965), where 300 rock anchors of 500 and 1,000 kN capacity were installed.

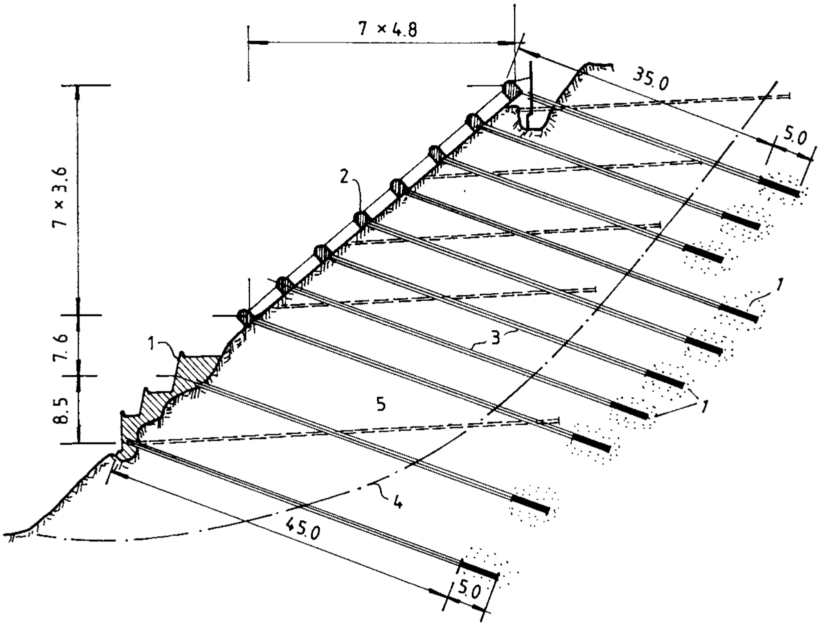


Fig. 8.12 Cross section through the field of rock anchors on Grančarevo dam abutment, 1 retaining walls with "Perfo" tendons, 2 reinforced concrete raft, 3 rock anchors, 4 critical slip surface, 5 drainage holes

For the construction of the new cokery in Bakar a large plateau had to be prepared partly by filling into the sea and partly by excavation of a geologically unfavorable slope consisting of Eocene limestone and Flysh beds of siltstone highly fractured and faulted. A stable slope could not have been constructed without high retaining walls at the foot, the construction of which had to be staged into 6.0 m long sections in order to prevent larger slides. The mass of the wall was not sufficient for stability, and rock anchors of 830 kN/m of wall had to be added.

A typical cross section of the retaining wall is presented in Fig. 8.14. On a 6.0 m long section of the wall five 1,000 kN anchors were installed in two rows. The holes for the anchors in siltstone were drilled, then injected with cement grout and redrilled before the bundles of anchor bars were installed. The fixed length of the anchors was injected with cement grout, and the free length was well isolated against corrosion. All anchors were tensioned to 1,500 kN and then fixed with a tension force of 1,000 kN. Some of the anchors were instrumented with 5 stress gauges each, by means of which the distribution of the tension force along the fixed length of the anchor was measured. The results measured on one anchor are presented in Fig. 8.15. It is interesting to note that more than 50 % of the anchor force was transmitted to the rock between the exterior end of the fixed length and the stress gauge 5 at the end of the anchor on a fixed length of about 1.8 m, and the rest was

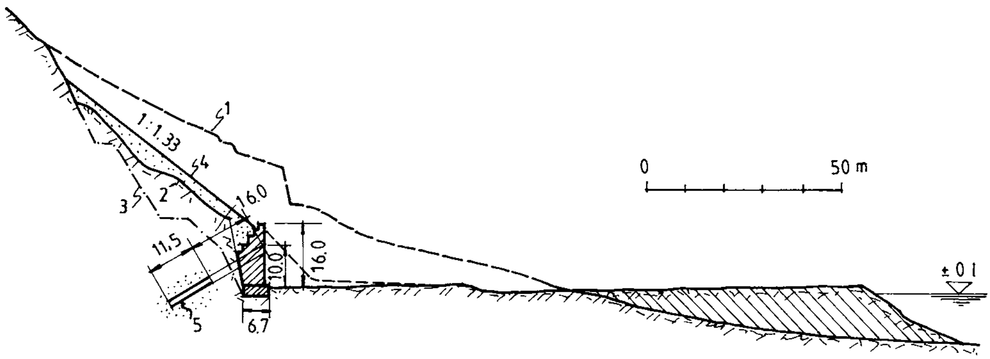


Fig. 8.13 Cross section through the plateau for the cokery at Bakar, 1 original slope, 2 excavated stable cut, 3 excavation for a retaining wall section, 4 stable slope with wall, 5 rock anchors

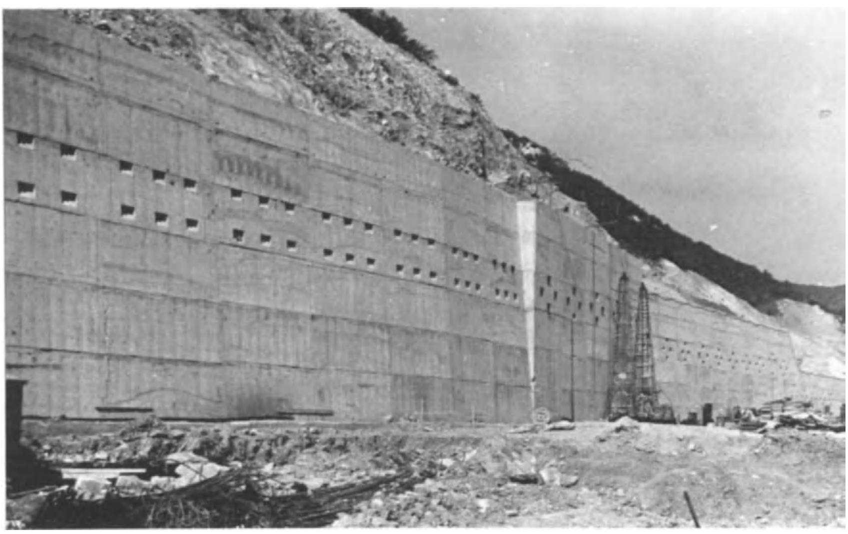


Fig. 8.14 View of the anchored retaining wall on the plateau for the cokery at Bakar

nearly distributed to the far end of the anchor. This confirms, as explained previously, that the fixed length of 11.5 m is more than needed, and that the bearing force of the anchors is much in excess of 1,500 kN.

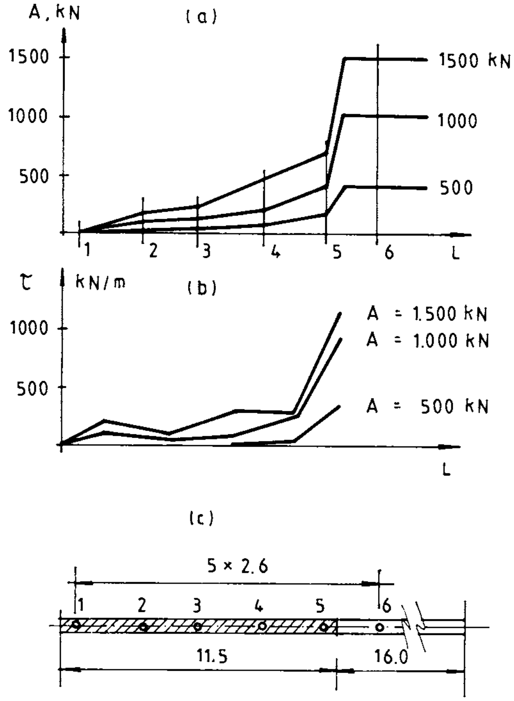


Fig.8.15 Tension test on a rock anchor, (a) distribution of tensile anchor force along the fixed length of anchor, (b) distribution of tangential stresses along the fixed length of the anchor, (c) fixed length of anchor with stress gauges 1 to 6

8.2. Tunnel lining prestressed by grouting

Tensile fissures develop in the concrete lining of pressure tunnels and shafts. They may be narrow and negligible in competent rock and at a low pressure in the tunnel, but generally the lining must be reinforced, so that excessive losses of seepage water are prevented, especially if the surrounding rock is fissured and very pervious. Possible measures to reduce the water loss from pressure tunnels and shafts are:

- construction of steel lining;
- construction of reinforced concrete lining;
- prestressing the reinforced concrete lining;
- consolidation grouting the surrounding rock to reduce its deformability and permeability;
- prestressing the lining by high pressure injections into the surrounding rock.

Standard solutions for a high pressure and weak rock are a steel liner and reinforced concrete lining.

Some prefabricated prestressed linings were used but the construction in the restricted tunnel site was too complicated to have a wide spread use. The example known to the author is the pressure tunnel for the Miocene hydroelectric power station in Italy (Kujundžić, Radosavljević, 1959). The inner diameter of the tunnel is 2.7 m, and the hydraulic pressure is 12 bar. Precast concrete pretensioned elements of the lining were 12 cm thick. The elements were mounted in the tunnel and the space between the elements and the rock was injected with cement mortar at a low pressure. In this case the lining supports the whole hydrostatic pressure.

The reinforced concrete lining, which is usually constructed in pressure tunnels, does not prevent fissures. It only distributes fine fissures along the perimeter of the lining. Concentrated seepage leaks are reduced, but not totally prevented. In fissured deformable rock the total seepage losses may still be important.

Other solutions were proposed to solve this problem by other means of prestressing. Kujundžić (1950) proposed to prestress the concrete lining by means of flat jacks built into the concrete and injected with mortar under pressure to induce compressive axial forces in the lining, as illustrated in Fig. 8.16. The idea was later on implemented in the derivation tunnel of the Rosshaupten HPP by Kunz.

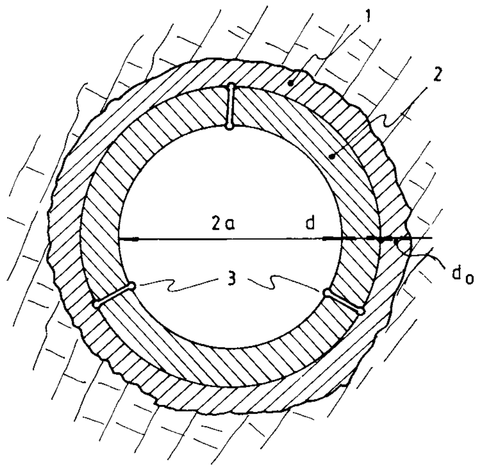


Fig. 8.16 Proposed method of prestressing concrete tunnel lining, 1 smoothing concrete layer, 2 concrete lining, 3 flat jacks injected with mortar (Kujundžić, 1950)

The diameter of the tunnel was 2.20 m, the length 260 m, and the hydrostatic pressure 4 bar. The contractor Kunz patented the solution. It was used also for prestressing the 1,000 m long tunnel of the HEPP Reisach - Reisenleite of 4.90 m inner diameter and a pressure of 25 bar. In this case, prestressing was not achieved as expected, and it had to be supplemented by pressure grouting the rock behind the lining.

Kieser (1950) has developed another solution. The prestressed concrete lining of the tunnel consists of two rings. The excavated irregular rock surface is lined with a plain concrete lining which provides a regular circular profile. An inner ring of prefabricated blocks is constructed on this, with protuberances which leave an annular space between the two linings. When a section 5.0 m long is completed, the annular space is injected with thick grout under 25 bar pressure, which creates a lasting compressive stress in the inner concrete ring. The principle and details of the construction are illustrated in Figs. 8.17 and 8.18. The injected grout is kept under pressure until it has set. This solution has been successfully applied on several tunnels in Austria, FR Germany, Italy and France on a total length of 1,500 m. The largest one was a shaft of 14 m diameter.

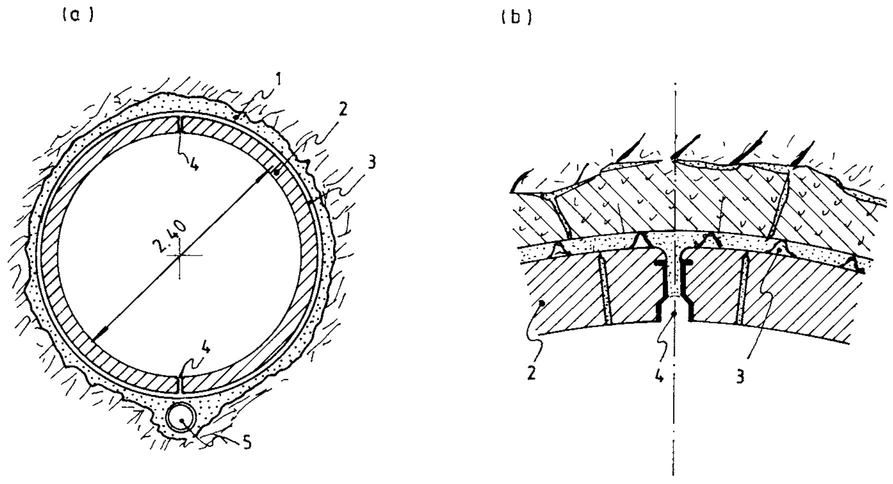


Fig. 8.17 Prestressed concrete tunnel lining after Kieser, (a) cross section, 1 external concrete ring, 2 inner ring of prefabricated concrete blocks, 3 annular space injected with mortar, 4 injection connections, (b) details of lining, 2 prefab.concrete blocks set in mortar, 3 protrusions on block to define annular space around the blocks, 4 injection connection, 5 drain as needed

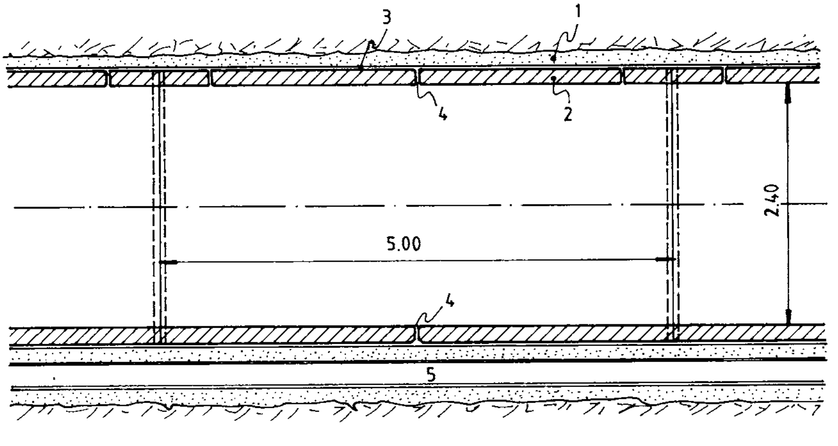


Fig. 8.18 Longitudinal section through a prestressed concrete lining tunnel after Kieser, 1 external lining, 2 ring of prefabricated blocks, 3 injected annular space between the linings, 4 injection connections, 5 longitudinal drain

Lauffer (1968) developed another solution which is not so complicated and expensive as the described ones. It is known in the literature as the TIWAG solution. Instead of a double lining with an annular space as in the Kieser solution, a normal concrete lining is constructed, and the prestressing grout is injected under a high pressure into the specially treated gap between the lining and the rock interface. Some essential details had to be solved in order to make this, in principle very simple idea,operative. The first is to keep the gap open to penetration of the prestressing grout, so that the grout pressure acts uniformly on the hole external surface of the lining. Extensive testing has shown that this can be achieved by painting the excavated rock intrados with a lime slurry after filling the largest irregularities with concrete. Tests have shown that the pressure needed to split the gap with the painted intrados was reduced to 1/7 to 1/3 of that without lime. The other detail concerns uniform penetration of the injected grout all around the interface. The solution consists of fixing plastic pipe rings at about 3.0 m spacing on the rock intrados,as illustrated in Fig. 8.19, before concreting the lining. Holes are drilled on the pipes at regular intervals of about 1.0 m and covered with synthetic sponge sleeves which function as release valves. On the intrados under and over the pipes PVC bands are fixed which ensure contact from the sleeves to the gap for uniform penetration of the injected grout.

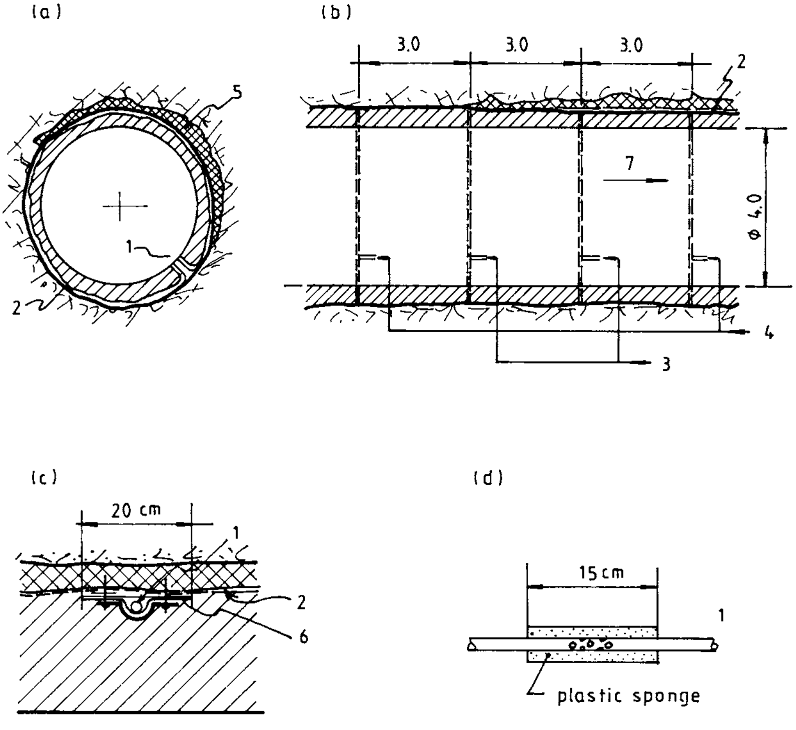


Fig. 8.19 Installation for prestressing the concrete tunnel lining system TIWAG, (a) cross section, (b) longitudinal section, (c) fixing of plastic pipes on the intrados excavation, (d) valves on the pipes, with sponge sleeves, 1 rings of plastic pipes, 2 insulating lime slurry, 3 injection connections, 4 connected to water pressure, 5 concrete filling of large excavation irregularities, 6 PVC bands, 7 direction of prestressing

Prestressing can start from one end in a continuous operation to the other end of the tunnel when the concrete lining has developed sufficient strength. The injection pumps are simultaneously connected to two or three rings of pipes which are injected to saturation. One ring behind and one ahead of the injected rings are connected to water pressure in order to keep the injected grout under pressure until the grout sets, and to prevent penetration ahead of the section being injected. Stable cement suspension of 1C:0.02B:1W is injected, the injection pressure depends on the diameter and thickness of the lining, but it should be higher than the hydrostatic inside pressure of the tunnel.

The TIWAG method of prestressing tunnel linings has been successfully applied for several pressure tunnels of hydroelectric power plants. The pressure tunnel of the HEPP Kaunertal in the Tyrol is 13 km long, the inner diameter is 4.0 m, and the hydrostatic pressure is 13 bar. It crosses weak beds of phillites and schisty gneiss of poor geotechnical parameters on a length of 3.5 km. On this section the TIWAG prestressing method was applied for the first time. The injection pressure was 40 bar, the consumption of cement amounted to 450 kg/m of lining and it was rather uniform along the tunnel. Check holes have confirmed that the thickness of the injected grout was between 1 and 6 mm, in average 3 mm thick. This corresponds to a cement consumption of 100 kg/m of tunnel and the result is that about 350 kg of cement per meter of lining was injected into the fissures of the surrounding rock. This is more than was injected in contact grouting of the tunnel section through sound rock. Thus some improvement of the surrounding rock was achieved in addition to prestressing the lining. The lining was inspected in 1968, four years after the operation of the tunnel started, and it was found that the prestressed lining was free of any fissures, while long horizontal fissures in uninterrupted lengths of lining were discovered in the other section of the tunnel through competent rock. This finding has proved that prestressing was a full success at a price about 5.5% the cost of the prestressed lining.

The TIWAG prestressing method was used in two parallel pressure tunnels of the Drakensberg pumped storage power plant in the South African Republic (Harris, 1982). The tunnels are 5.5 km long, and the dynamic inside pressure reaches up to 62 bar. The horizontal tunnels are in horizontally bedded Triassic sandstone and siltstone, the horizontal stresses in the rock were twice the overburden stress. Details and the construction of the concrete lining are identical to those described for the Kaunertal tunnel. The spacing of the injection pipes was 2.4 m, and the sleeve valves on the pipes were spaced at 1.7 m. The injection started with a thinner cement suspension and it was saturated with a denser one. The pressure needed to open the sleeve valves amounted to 160 bar, and the injection pressure was about 50 bar. The cement consumption amounted to 680 kg/m in one, and to 380 kg/m in the other tunnel, which is 32 kg/m^2 and 18 kg/m^2 respectively of the injected contact area. These examples demonstrate that the "gap grouting" method is very successful for prestressing pressure tunnel linings in order to prevent the development of horizontal fissures and to minimize the related water losses. In the Kaunertal tunnel, the specific water losses were only 0.14 l/s on 1,000 m of lining at a pressure of 16 bar.

The initially mentioned method of prestressing concrete tunnel linings by a high pressure injection of the surrounding rock seemed at first sight to be a simple and attractive possibility, and several attempts were made to implement it. The method was first proposed by Wolfsholz (1924) and Randzio (1927), and it was first realized in 1955 in the HPP Reisach - Rabenleite pressure tunnel which was unsuccessfully prestressed by the Kunz method, as described previously. Through the lining of the tunnel, 3 to 5 m deep holes were drilled possibly perpendicular to the bedding, as shown in Fig. 8.20. Groups of holes were first washed clean by injecting water with additives to soften clayey fill in fissures. Then cement grout was injected under a pressure 2.5 times the inside hydrostatic pressure. Some prestressing of the concrete lining resulted. The cement consumption was 22 kg/m^2 of the lining. According to the report,the seepage water loss after the treatment was negligible.

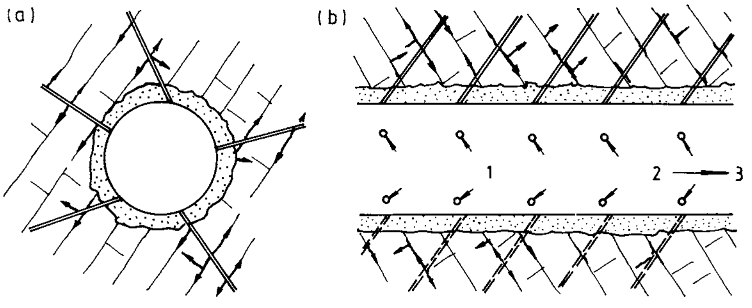


Fig. 8.20 Disposition of injection holes, sequence of washing and injection for rock prestress grouting, (a) cross section, (b) longitudinal section, 1 injected, 2 washed holes, 3 direction of activity

The idea was studied and promoted in Yugoslavia in order to economize on reinforcing steel for pressure tunnel linings. The first to promote the idea were Kujundžić and Radosavljević (1959) who gave practical instructions based on the experience available at that time. They mention the use of expansive cement for grouting, the application of grouting pressure 2.5 times the inner hydrostatic pressure in the tunnel, directing the holes perpendicular to the bedding and simultaneous injection of groups of holes. Some trial works were later explored.

A test chamber was excavated at the construction of the HEPP Rama pressure tunnel in formations of Lower Trias consisting of quartz-sericitic sandstone and clayey sericitic schist (Langhof, 1967, Krmanović et al., 1969). Seismic measurements of rock deformability and measurements with flat jacks have uncovered that the formations were more deformable in the vertical (E_d = 16 MN/m^2) than in the horizontal direction (E_d = 30,5 MN/m^2), the permeability being negligible. The test chamber,excavated parallel to the tunnel axis, was 32 m long, the diameter was 5.0 m, and the concrete lining was 50 cm thick, as shown in Fig. 8.21. During the excavation of the chamber, some outfalls occurred from the arch because of the unfavorable bedding of the rock,as shown in the figure, which were filled with concrete before the lining was constructed. Five measuring profiles were instrumented on the lining for stress measurements in the concrete lining and measurements of radial deformations. In the first run the contact of the lining in the arch was injected with 1C:10S:0.03B:2W grout, the porosity of which was up to 40% after injection, and the volume of injected grout was 85 m^3. Eight holes were drilled in the second run 2.5 to 4.5 m deep in each of 15 profiles along the chamber. Four sections I-IV were separately injected. In every section the lower holes were injected at 5 bar pressure, then all the holes were redrilled and regrouted at a 30 bar pressure. A total of 119 t of cement was injected in the second run, which amounts to 3.7 t/m of the lining.

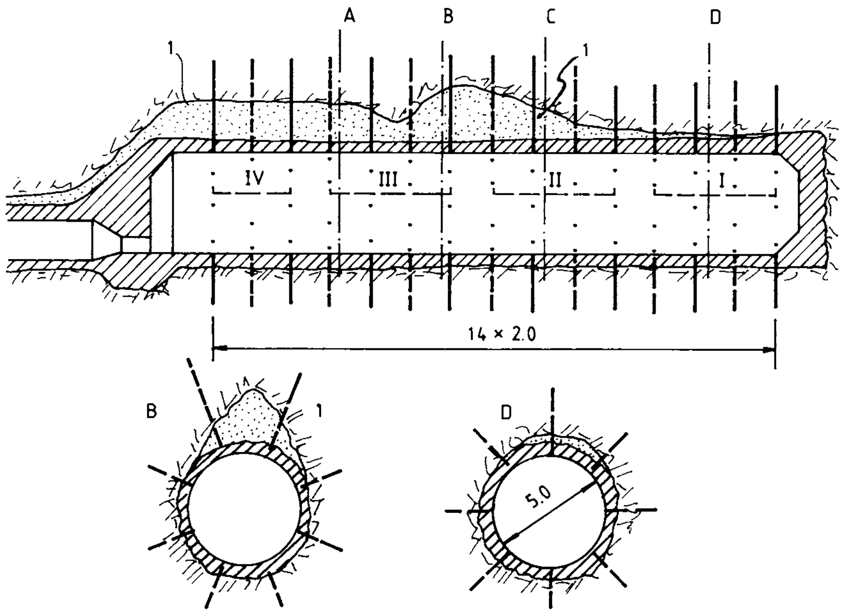


Fig. 8.21 Test Chamber for testing the prestressed tunnel lining by pressure grouting the surrounding rock, 1 rock falls from arch, 2 instrumented measuring profiles A, B, C, D, injection sections I-IV

The grout composition was 1C:0.01B:5-3W with 1% of Intraplast. The stress distribution measured on the lining intrados is presented in Fig. 8.22. The radial compressive stress in the lin-

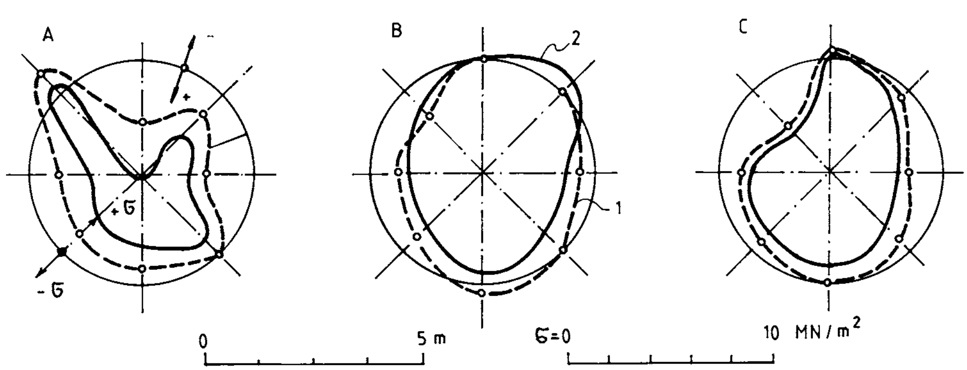


Fig. 8.22 Prestresses measured on intrados of the concrete lining achieved after pressure grouting the surrounding rock in profiles A, B, C of the test chamber, 1 immediately after grouting, 2 stresses 66 days after grouting, ompressive stresses

ing is very irregularly distributed along the perimeter of the lining and a rather high stress relaxation occurred 66 days after the injection. Tensile stresses were also induced,and obviously bending moments developed in the lining.

One hundred days after injecting, the test chamber was filled with water of the same temperature as the surrounding rock and lining, and it was tested under pressure from 2 to 10 bar in four loading and unloading cycles. The measured stresses in the lining and radial deformations are shown in Fig. 8.23.

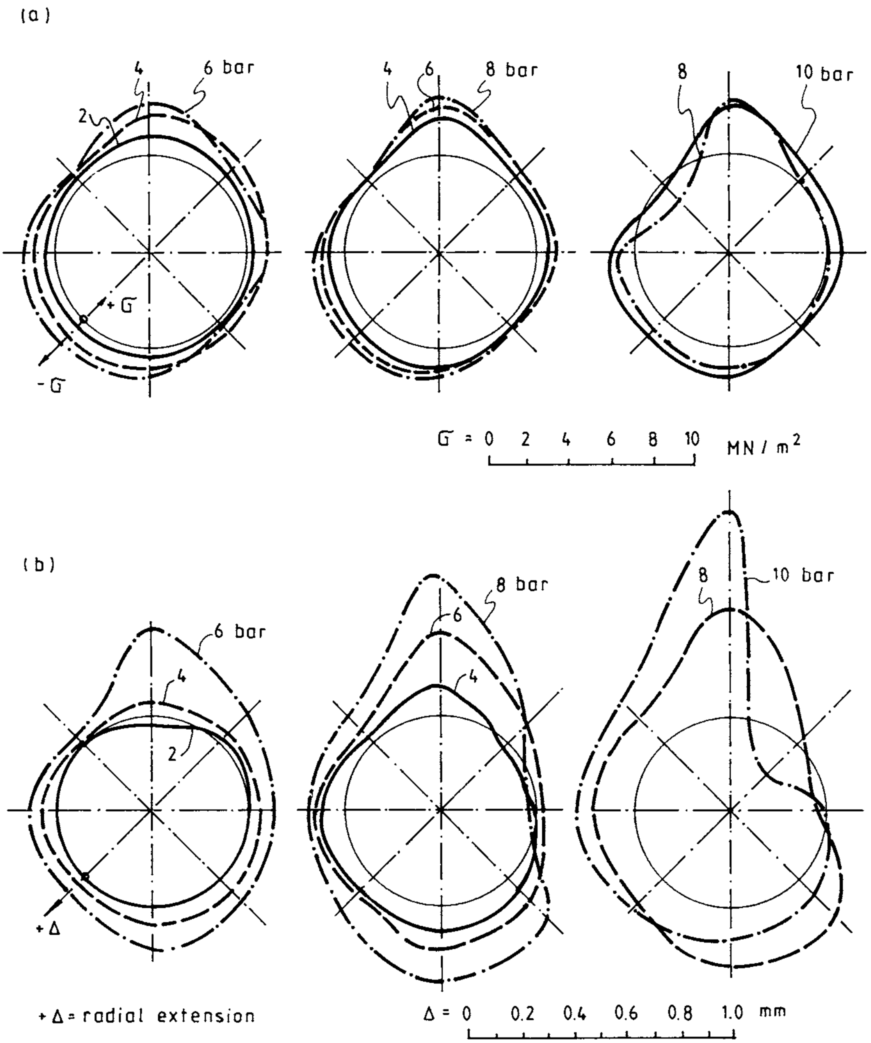


Fig. 8.23 Stresses in the lining and radial deformations in profile A, (a) stresses tangential to the perimeter, (b) radial deformations, 2-10 bar test pressures in three loading/unloading cycles

Evidently the resulting stresses are very irregularly distributed along the perimeter of the lining. The total stresses in the lining resulting from prestress and a hydrostatic inner pressure of 6 bar are shown in Fig.8.24 as measured in profiles A, B and C. Again the stresses are very irregularly dis-

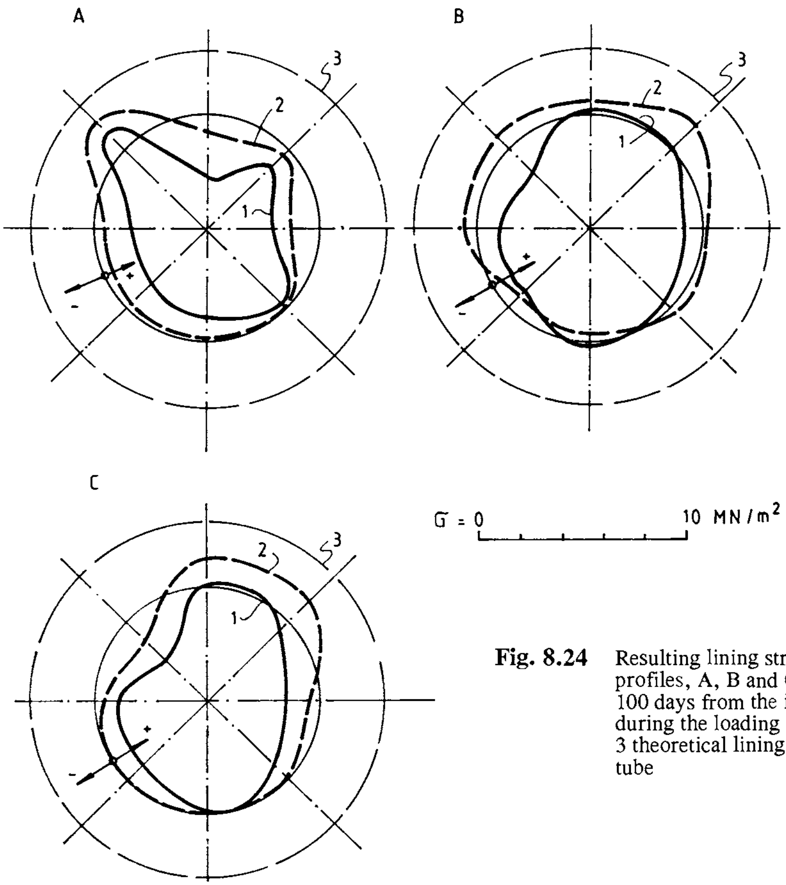


Fig. 8.24 Resulting lining stresses in the measuring profiles, A, B and C, 1 prestress after 100 days from the injection, 2 resulting stress during the loading cycle to 6 bar pressure, 3 theoretical lining stress in the unsupported tube

tributed around the perimeter and they are different in the three profiles. For comparison, the theoretical stress in a free standing tube is also shown, from which it is concluded that prestressing has reduced the tensile stresses in the lining, but regions of tensile stresses are not eliminated.

Another test was carried out in the HEPP Trebišnjica pressure tunnel (Langhof, 1967). The tunnel was excavated in layered Upper Cretaceous limestone with small fissures filled with clay. The dynamic deformation moduli were between E_d = 3,000 and 4,700 kN/m^2. The inner diameter of the tunnel was 6.0 m, and the thickness of the concrete lining was 30 cm. Injections for prestressing the lining were carried out in two phases, the first to fill larger fissures and voids, and the second to induce compression prestress in the lining. The injection pressure was 8 bar in both phases. Stable cement suspensions with some bentonite and with variable water ratios were injected.

Five holes, in rings spaced, by 6 m were injected in the first phase, and in the second phase five holes in rings between those of the first phase. Always one hole, starting from top of the arch down, was injected to saturation. Deformations and stresses in the lining were measured in three sections of the injected length of the tunnel, during the injection and after the tunnel was in operation. It was found out that the results were not satisfactory. Probably higher injection pressures and simultaneous injections of the whole ring of holes should have been carried out.

These documented lining prestressing tests by high pressure injection into the surrounding rock show that a uniform compressive stress can not be achieved. The result, if any, is erratic and unreliable. The rock deformability may be reduced as in consolidation grouting, which also reduces the danger of longitudinal fissuring, so that some reduction of seepage losses from the tunnel may be achieved, but many uncertainties make the method unreliable.

9. LIFTING AND LEVELLING OF STRUCTURES BY GROUTING

The first experiences with lifting and levelling of structures by injection were accrued in the United States about 1930 when it became usual practice to inject cement grout for levelling sagged concrete slabs during highway maintenance and reconstruction works. Grouting was used for the first time in 1953 to level the uneven settlement of the foundation slab of a coking plant and of a hydroelectric power station in West Germany.

Heavy structures which settled unevenly may be levelled in several ways: by softening the ground below a part of the structure which has settled less, by underpinning and widening the foundation on the side which has settled more, and by lifting the structure with hydraulic jacks, but also compaction grouting-injecting thick grout, which increases the volume of soil under the settled foundation, was successfully applied on several occasions in raising the settled parts of the foundation.

Injected grout may penetrate the ground in two ways, depending on the properties of the ground and the grout composition:

- suspensions penetrate the voids of the soil so that it becomes denser and less deformable after the grout sets and gains strength;
- thick grout penetrates the ground displacing the soil, compacts it and lifts its surface if the pressure and volume of the injected grout are sufficient.

Displacement or compaction grouting with thick grout is efficient in all kinds of soil from gravel to cohesive silty and clay soils. Some basic conditions must be satisfied for successful lifting and levelling of structures by compaction grouting:

- the structure must rest on a single foundation and be sufficiently stiff to withstand unavoidable small concentrations of soil reaction;
- injection pressure should not be limited by the danger of damaging the surrounding properties;
- stiff grout must be injected which penetrates into the soil by hydraulic jacking (claquages) and compaction of the surrounding soil in concentrated volume and not by permeating the voids of the soil.

For lifting shallow structures it is convenient to inject grout at a high pressure deeper into the soil where penetration of the grout into the desired volume is easier to check and hydraulic jacking can be effected. Depending on the soil composition, on the depth of the foundation below the ground surface and on other local conditions, it may be necessary to confine the region by constructing a sheet pile wall or a grout curtain around the perimeter of the foundation in order to prevent the useless penetration of the grout into the surroundings of the foundation. Such a grout curtain must not be impermeable. It is sufficient to grout it by split spacing with a pressure of about twice the overburden pressure.

Several possibilities for the levelling and lifting of structures and for improving the ground around foundations are sketched in Fig. 9.1.

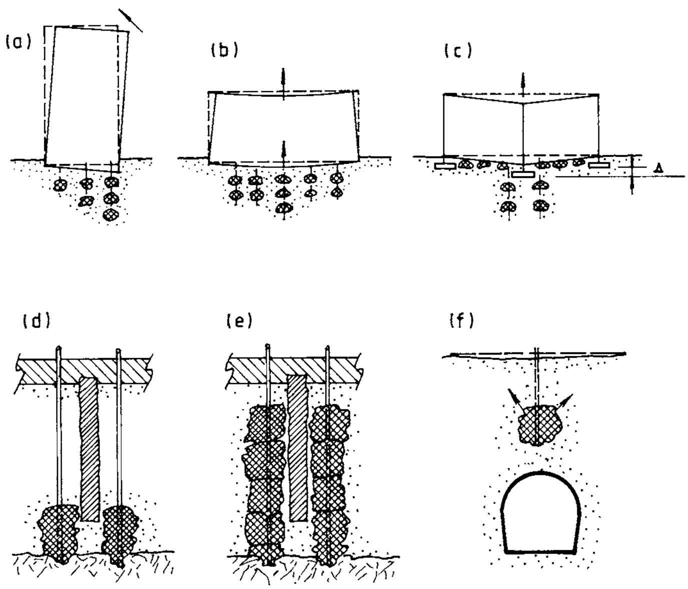


Fig. 9.1 Several applications of compaction grouting for the levelling and lifting of structures, (a) levelling of a leaning building, (b) elimination of uneven settlement of slab foundation, (c) compensation of the differential settlement of neighboring structural foundations, d) compaction of soil around the pile base for improving the bearing capacity, (e) compaction of soil around the pile shaft for increasing skin the friction, (f) lifting subsidence above a tunnel in soil (Warner, 1982)

A detailed exploration of the foundation soil must be available for the design of such grouting projects on which the specifications for the development of the activities are based. The deformation characteristics of the strata and their spatial distribution are of prime interest.

9.1. Theoretical background

The mechanism of grout penetration by displacement into soil is very complex. The injected grout may penetrate the ground by compressing loose soil where the porosity is reduced, or either the surface or the foundation plane are lifted, or these mechanisms occur simultaneously in different proportions. When grout is injected into a cylindrical cavity it first fills it and then expands, whereas when it is injected through the bottom of an injection hole, a spherical volume is expanded and filled with grout. When the critical injection pressure is exceeded the surrounding ground is either compressed or displaced toward any free surface, and the resulting volume is filled with grout and growing as long as injection progresses.

A theoretical solution for the expansion of a spherical and a cylindrical cavity in the center of an infinite half-space of homogeneous saturated clay, the strength parameters of which are $c_u = 0$, was developed by Ladany (1963). For the expansion of a spherical cavity in a normally consolidated clay and in a preconsolidated clay the following results are obtained:

$p = 10\ c_u$ (normally consolidated clay),

$p = 5\ c_u$ (preconsolidated clay), (9.1)

depending on the unconfined strength c_u, or:

$p = 2\ \sigma_z'$ (normally consolidated clay),

$4{,}5\ \sigma_z' < p < 6\ \sigma_z'$ (preconsolidated clay). (9.2)

In these expressions

p = injection pressure,

c_u = unconfined strength,

σ_z' = overburden effective normal stress at depth z.

The same order of magnitude of the pressure **p** applies for the expansion of a cylindrical cavity.

Vesić (1972) has solved the problem for the expansion of a spherical cavity in cohesive Mohr-Coulomb material with strength parameters (c, ϕ) and a volume strain in the plastified zone around the cavity. The strength parameters and the volume strain are obtained from triaxial laboratory tests. The pressure needed to start the expansion of the cavity is:

$$p = c\ F_c + \sigma_z\ (1+2\ K_o)\ F_q/3 \tag{9.3}$$

F_c and F_q are expansion factors which depend on Φ, E, and Θ and they are tabulated in the paper. From an example in the Vesić paper it can be estimated that in a non cohesive soil with $\Phi = 35°$, $E = 600$ bar, $\Theta = 0.012$, $K_o = 0.33$ the expansion pressure amounts to $p = 7.8\ \sigma_z$ and it is a linear function of depth, it increases with the modulus E and decreases with the volume strain Θ. More details can be obtained from the cited papers by Ladany and Vesić. These theoretical results on the critical pressure needed to expand a cavity and inject the grout are informative only and field test are needed for any actual problem.

Lifting and levelling by grout injection has the following advantages:

- no disturbance is caused to the environment and nearby structures;
- any risk for the structural and functional integrity of the structure is minimal;
- the operation may be carried out without any interruption of the structure function;
- there is no need for extensive soil investigations;
- it is usually the most economic alternative.

The limiting factors for its use may be:

- the effect may not be lasting in saturated clay foundations because of the consolidation due to the induced pore pressures;
- the possibility that the injected grout penetrates into the subterranean sewage systems, pipes and other installations or that they become deformed.

The best results are achieved in silty sands of low relative density. Clean sand and materials of high relative density require a higher pressure because they have a higher shearing strength.

9.2. Grouting compounds

For compaction grouting the best results in the levelling and lifting of structures are achieved with mortar grouts of very thick consistency prepared with well-graded sand not coarser than 2 mm, containing some 2% of silt finer than 50 μm. Sands containing a larger proportion of clay is not suitable because it may shrink when injected. Mixes of about 0.1C:0.9S are used. Up to 50% of cement may be substituted by pozzolana in order to improve the plasticity and pumpability of the mortar. Water is added in such a quantity that a slump of 50 mm is obtained (see Fig. 9.2). In some cases, instead of cement, fly ash (FA) and hydrated lime in proportions of 1FA:2S to 1FA:10S were successfully used.

Fig. 9.2 Consistency of mortar for compaction grouting

9.3. Grouting procedures for compaction grouting

Injection holes are drilled through the foundation slab of the structure. If the construction or the purpose of the building require it, sloping injection holes can be drilled from the perimeter, of the

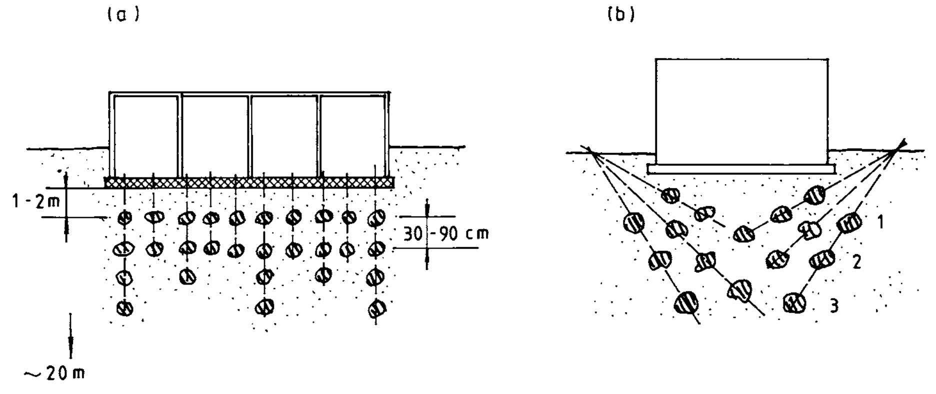


Fig. 9.3 Disposition of holes for lifting the foundation, (a) through the foundation slab, (b) inclined holes from the perimeter

structure,as shown in Fig. 9.3. The holes in the ground may be drilled and cased, or the casing may be driven into the ground with a lost point. Drilling and grouting may be from the bottom up, which is cheaper, but it may have the disadvantage that the casing does not seal sufficiently to prevent the grout from escaping to the surface. Drilling and injecting from top to bottom is more efficient in many cases.

Spherical grout bulbs for lifting or levelling structures are injected through the bottom of cased holes, starting from the top and proceeding downwards. The injection is stopped at every bulb when the required volume of grout is injected. The penetration of thick grout may be facilitated when it is preceded by an injection of water with some detergent or surface active additive somewhat reducing the shear strength of the ground around the injected space. The injection of a lime slurry may prove effective in clayey ground.

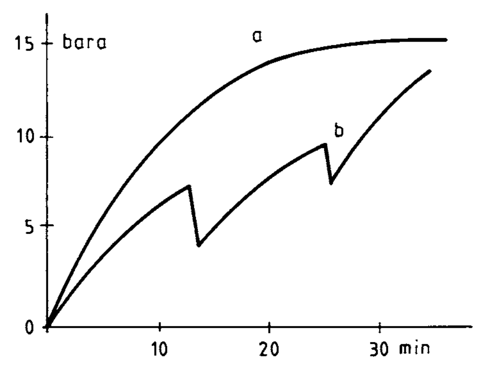


Fig. 9.4 Pressure increase diagrams, (a) in a homogeneous soil, (b) in ground with a weaker layer

A high pumping rate is convenient for lifting the foundations. The injection is initiated with a pumping rate of some 300 l/min, and it is kept constant until the specified volume is injected or the required amount of lifting is reached. Lower pumping rates are advisable when some compaction of the ground is required. The rate of pressure increase at the start should be about 0.3 to 0.5 bar/min, and it can be regulated by the rate of pumping. The pressure development during the injection reflects the homogeneity of the ground, as illustrated in Fig. 9.4. The pressure increase is gradual in homogeneous ground, and it may suddenly decrease when the grout begins to penetrate a weaker layer. When it is necessary to compact the ground, as well as levelling or raising the structure, a slow pumping rate in the range of 3 l/min in cohesive and up to 10 l/min in permeable soils is applied, until the required volume is injected or the amount of lifting is reached. It has been observed that a greater quantity of grout can be injected with a slow pumping rate in order to obtain the same amount of lifting as with a high pumping rate, the difference being a larger compression of the surrounding ground. Cohesive soils can not be efficiently compacted by compaction grouting, because of the time lag due to the dissipation of the induced pore pressures. Some good results can be achieved by the installation of vertical drains around the injected bulbs. It is advisable to install pore pressure gauges at the injection locations, because the dissipation of pore pressures causes some settlement of the lifted foundation, which can be computed and compensated for by higher lifting. It is also necessary to interrupt the injection when pore pressure reaches the overburden, pressure and to resume the operation after it has sufficiently dissipated.

In some cases, when the compaction of the ground below the building is not necessary, lifting and levelling can be achieved by injecting mortar just below the foundation slab. Injection holes are drilled at regular spacings of 2 by 2 m, or 3 by 3 m, to about 10 cm deep into the ground, and injection pipes are set in mortar. The holes are first flushed with water in order to facilitate the penetration of the grout. The injection starts at the region of maximum settlement. When lifting starts, the following holes are connected to the pump and injected until the required lift is achieved. If a large amount of lifting is required, the work can be done in stages. In such cases, the consistency of the grout is more plastic than when injecting into the ground; even more water is added in the last stage in order to regulate better the level of the slab.

9.4. Examples of the lifting and levelling of buildings

The application of grouting for the lifting and levelling of heavy structures and buildings will be illustrated with some typical examples.

The Emil Coking Plant in Essen

Over many years of operation the 18 m wide and 34 m long foundation slab of the coking plant had unevenly settled,so that the production was threatened to close down. The layout of the slab and the equal settlements contours are shown in Fig.9.5. One end of the slab settled more than 100 mm and the slab deformed about its middle section. A thick cement bentonite thixotropic suspension was injected through the holes in the foundation slab in successive small portions.The slab was slowly

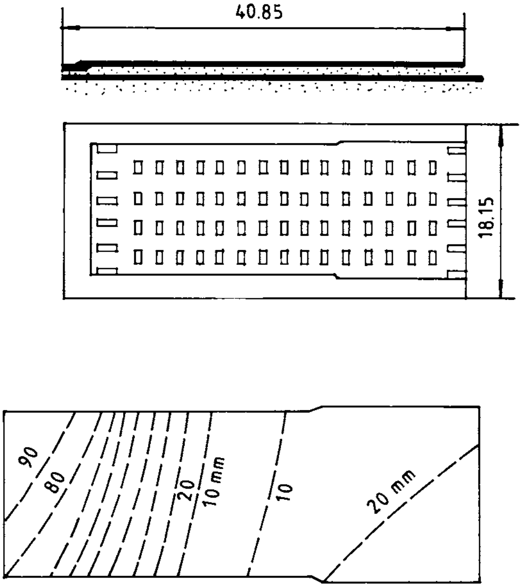


Fig. 9.5 Layout and section of the foundation slab of the Emil Coking Plant at Essen and contours of equal settlements

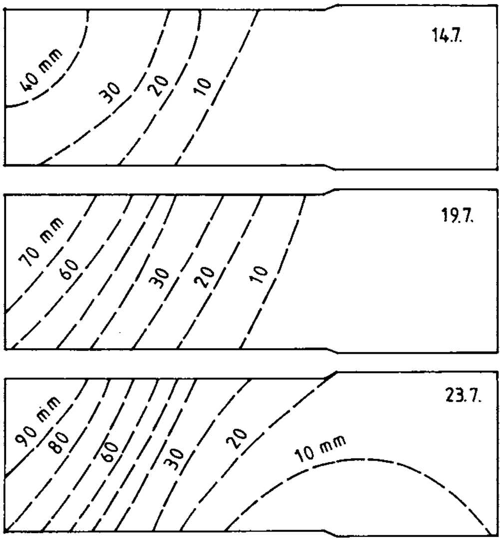


Fig. 9.6 Three successive phases of lifting the foundation slab of the Emil Coking Plant with contours of equal settlements

raised, as shown in Fig. 9.6, from the left end to the middle. The left end of the slab was raised over nine days to its design elevation. A total of 80 t of cement and bentonite was injected, and the operation took about three weeks, with no interruption in coke production.

Hydroelectric Power Plant Hessigheim

The foundation of the Hessigheim hydroelectric power plant on the Neckar in West Germany settled unevenly with a maximum difference along the diagonal of 10 cm, as shown in Fig. 9.7, which caused difficulties in the operation of the turbines. After the study of several possible remedial measures,it was decided to try to level the foundation of the building by injections. A sheet pile wall was driven around the slab in order to confine the grout below the foundation slab. Injection holes

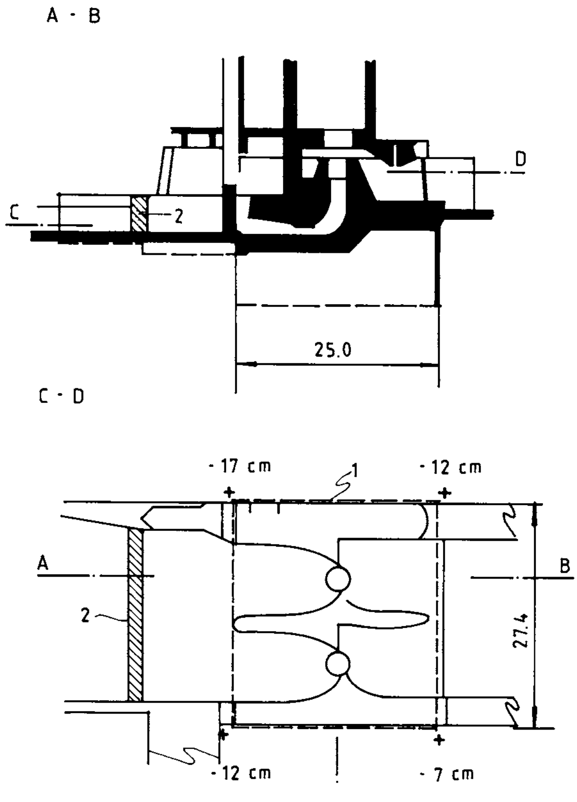


Fig. 9.7 Layout and section of the Hessigheim hydroelectric power plant, 1 sheet pile wall, 2 cofferdam, (Pleitner and Bernatzik, 1953)

were drilled through the slab to the contact plane with the ground. Thick stable cement suspensions were injected, first at a low pressure in order to detect and repair leaking parts of the sheet pile wall. Then grout was injected at a high pressure selectively, first to level the slab and then to lift it as a whole to its original elevation. The whole operation took about three months, and a total of 900 t of cement was injected. It should be mentioned that the theoretically required volume of grout was only 150 t of cement.

Leaning grain elevator

The grain elevator near Goodland, Kansas, USA started leaning when it was built in 1923, and in 1946 it was about 90 cm out of plumb, leaning toward the railway line (King and Bindhof, 1982). The elevator 35 m high and 9 m in diameter, rests on a shallow slab and it was decided to level it by grout injections below the slab.

For this purpose the slab was widened by 2.0 m to the side which had settled more. The added section was heavily reinforced and tied by additional bars grouted into the holes extending some 6 m into the foundation slab. Buttress walls were added to help transmit the lifting force from the slab to the cylindrical silo structure, as shown in Fig. 9.8, and the end of the extended slab was deepened by 75 cm in order to prevent leakage of the injected grout. The soil below the slab which had settled less was moistened to facilitate deformation during the rotation of the structure. The injected

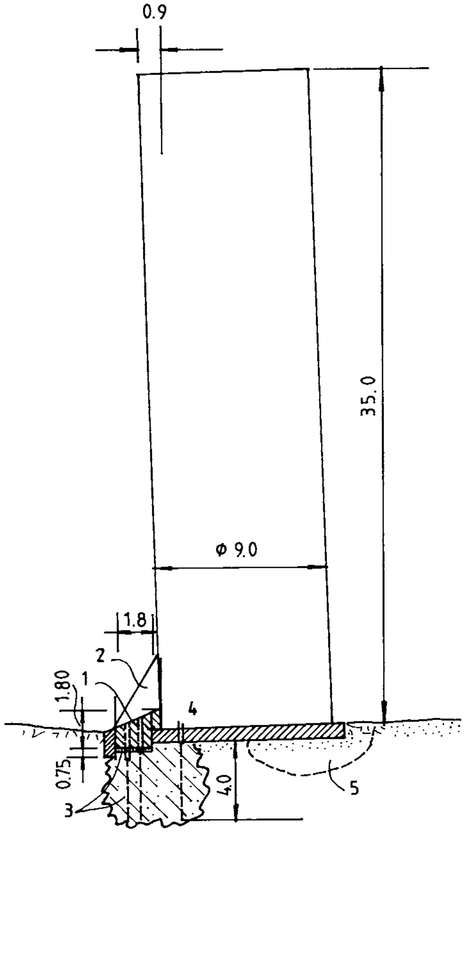


Fig. 9.8 Leaning grain elevator in Goodland, 1 extension of foundation slab, 2 temporary buttresses, 3 gravel layer, 4 injection holes, 5 moistened soil section

grout was composed of 0.41C:0.34FA:0.25W with some fluidifier and air entrainer. The elevator was levelled in five phases during 10 days. In the extended part and within the elevator slab six holes 38 mm in diameter, were drilled into the layer of coarse gravel. Injection started on two holes until it emerged from the other holes and the gravel layer became gradually saturated. Then all holes were connected to the injection pump and the pressure was increased to 14 bar until the overhang of the elevator was reduced by 27 cm. Then additional holes were drilled in the extension of the slab, and within the elevator into the foundation soil. The contact of the slab with the ground was first flushed with water before it was injected with a pressure of 21 bar at which the grout started penetrating into the contact plane. After the process was repeated a few times the vertical position of the elevator was restored. A total of 96 t of grout compound was injected.

Leaning silo cells

The grain silo at Dubuque, Iowa, USA consists of four cells on a common square foundation slab which is 75 cm thick. The foundation soil was not well explored. While driving the casing of injection holes, these were only indicating that it contained a weak layer, as shown in Fig. 9.9. When

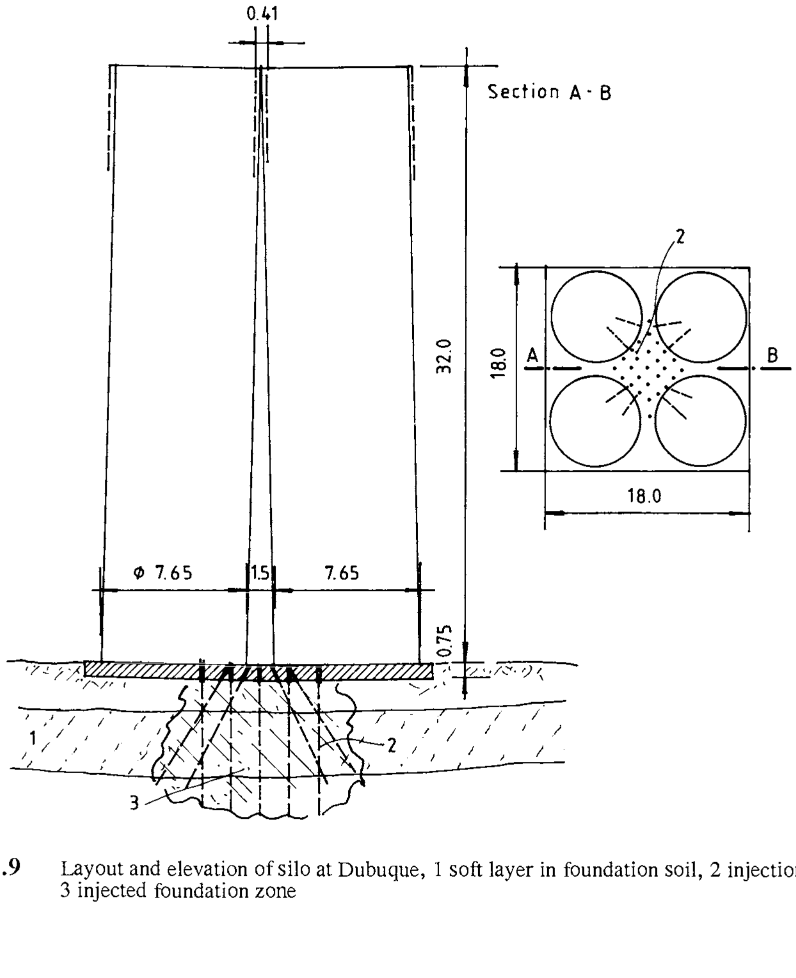


Fig. 9.9 Layout and elevation of silo at Dubuque, 1 soft layer in foundation soil, 2 injection holes, 3 injected foundation zone

the load of the silo reached about 25 % of the total during the first filling it was observed that the tops of cells started converging because the foundation slab settled more in the middle than at the rim. After subsequent seasonal loading with about 3,500 t of grain, the tilt increased to touching the tops of two cells which were originally 60 cm apart. It was then decided to inject grout into the foundation soil in order to lift the center of the slab. Injection holes were drilled through the slab and the casing was driven to about 8 m deep where injection started and it was continued in sections upward to under the slab. The injected grout raised the central part of the slab and the operation was ended when the touching cells were again 41 cm apart. The grout consisted of 1C:1FA:3.2S:0.8W with some plastifier added, and about 105 m^3 of grout compound were injected with pressure up to 42 bar. Out of this volume of grout 10 m^3 went into the raised volume, and the rest probably to displace and compact the ground. The work was carried out in 1975 (King and Bindoff, 1982).

Mine hoist headframe

The hoist headframe for the 900 m deep shaft of the copper mine at Tooele (Utah, USA) founded on a 24 m by 16 m slab, 1.4 m thick, settled unevenly so that the top of the headframe tipped by 23 cm to the north and 11 cm to the east, enough to impair the operation of the skips in the shaft. Injection of the grout was selected to level the hoist frame. A peripheral and inner grout curtain was first injected in order to confine the lifting injections to the region of the slab which settled more, as shown in Fig.9.10. Some 460 m^3 of cement/sand grout was injected in the curtain and some si-

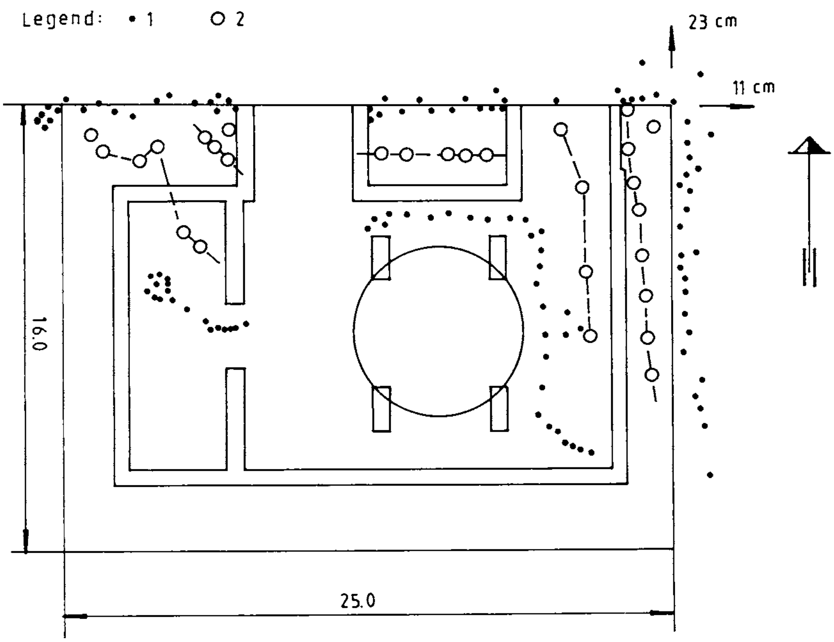


Fig. 9.10 Plan fo foundation slab hoist headframe, 1 injection holes for the grout curtain, 2 injection holes for lifting the slab

licate grout at certain locations. Then the lifting holes were injected in two phases. In the first phase of lifting the ground was injected 10 to 11 m deep under a pressure of 7 and 14 bar. Then the zone from 10 to 6 m deep was injected under a 28 bar pressure alternately at the south and the west side of the foundation slab. The grout consisted of 1C:0.5 pozzolana: 0.6W and additives for fluidification. A total of 530 m^3 of compound was injected.

The top of the frame was constantly surveyed during the lifting operation, the result is shown in Fig. 9.11. The headframe was straightened by 20 cm to the north and by 10 cm to the east and the troublefree operation of the skips was reestablished.

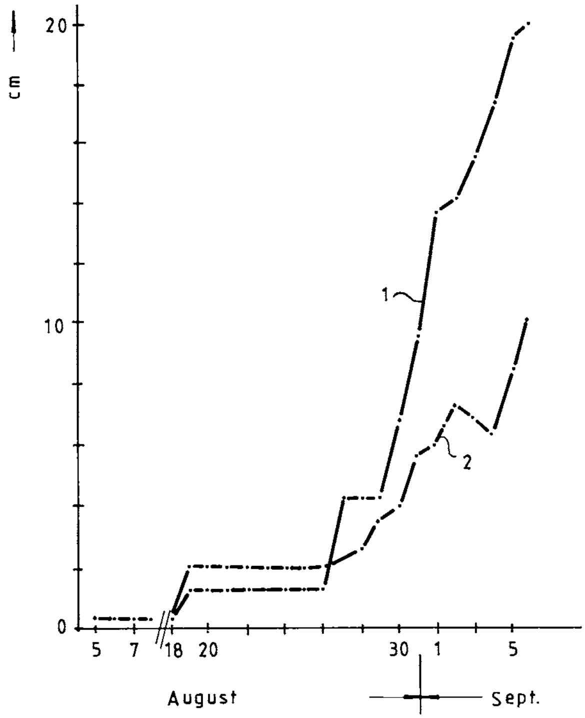


Fig. 9.11 Straightening of the SW top edge, of the hoist headframe, 1 movement to South, 2 movement to West

Foundation of Ninian Platform in the North Sea

The precast reinforced concrete structure of the Ninian Platform near the Shetland Islands in the North Sea was launched shipped to the site. It was sunk onto the prepared foundation bed 136 m below the sea level, and a tight contact between the slab and the ground was established by injecting mortar (Anonymous, 1978). A cross section of the structure is shown in Fig. 9.12 and a layout of the foundation slab with partitions for grouting in Fig. 9.13. The bottom of the slab is divided by four concentric and eight radial training walls into 33 sections. The training walls penetrated the sand foundation under the weight of the structure and provided resistance to horizontal forces. Each section could be injected individually. A free space about 40 cm high remained after the training walls penetrated into the foundation, and it had to be filled with grout in order to achieve a tight contact between the structure and the foundation material, which was able to transmit the self weight of the structure, wave and other forces safely into the foundation soil.

After extensive laboratory tests, a grout suspension consisting of cement and fly ash suspended in sea water was selected for grouting the space.

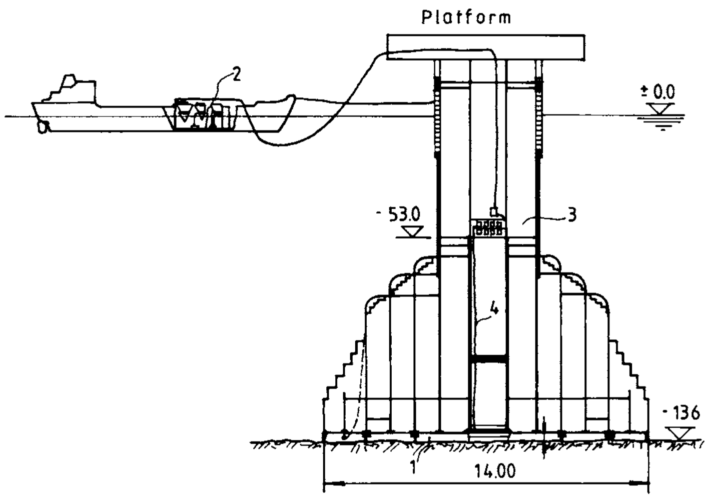


Fig. 9.12 Cross section of the structure of the Ninian platform, 1 space below the foundation slab filled with the injected grout, 2 ship with weight batching and injection equipment, 3 additional mixing and pumping equipment, 4 pipes for connection to injection sectors

After the platform was set on the ground and ballasted for stability and penetration of the training walls, a 20,000 deadweight ship with all the necessary materials and equipment for preparing, mixing and pumping was anchored and moored near the platform. The pumps on the ship, connected to the platform by means of a 50 mm Dia pressure hose, delivered the grout mixes to a secondary mixing plant which fed the injection pumps in the central shaft of the platform at 53 m below the sea level in order to compensate for the hydrostatic pressure at the bottom of the foundation slab. Eight piston pumps for 10 bar pressure at high capacity and 40 bar at low capacity delivered the grout to eight sectors of the slab which were simultaneously injected.

Eight sectors were simultaneously injected, as shown in Fig. 9.13, in each of the four phases of grouting. As the sectors were grouted, sea water was drained out by special outlets which were closed when diluted grout started to flow out. Then injection was resumed under a 10 bar pressure in the central sectors and a 10 to 20 bar pressure in the peripheral sectors of the slab. While injecting the

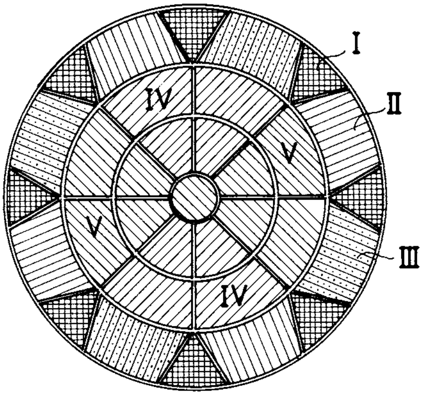


Fig. 9.13 Plan of foundation slab of the Ninian platform with training walls dividing slab into 33 injection sectors, I-V simultaneously injected sectors

external sectors, divers controlled the tightness of the seal along the peripheral training wall. During 7 days of continuous work, all the sectors were injected and a total of 4,900 m^3 of grout was consumed. Considering the importance of completely filling all the injected space all over the foundation base for the permanent safety of this outstanding structure, all phases of the work were rigorously checked and documented, grout samples tested as a basis for the final acceptance of the completed platform.

10. SYMBOLS

The SI system of units and symbols is consistently used in the book. For practical reasons, as predominate in the grouting profession, the unit bar, tolerated in the SI system, is used for fluid pressure instead of Pa.

In order to simplify and unify the definition of grout mixes and suspensions, their composition is described exclusively for a weight unit of mix and the weight of water on one weight unit of compounds.

The following symbols are used for the definition of the compounds:

C for cement,

CL for clay,

B for bentonite,

A additives,

FA for fly ash,

S for sand.

The mixing ratio is written as:

1C:1W for a one to one cement suspension in water, or 0.2C:0.5CL:0.02B:0.28S:3W for a complex mix consisting of 0.2 parts of cement, 0.5 parts of clay, 0.02 parts of bentonite and 0.28 parts of sand, yielding 1.0 part of compound on 3 parts of water, all by weight. Additives are defined as a weight percentage to the weight of cement or clay or both as specified.

The symbols used for physical magnitudes are:

A	area	m
c	cohesion	kN/m
c_f	shear strength parameter	kN/m
c_p	unit for viscosity, centipoise	10^{-3} N s m
E	modulus of elasticity	kN/m^2; MN/m^2
E_d	dynamic modulus of elasticity	kN/m^2; MN/m^2
h	hydrostatic head difference	m, cm
i	hydraulic pressure gradient	-
IP	plasticity index	%

K	earth pressure coefficient	-
k	permeability coefficient	cm/s
L, l	length	m, cm
LU	Lugeon unit for rock permeability	lit/m min 10 bar
N	number of Lugeon units	lit/m min 10 bar
p	pressure	kPa, bar
Q	flow volume	m^3, cm^3
q	flow rate	m^3/s, cm^3/s
V	volume	m^3, cm^3
W	mass	g, kg, t
W	force	kN
w	moisture content of soil	% of dry weight
w_l	liquid limit	%
w_p	plastic limit	%
γ	unit weight	kN/m^3
θ	specific volume strain	%
ν	viscosity coefficient	kN s/m
λ	linear shrinkage	%
η	kinematic viscosity	m/s
ρ	unit mass	g/cm^3
σ	normal stress	kN/m^2
τ_f	shear strength	kN/m^2
τ_0	treshold shear resistance	kN/m^2
ϕ_f	angle of shear strength	°
Φ	potential function	-
Ψ	seepage function	-

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Notations:

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GDMEP - Symposium on Grouts and Drilling Muds in Engineering Practice, ICE, London;

ICISRM - International Conference Int. Society of Rock Mechanics;

ICOLD - International Conference on Large Dams;

ICSMFE - International Conference on Soil Mechanics and Foundation Engineering;

JDMSPR - Yugoslav Society of Rock Mech. and Underground Works;

JDMT - Yugoslav Society of Soil Mechanics and Foundation Engineering;

JGE ASCE - Journal of geotechnical Engineering, American Society of Civil Engineers;

JSMD ASCE - Journal of soil Mechanics Division, ASCE;

JUSIK - Yugoslav Symposium on Ground Injections;

JUCOLD - Yugoslav Conference on Large Dams.

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12. GLOSSARY AND INDEX

12.1. Glossary

In the branches of drilling and grouting a professional jargon is customary some terms of which may be difficult to understand to outside readers for whom the book is also intended. It is therefore deemed useful to add a few explanations of words and terms which appear in the text. Here follows a list:

Absorption - penetration of grout into fissures or voids in the ground,

Additive - substance added in small quantity to grout mixes in order to improve their properties,

Adsorption - bond of water molecules or ions to the surfaces of soil particles,

Batching - weighing and proportioning the ingredients of grout mixes,

Circuit grouting - grouting a section of hole with return line which conveys surplus grout to the mixer in the grouting station,

Claquage - jacking up fissures in the ground by high injection pressure,

Clay concrete - concrete in which part of gravel and sand is replaced by clay,

Coagulation - forming of clusters of particles having different electric charges suspended in water,

Core - a sample removed from the ground by drilling,

Core barrel - tube at lower extremity of drilling rod with coring bit at its end by means of which the core is extracted,

Diaphragm wall - continuous wall excavated into soil under protection of fluid pressure, filled with concrete, reinforced as needed,

Drilling - process of boring holes in rock and soil for exploration or for grouting,

Drilling rig - machine device for drilling holes into rock and soil equipped with regulating mechanisms of rotational speed, pressure on the bit, clutches and hoists,

Drilling rod - steel tube consisting of 1.0 to 3.0 m long coupled sections rotated by the drilling rig, with the coring or noncoring cutting bit at the end,

Emulsion - disperse system of two liquids which do not mix together,

Flush - to wash out a hole by a rush the water pumped through the drill rods during drilling, or by water after drilling,

Gel - a jelly like material formed by the coagulation of a colloidal liquid,

Gel time - time during which a colloidal liquid or resin prepared for grouting remains liquid before gelling,

Grouting gallery - a concrete gallery constructed on foundation level, excavated below the dam foundation or in the abutments from holes are drilled and injected,

Idle drilling - drilling of sections of holes through the ground above the level of grout curtain,

Inject - to force a suspension or fluid by pumping into fissured or voids of the ground,

Lugeon Unit - measure of permeability from water pressure test in section of bore hole,

Mortar - mixture of cement, sand and additives with water in stiff or plastic consistency,

Packer - seal fixed at end of injection pipe in order to prevent injected grout from rising in the hole,

Percussive rotary drill - drilling rig with noncoring bit at end of drill rod which cuts the material under rhythmic blows of a hydraulic or pneumatic hammer,

Piezometer - cased bore hole with previous section at bottom sealed toward the casing, from which narrow standpipe leading to surface or pressure transducer is placed for recording ground water level,

Rotary drill - drilling rig with rotating coring or noncoring bit at the end of the drilling rod,

Section - length of hole being individually grouted,

Suspension - a two phase system consisting of finely divided solid particles dispersed in a liquid,

Thixotropy - property of suspension of particles of equal electric charge to develop weak mutual bonds resulting in reversible gel when at rest and become liquid when shaken,

Wing curtain - extension of grout curtain into the abutments.

12.2. Index