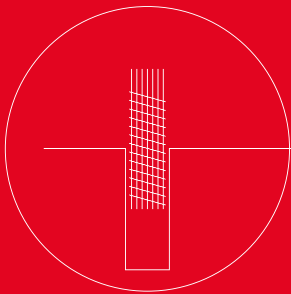
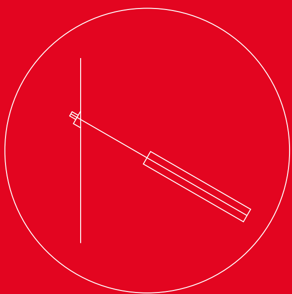
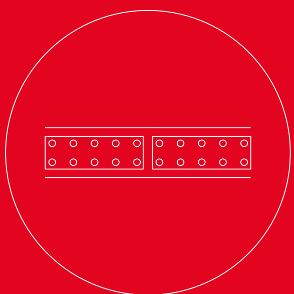
Technical guide







Build on us SOLETANCHE BACHY

TECHNICAL GUIDE

SOLETANCHE BACHY

INTRODUCTION

The previous edition of the Soletanche technical guide dates back to the 1980s!

Project owners, engineers and design offices have long been calling for a new edition and it is certainly true to say that most of our techniques and equipment have changed quite considerably over the years.

A number of processes, such as compaction grouting, compensation gouting and soil mixing have changed to such an extent that completely new chapters have been required, while the advances in electronics have led to substantial changes in our control systems. Soletanche Bachy experts have given this guide a thorough review.

It gives us great pleasure to present you with this new edition and we hope it will feature prominently on your bookshelves.

The authors

Contents

RETAINING STRUCTURES

1. Definitions

A retaining wall is defined, for the purposes of this chapter, as a slim vertical structure used for ground retention during excavation. Unlike gravity retaining walls, the weight of a slim retaining wall has little effect on its capacity to balance the pressures. A slim retaining wall acts like a series of juxtaposed vertical beams holding back the pressures exerted by the soil, water and existing structures. It is supported by struts or ground anchors and is embedded into the ground below excavation level, in order to:

- mobilise the passive pressure at the toe,
- allow the wall to support vertical loads if necessary,
- Provide hydraulic stability (for which a continuous wall is required).

There are two main types of retaining walls:

- continuous walls: diaphragm or precast walls, sheet piling, slurry trenches, secant piles, etc.,
- discontinuous walls: Berlin and Parisian-type walls, Lutetian and Moscow-type walls, contiguous piles, etc.

The vertical structural components which provide the flexural capacity of the wall are installed before the excavation work commences and therefore do not interfere with the earthworks.
The temporary supports (and sometimes permanent supports), and lagging in the case of discontinuous walls, are installed during earthworks. This needs to be considered in the construction sequence.

2. The different types of retaining structures

2.1. Discontinuous walls

Discontinuous walls include:

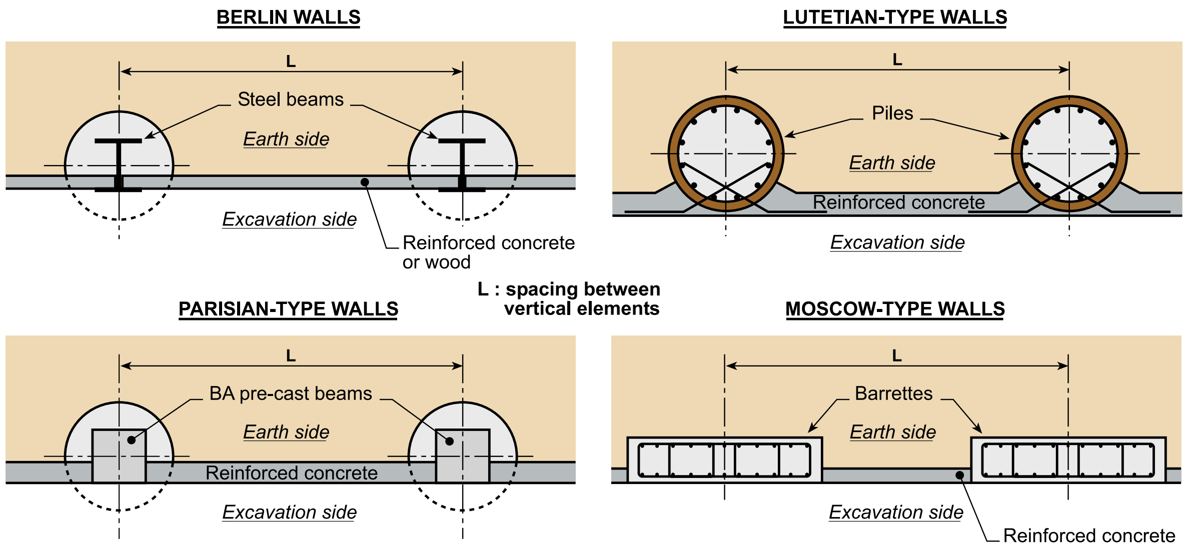
- regularly-spaced deep foundation elements (piles, micro piles, barrettes) that form the rigid vertical structure. These elements are installed before the start of earthworks,
- shoring between these elements, hence transferring the soil loads to them. The lagging is installed in successive phases during the excavation process and often includes a drainage system to prevent the build up of water pressure.

The following conditions must be satisfied during excavation and the lagging phase:

- there must be no significant flow of water through the ground,
- the ground must be able to stand vertically until the lagging is installed.

The table below summarises the various types of discontinuous wall:

Name	Vertical elements	Shoring
Berlin-type wall	Steel sections	Wood, shotcrete or cast-in-situ concrete
Lutetian-type wall	Bored piles	Sprayed or formed concrete
Parisian-type walls	Precast piles	Sprayed or formed concrete
Moscow-type walls	Barrettes (elements of diaphragm walls)	Sprayed or formed concrete




MONACO - Minerve


FRANCE - Sèvres

2.2. Continuous walls

2.2.1. Diaphragm walls

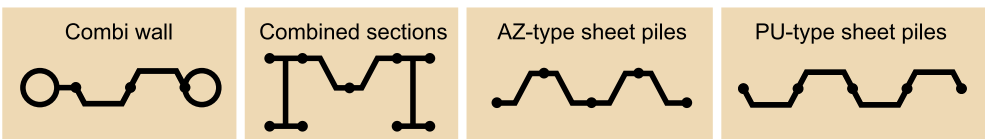
See the chapter on diaphragm walls.

2.2.2. Sheet pile walls and associated techniques

These walls are built using vertically interlocking metal sheet piles. They can be installed in three ways, depending on the soil and the environment:
- driving,
- jacking,
- vibro-driving.

The vibrations caused by driving can restrict the use of sheet pile walls in urban settings, except when special static driving methods (jacking) are used.

Boreholes can be pre-drilled to decompress the soil and facilitate the installation of the sheet piles.



For hard soils, it is also possible to install a sheet pile wall in a trench of bentonite cement slurry, excavated using a grab (see the section on reinforced slurry walls).

The walls can be made of hot-rolled (PU profiles, for example) or cold-rolled sheet piles.

Composite walls can also be installed, including H-type profiles and sheet piles. Other examples include circular or rectangular sections combined with conventional sheet piles.

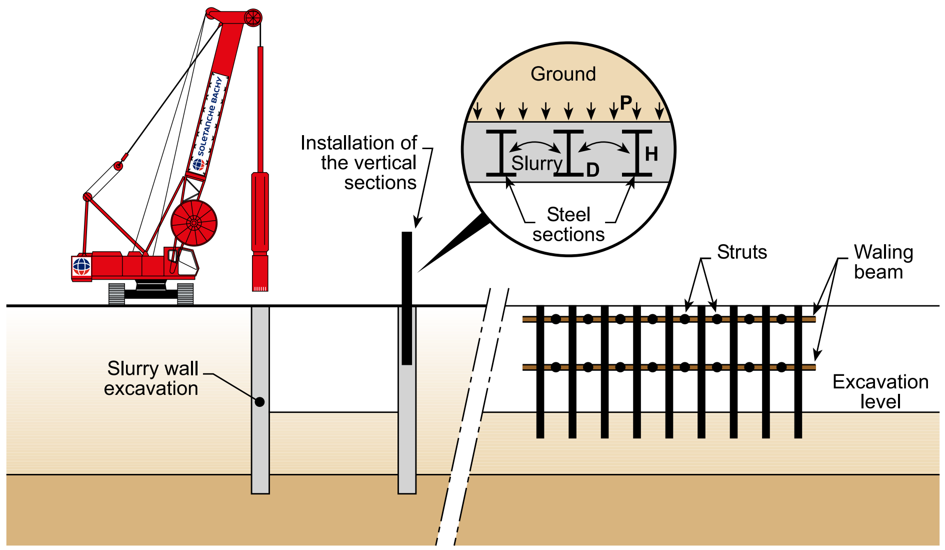
In certain cases, the clutches in the sheet pile walls require additional sealing as they are not generally watertight.


URUGUAY - M'Bopicua


FRANCE - Port 2000

2.2.3. Reinforced slurry wall

This technique lies between the Berlin and diaphragm walls insofar as it provides a temporary but watertight wall. Trenches are excavated under a bentonite cement slurry, using a tool such as a grab, in the same way as for a diaphragm wall. Vertical steel sections or sheet piles are lowered into the fresh, liquid slurry. Where non-continuous steel sections are used, the arching effect through the slurry will allow the pressure exerted by the ground and water to be transferred to the sections.
With sheet piles, the wall acts as a conventional conti-nuous wall.




SLOVAKIA - Zilina

GERMANY - Leipzig Burgplatz

2.2.4. SOIL MIXING wall: TRENCHMIX® and GEOMIX®

These techniques involve the in-situ mixing of the soil with a binding agent. The mix can be reinforced with steel beams. See the chapter on soil mixing.

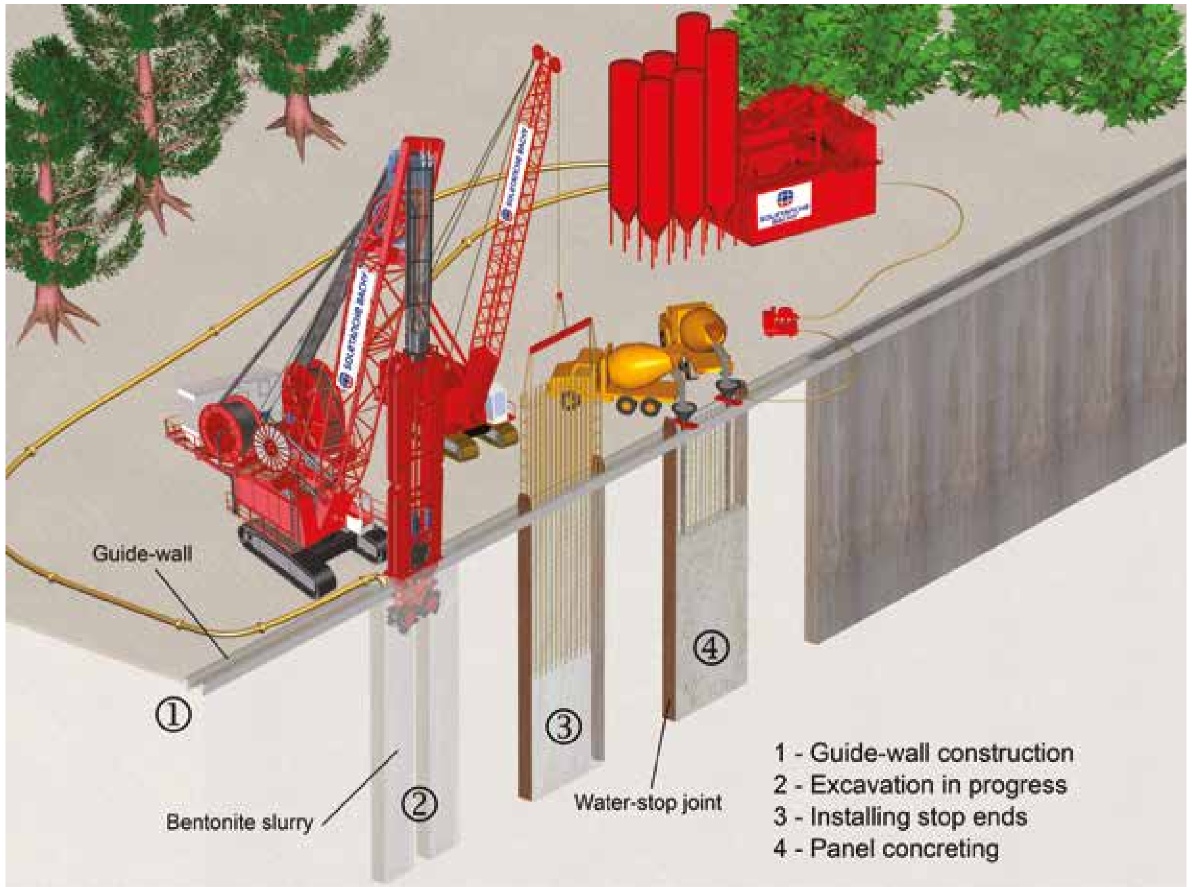

GEOMIX®


TRENCHMIX®

DIAPHRAGM WALL

1. Definitions

A diaphragm wall is a reinforced concrete wall that is cast in sections or panels excavated in the ground. The trench held open during excavation, and installation of reinforcement and concrete by the use of a supporting slurry. The slurry forms an impervious deposit (cake) on the walls of the trench, isolating the hydraulic pressure of the slurry from the surrounding soil and ground water, such that this pressure exerts sufficient outward force to keep the trench open. The slurry mix can be based on the use of bentonite, or polymers or a mixture of the two.



The temporary guide walls are constructed in advance and consist of two reinforced-concrete sections each about 30cm thick and 1m deep. The guide-walls have several functions:

- to provide physical confirmation of the location of the wall,
- to guide the excavation tool,
- to provide a reservoir for drilling mud,
- to provide a fixed support for suspension of the reinforcement cages.

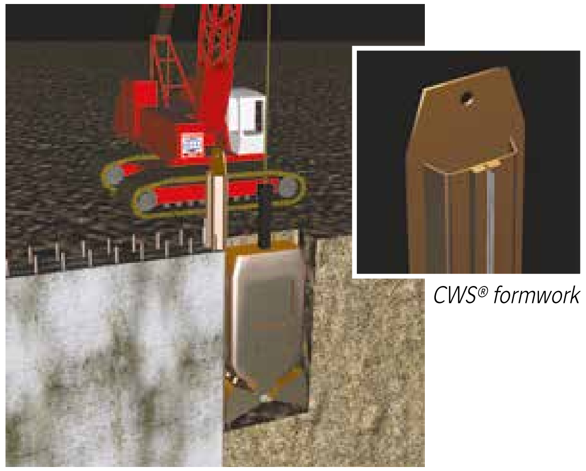
Individual panel lengths are determined by a number of factors including trench stability and the sensitivity of the surroundings to movement. Typically they do not exceed 7m. The wall can be constructed very close to existing structures though a minimum clearance is required for the thickness of the guide wall.

When excavation of a panel is complete the slurry is treated to reduce the quantity of solids in suspension to a predetermined acceptable level.

Thereafter, the reinforcement cage is installed and concrete poured using a tremie pipe.

The joint between adjacent panels can be achieved in one of two ways:
- By use of a temporary steel stop end allowing the placement of a waterstop across the joint and providing at the same time a guide for the excavating tool.
- By cutting back into the concrete of the previously constructed panel when excavating with a Hydrofraise®.

The standard thicknesses of diaphragm walls are:
0.50m, 0.60m, 0.80m, 1.00m, 1.20m, 1.50m and 1.80m.


CWS® formwork

2. Excavating tools

Excavating tools fall into two main categories:

Cable operated and hydraulic grabs
Cable operated grabs were the original tools employed for diaphragm wall excavation. Chiselling is required when the ground becomes too hard for the unaided grab to progress. Hydraulic grabs benefit from the versatility of this power source allowing greater productivity and monitoring and correction of the verticality if necessary.

Hydrofraise®
A Hydrofraise® consists of two counter-rotating drums on horizontal axes fitted with cutting teeth. The spoil produced is brought to the surface by reverse circulation of the drilling fluid. The machine can penetrate very hard ground (with compressive strength of up to 80MPa) without the need for chiselling.

Tolerance on verticality of the wall:
Without special precautions the tolerance is around 1% of the excavated depth. This can be reduced to 0.3% in certain case, by taking certain precautions, such as:
- the use of on-board electronics for 3-D monitoring of the trajectory of the excavating tool and correction as required,
- reduced operating speeds.

3. Construction control and monitoring

Construction of the diaphragm wall
Each stage of the construction of the diaphragm wall presents technical issues and each affects the quality of the finished product.

Excavation: the tools (hydraulic and Hydrofraise®) can provide real-time measurements of any alignment error and steps can be taken to correct it. (see the Deep Foundations chapter, section 2.2). JOE2000 software is used to process the data sent by the acquisition systems carried on the excavation tool.

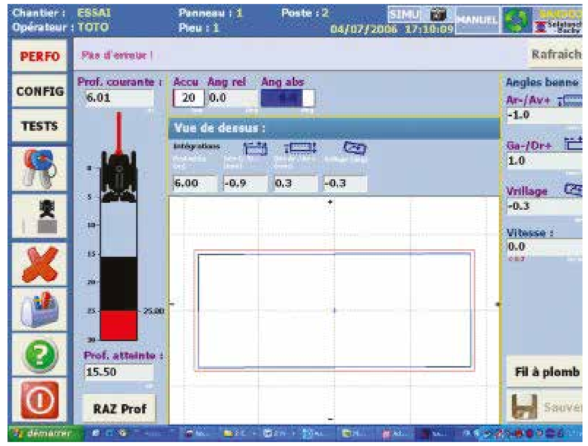
When the excavation is complete, it is possible to check the trench geometry by purpose-designed sonar.

Concreting: during the excavation process and before the concrete is poured, the physical and chemical properties of the slurry are monitored. A concreting curve is plotted during the pour to detect any under-consumption and concrete samples are taken to test the compressive strength.
Concrete quality in-situ can be assessed by sonic testing.

Excavation in front of a diaphragm wall
The following instruments can be used when required:
- inclinometers in the diaphragm wall,
- strain gauges and settlement gauges in the retained soil,
- targets for topographical measurements on the wall itself and on neighboring buildings.

A monitoring programme is set up for each stage of the work and can be used as part of the observational method.


ENPAFRAISE Hydrofraise® control unit



SAKSO screen showing actual and theoretical grabber positions

4. Applications - Advantages

Diaphragm walls are ideal for use in water-bearing ground.

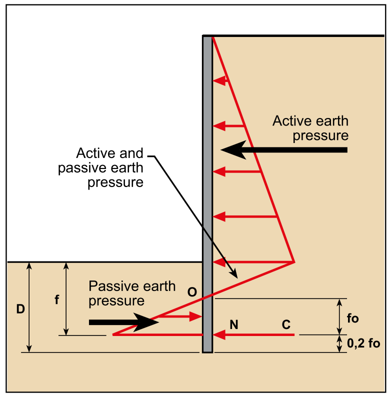
Unlike soldier-pile (Berlin-type) walls, the entire wall is constructed before excavation, considerably simplifying the earthworks.
The wall can be used as either a temporary or permanent structure, and in the latter case it is incorporated into the permanent works.

The great stiffness of a diaphragm wall reduces deflection compared with a Berlin type wall or sheet piling.
This is a very important consideration when working in an urban environment.
In addition to its retaining role, a diaphragm wall can have other functions:
- load bearing foundations,
- hydraulic cut-off.

5. Geotechnical design

5.1. Determination of the embedment by limit state

The conventional method for determining the embedment of a retaining wall is by limit state analysis on the basis of rigid plastic soil behavior.
The embedment depth is determined by applying a reduction cœfficient to the passive earth pressure of between 1.5 and 2.0.



< Freestanding or cantilever wall

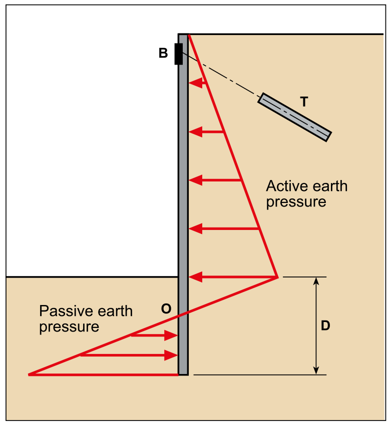
For the wall to be stable, it needs to generate counter thrust, C.
The stresses acting on the wall above the point N are the active pressure on the retained face and the passive pressure on the excavation face, each with the required factor of safety.
The equilibrium for the horizontal loads and moments will determine the unknown values:
- the counter thrust, C
- the length ON (fo)

We assume the counter-thrust is spread over 0.2 fo above and below N.
Wall embeddment depth, is therefore equal to **f + 0.2 fo**

Wall with one support >

The embeddment depth, D, is found by taking moments about point B (point of application of the support load) with appropriate factor of safety on the passive pressure.

The balance of external forces acting on the wall can be used to calculate the stress, T, in the ground anchor or strut (the calculation is made without reduction in passive pressure).



The following standards should be referred to
Eurocode 7: Geotechnical design and its national appendix.
NF P 94-282 standard: Design of retaining walls.

5.2. Arch Structures

A number of structures or parts of structures, such as storm retention tanks and tunnel access shafts, are circular in shape.

They behave in the same way as a succession of horizontal rings subjected to the pressure exerted by the soil and water. The compressive hoop stress in the concrete needs to be checked.

With this type of structure, intermediate supports are generally not required. The embedded depth of the wall can be determined on the bases of stability with respect to base heave, hydraulic considerations and load-bearing capacity.

Openings in the wall (for a tunnel for example) or asymmetrical loading can distort the hoop stress and need to be carefully checked.

5.3. Soil-structure interaction

Once the wall embedment depth has been determined, the soil-structure interaction can be analysed in order to:
- determine the stresses in the wall and its supports,
- estimate the magnitude of wall deflection.

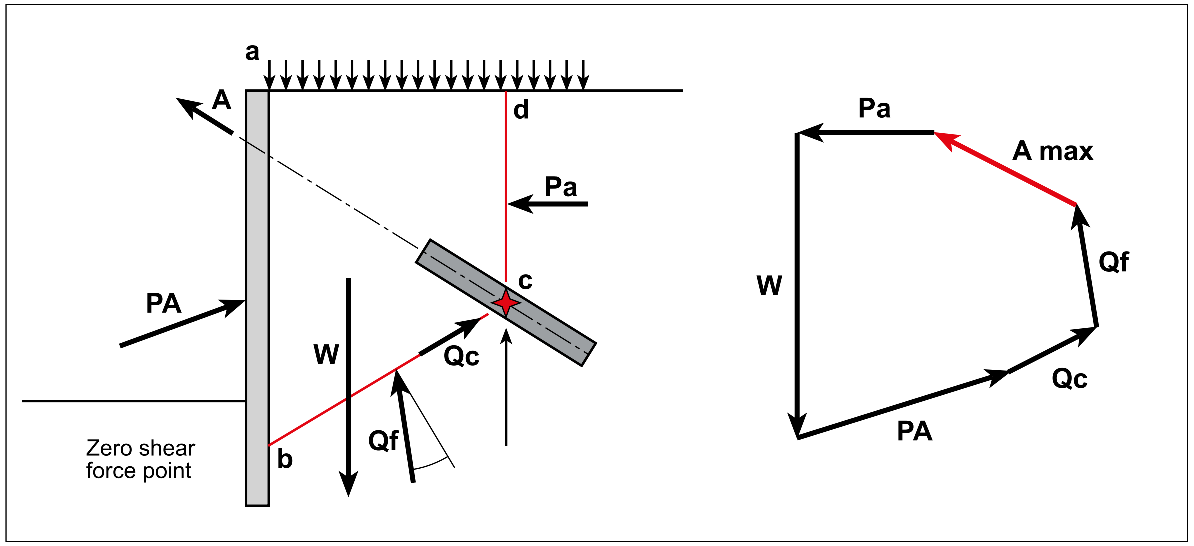
It is then possible to:
- establish the wall thickness and calculate the reinforcement,
- establish support loads.

Normal practice for these analyses are:
- In relatively straightforward cases, modeling the soil as a series of elasto-plastic springs with a stiffness based on the cœfficient of subgrade reaction.
- In a more complex geometrical configuration, for example such as structures constructed on slopes or where there is interaction with existing nearby structures, finite element analysis is used.

5.4. Other Checks

Overall stability
The overall stability of the system must be checked (walls, supports and surroundings), especially in the case of:
- walls constructed on slopes,
- substantial surcharge loads exerting pressure behind the wall.

Stability of the soil mass confined by prestressed anchors.
The bond zone of the anchors must be formed sufficiently far from the wall. The stability of the soil mass is checked using the Kranz method.



A: anchorage reaction
PA: reaction of the wall on [a b]
Pa: earth pressure on [c d]
W: weight of soil mass (abcd)
Qf: reaction on [b c] due to friction
Qc: reaction on [b c] due to cohesion

A max, the greatest anchorage load compatible with soil mass equilibrium, is calculated graphically.

F: safety cœfficient = A max/A around 1.5

Base heave

A check against base heave, in which the ground outside the wall undergoes bearing failure at the toe of the wall with resulting heave in the base of the excavation, needs to be carried out.

Load-bearing capacity of the wall

This check is usually only carried out when a structure is subjected to high vertical loads, for example:

- a wall supported by high-capacity, steeply inclined ground anchors,
- a wall supporting superstructure vertical loads.

Hydraulic considerations

When below the water table the following issues need to be addressed:

- possible effects of water flow reduction of passive earth pressure, boiling, regressive erosion,
- environmental effects of the works and the structure, external drawdown, dam effect,
- overall stability of the structure.

5.5. Reinforced concrete

General order of moment of resistance at SLS

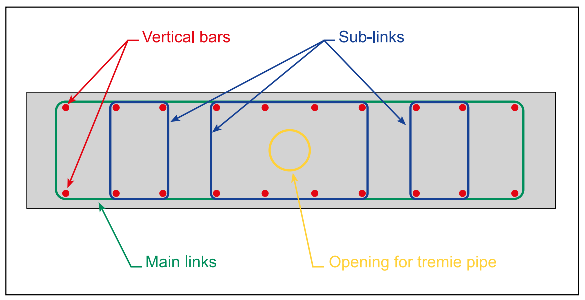
Thickness	Usual «moment»	Associated tensile steel section
0.50 m	300 kNm/m	30 cm^2/m
0.60 m	600 kNm/m	37 cm^2/m
0.80 m	1,200 kNm/m	52 cm^2/m
1 m	1,800 kNm/m	67 cm^2/m
1.20 m	2,600 kNm/m	82 cm^2/m
1.50 m	4,000 kNm/m	105 cm^2/m
1.80 m	7,000 kNm/m	150 cm^2/m

FeE500 steel, allowable concrete compressive stress - 12 MPa

Reinforcement and buildability considerations

A diaphragm wall reinforcement cage includes the following:

- Structural bars:
  - main vertical bars, possibly with additional reinforcement, to resist bending moments,
  - horizontal bars to resist shear force.
- Starter bars or couplers for tying to the final structure.
- Block-outs for ground anchors etc.
- Bars required for installation:
  - lifting bars,
  - suspension bars,
  - stiffening bars.



As the concrete is poured under drilling fluid using a tremie pipe from the bottom up, the provisions in EN1538 standard must be incorporated from the preliminary design stage, and in particular:

- the concrete used must be highly fluid (slump of around 20cm), and retain its workability for several hours. Additives are often used.
- The following requirements must be met:
  - minimum clear spacing of 100mm between the bars for satisfactory concrete placement.
  - Allowance for one or more clear openings in the cage for the tremie pipe.

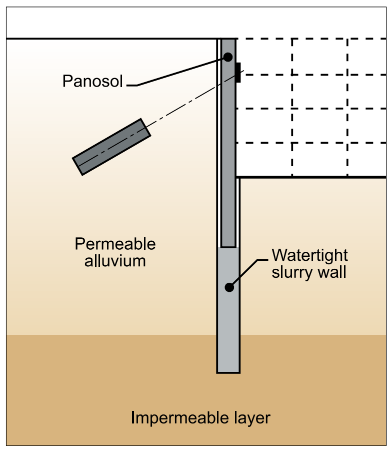
Main standards

Eurocode 2: design of concrete structures and national appendix
Eurocode 7: geotechnical design and national appendix
NF P 94-282 standard: design of retaining walls
EN 1538 standard: execution of special geotechnical works: diaphragm walls

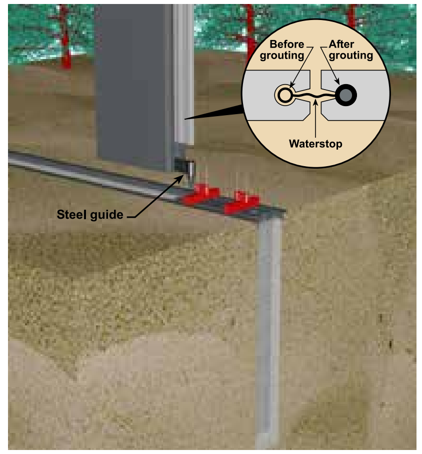
6. Precast diaphragm walls

Instead of pouring concrete in a trench in situ, precast reinforced concrete elements are lowered into the trench. A bentonite cement slurry is used as the excavation fluid and seals the prefabricated element in the ground.

The advantage of this process is that it separates the retaining function (precast reinforced concrete element) from the cut-off function (bentonite cement).



A groutable water-stop can also be inserted between the elements. The only limitation on the use of precast diaphragm walls is the lifting capacity of the cranage on site which needs to be sized to handle the dimensions and weight of the precast panel units.



7. References


FRANCE - Le Havre - Port 2000 - 1.6km of quay wall constructed using diaphragm walls


SPAIN - Valencia - Corte Inglés - Diaphragm wall 54m deep


POLAND - Warsaw - Prosta Center - 3,600m² of diaphragm wall 0.60m thick


SPAIN - Madrid - Palacio Municipal del Hielo - 6,400m² of diaphragm wall


SINGAPORE - The Sail@Marina Bay - 6,800m² of multi-cell diaphragm walls


FRANCE - Floirac - Circular diaphragm wall 31m in diameter


MONACO - Exhibition and Cultural Center - Excavation 25m in depth

SOIL MIXING

1. Principles

SOIL MIXING uses a wide range of techniques to inject binder agents to mix with the soil and form columns, for example, to reinforce the ground for subsequent construction.
The type and amount of binder will determine the hydraulic and mechanical characteristics of the soil. When the technique is used for treating contaminated ground, a specific binder agent can be chosen to neutralise the particular type of contamination.

Soil mixing generally comprises three stages: premixing of the soil, injection of the binding agent and incorporation of the soil/binder mix.
The inclusions produces no, or very little, spoil.
The structures produced by soil mixing can be columns, or panels or continuous trenches.
The technique works with all types of loose soil which are free from coarse elements.

2. Applications

Improvement of compressible soils beneath loaded areas: industrial units and warehouses, road and rail embankments.

Construction of cut-off walls for containment of contaminated zones or control of groundwater.

Construction of temporary ground support in combination with vertical reinforcing elements (steel beams, tubes, posts).

Reinforcing embankment slopes by constructing transverse walls.

3. Methods

3.1. Trenches

3.1.1. GEOMIX®

CSM: a hybrid solution with many uses
CSM combines the advantages of soil mixing techniques and hydromill cutter technology: the process has the robustness and proven track record of the Hydrofraise® and the effectiveness of soil mixing, which consists of mixing soil in situ with a cement/bentonite grout.
CSM, or Cutter Soil Mixing, comprises two pairs of drums mounted on compact hydraulic motors which have the dual purposes of cutting into the ground and mixing in the binder. The CSM equipment is compatible with various types of base units, which provides great flexibility.

An advanced control system
The control system simultaneously provides real-time monitoring of the two key process parameters: the homogeneity of the soil/binding agent mix and the amount of binding agent injected into the volume of soil treated. At the same time it allows the verticality of the inclusion to be checked. The equipment can be supervised and operated from the cab, through the on-board computer.

The soil is premixed during the downward excavation and the spoil is moved towards the top of the cutting head. As the machine moves upwards again, it moves the mix from above the cutting head to below it. During this phase a binding agent is injected and mixed with the soil.

The rotating drums cut the soil and mix it with the binder.

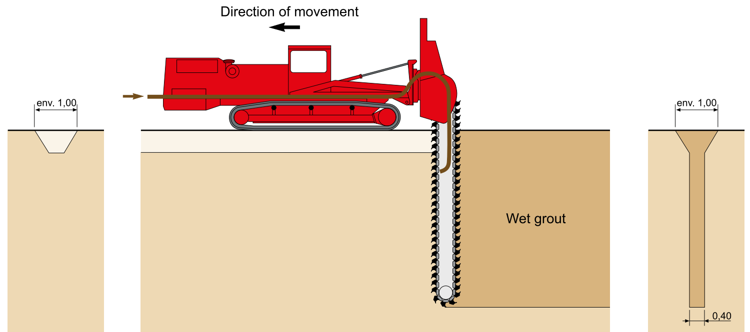
The latest technologies are used for monitoring work quality

Geomix® rig

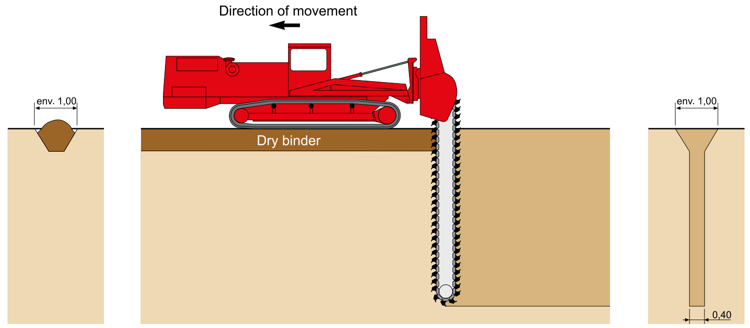
3.1.2. TRENCHMIX®

The trenches of mixed soil and binding agent are constructed using a specially designed trencher, which has the following features:
- the soil is broken up and mixed, rather than excavated,
- the soil and binding agent are mixed in situ.

The binder may be introduced as a powder (dry method) or as a pre-mixed grout (wet method).



< Met method



< Dry method (the binder is introduced as a powder in the trench prior to mixing)

TRENCHMIX® rig

TRENCHMIX® JUNIOR: reinforcing pylons

3.2. Columns

3.2.1. Single or multiple columns

Columns are made using tools rotary about a vertical axis.

Single column rig

Multiple column rig (COLMIX)

SOIL MIXING

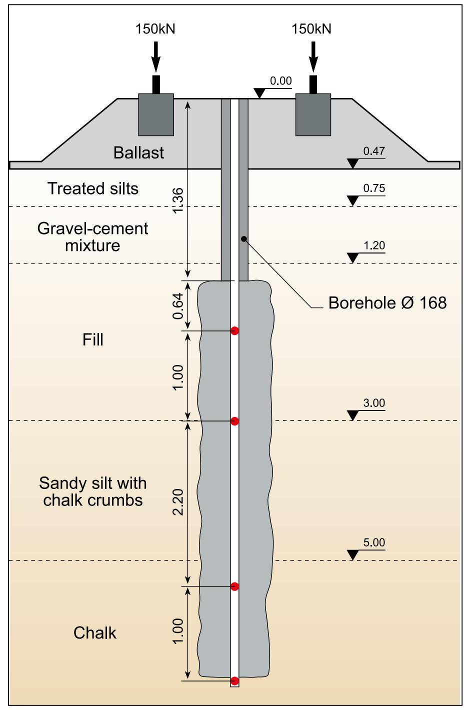
3.2.2. SPRINGSOL® Process

This process was developed for introducing soil-mixing columns under railway tracks:

- between sleepers,
- through the ballast, without contaminating it.

A tube of 168mm in diameter is lowered down a borehole drilled through the base of the railway track. The column is then installed beneath the tube, using a retractable tool.
A monitoring system is used to open the tool.

Drilling rig being used on a railway track



400 mm columns built under a railway track

Introduction of the retractable tool in the tube

Retractable tool

Excavated test column

4. Monitoring systems

The monitoring systems manage the quantities of binder injected and also log the soil mixing parameters.
Samples of the mix are taken to make sure that the design parameters have been met.

5. Examples of applications

Switzerland - Viège
GEOMIX® cut-off wall

FRANCE - Marseille - Axe littoral
GEOMIX® retaining wall

FRANCE - Montereau
Warehouse (soil improvement by TRENCHMIX®)

UNITED STATES - Pittsburgh
GEOMIX® retaining wall

DEEP FOUNDATIONS

1. Definitions

Deep foundations, as described in this chapter, cover the following geotechnical structures:
- piles, including barrettes,
- micropiles.

A pile is defined as a structural element placed in the ground to transfer loads and limit settlement. There is no limit to its slenderness ratio.
Pile shafts can be of uniform cross-section, tapered or with enlargements on the shaft or at the base.
Piles can be installed either in isolation or in groups. They can also be used to form a mixed retaining wall, contiguous pile wall, secant piles and composite walls, such as soldier pile (Berlin-type) walls and similar.
Piles can also be used as the foundation for plunged columns, integrated into the structure of the building they support.
They may be inclined depending on the requirements of the relevant codes.

2. Scope and Methodology

There are three main types of piles:
- displacement piles, as described in the construction standard for special geotechnical works NF EN 12699,
- bored piles, as described in the construction standard for special geotechnical works NF EN 1536,
- micropiles as described in the construction standard for special geotechnical works NF EN 14199.

Piles have now been constructed with diameters of up to 5 to 6m and depths of over 100m, on land, over water and/or offshore.

2.1. Displacement piles

2.1.1. Principle

Displacement piles are installed by forced penetration.
This forced penetration includes the following installation techniques:
- impact or driving,
- vibration,
- jacking,
- screwing,
- a combination of these methods.

Displacement piles will mobilise the maximum point load by compaction of soil under the pile toe if the toe is closed or plugged.
There are two types of displacement piles:
- prefabricated in either steel, reinforced concrete, prestressed concrete, wood, composite materials and/or any combination of these materials,
- cast-in-place, usually in reinforced concrete.

2.1.2. Prefabricated displacement piles

Prefabricated steel piles
Prefabricated steel piles may be:
- butt-welded tube sections,
- H sections, purpose designed as they have the same web and flange thickness,
- box sections; made from the assembly of two U-sections or sheet piles.

Grouting, during or after pile driving is necessary in certain soil conditions (chalk, calcareous sands) to restore lateral friction.
Steel corrosion due to the contact with water and/or the ground is dealt with by:
- using an extra sacrificial steel thickness,
- applying paint,
- using passive protection (sacrificial anodes),
- using active protection (imposed current).

Precast reinforced concrete piles
Precast concrete piles can have the following crosssections:
- circular,
- square,
- rectangular,
- polygonal.

The elements are joined together using metallic connectors. Pile manufacture is carried out either by vibrating the concrete or by centrifuging, which is a more suitable for prestressed piles.

2.1.3. Cast-in-place displacement piles

Cast-in-place displacement piles are made by driving a temporary or permanent tube fitted with a non-overlapping drive shoe and/or a toe plug, retrievable or otherwise.
The installation of the tube is followed by the placement of the reinforcement and concrete within the casing.

Concreting is usually carried out in the dry.
Removal of the temporary casing requires a certain precautions to be taken to avoid damage to the pile.
Among the different types of cast-in-place displacement piles, Soletanche Bachy has developed a rotary displacement pile that provides enhanced lateral friction due to a groove cut into in the ground over all or part of its height. There is also an end bearing component. These piles are marketed under the name of SCREWSOL.® Soletanche Bachy has also developed a displacement pile with an extendable tremie tube. These piles, marketed under the name REFSOL, have their own specification.


SCREWSOL® tool mounted on a boring machine

2.1.4. Preliminary design for driven piles

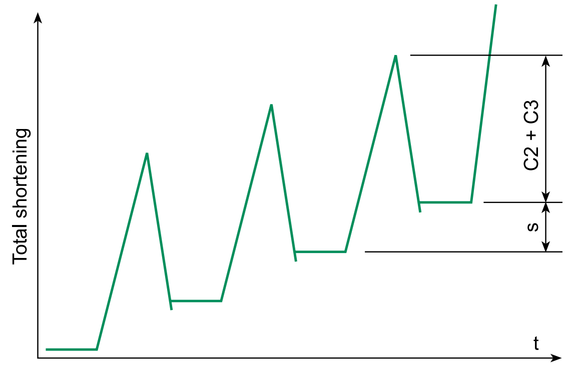
For driven displacement piles a preliminary design needs to be carried out:
- to define the procedures and equipment to be used to install the piles to the desired depth,
- to define the energy required to overcome the resistance of the ground, compatible with the structural strength of the pile,
- to define pile-driving termination criteria.

The preliminary design is based on either:
- various dynamic formulæ using the energy delivered by the hammer and transmitted to the pile element. This is used to develop driving criteria so as to ensure that the required bearing capacity is provided,
- the theory of wave propagation in solid media.

Hiley formula
The Hiley formula is the most widely used dynamic formula used for pile driving energy:

$$R_d = \frac{f.Er.(Wr+e^2.Wp)}{(s+1/2(C1+C2+C3)).(Wr+Wp)}$$

where:
R_d: total dynamic resistance,
Er: hammer energy according to the manufacturer,
f: efficiency of the hammer,
Wr: hammer weight,
Wp: weight of the pile,
C1, **C2**, **C3**: elastic shortening of the pile driving helmet and packing, the pile, and the soil respectively,
e: rebound coefficient,
s: permanent displacement.



Definition of the elastic and permanent shortening of a driven pile

Wave propagation formula in continuous media
The theoretical basis of this analysis is the equation of superposition of waves, the general form and solution of which can be expressed as follows:
$\partial^2u/\partial z^2 - 1/c^2 . \partial^2u/\partial t^2 = 0 \quad u(z,t) = f(z+c.t) + g(z-c.t)$
The displacement, u, is the displacement produced at a given point by 2 waves of opposite direction and the same speed. Smith has proposed a relationship between the static resistance, **Rs**, and the dynamic resistance, **R_d**:
$R_d = R_s (1 + j.v)$ with **j** as the dynamic amplification factor (it varies from 0.1 to 0,8s/cm) and **v** as the displacement speed at a given point on the pile. **Rs** is equal to the soil resistance to pile driving at zero speed. This approach is the most commonly used in the United States, under the name of the Case method.
Other predictive methods for load bearing capacity have been developed using the same theory, such as CAPWAP, SIMBAT, TNO WAVE, STATNAMIC, CALYPSO.

2.1.5. Monitoring and control of pile driving

These formulæ should be used on the basis of a preliminary static axial loading test and/or dynamic loading tests.

The preliminary design is used in conjunction with continuous monitoring as the piles are being driven and a dynamic loading test performed after pile driving.

This allows for measurements to be taken of stresses and accelerations in the pile head for each impact, the total soil resistance to pile driving and the pile driving resistance after soil set-up. This last measurement is taken to be the load bearing capacity of the pile.

2.2. Bored piles, including barrettes

Bored piles and barrettes are distinguished from each other by their cross sections:
- a circular section is referred to as a pile,
- square sections, rectangular sections, T-sections and L-sections or any other similar configuration are referred to as barrettes.

Construction methods for bored piles vary considerably:
- cased piles,
- piles excavated under drilling fluid, including barrettes,
- continuous-flight auger piles.

Sometimes it is possible to bore open-hole in the dry if the soil conditions permit.

The method chosen depend on a number of factors including ground conditions, depth and the equipment available:
- auger, mounted on telescopic kelly for discontinuous drilling in a casing or under drilling fluid,
- cable grab (circular for piles or rectangular for barrettes) used under drilling fluid or possibly within a circular casing,
- reverse circulation drilling rig with air lift and casing for continuous pile drilling,
- a milling type reverse circulation machine for continuous barrette drilling under drilling fluid,
- a continuous flight auger drill for continuous pile drilling,
- down-the-hole (DTH) hammer with direct or reversed circulation with or without casing for continuous pile drilling.

The diameters of the drilled piles commonly range from 300mm to 3m, but are constantly increasing and now reach 5 to 6m.
Depths of 100m are also becoming relatively common for inland sites.
Enlarged pile bases are formed using mechanical or hydraulic under ream tools to construct base diameters of up to 4.50m.

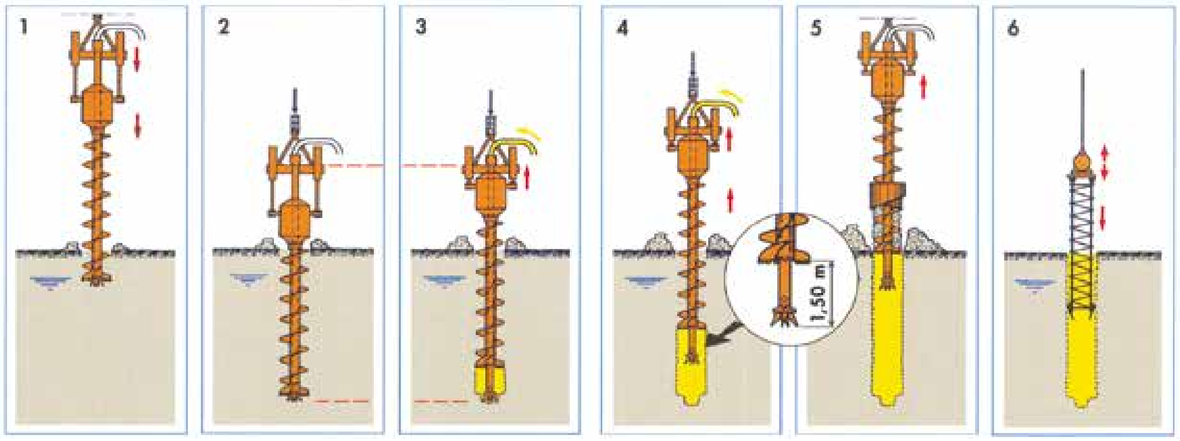
Bored piles are usually of reinforced cast-in-situ concrete. The reinforcement can also be a precast concrete section or a steel section to build retaining walls such as soldier pile walls or as pre-founded (plunge) columns.

Except under very specific conditions, concrete is placed in the wet, using a tremie tube which is either:
- independent and placed after the reinforcement cage has been installed,

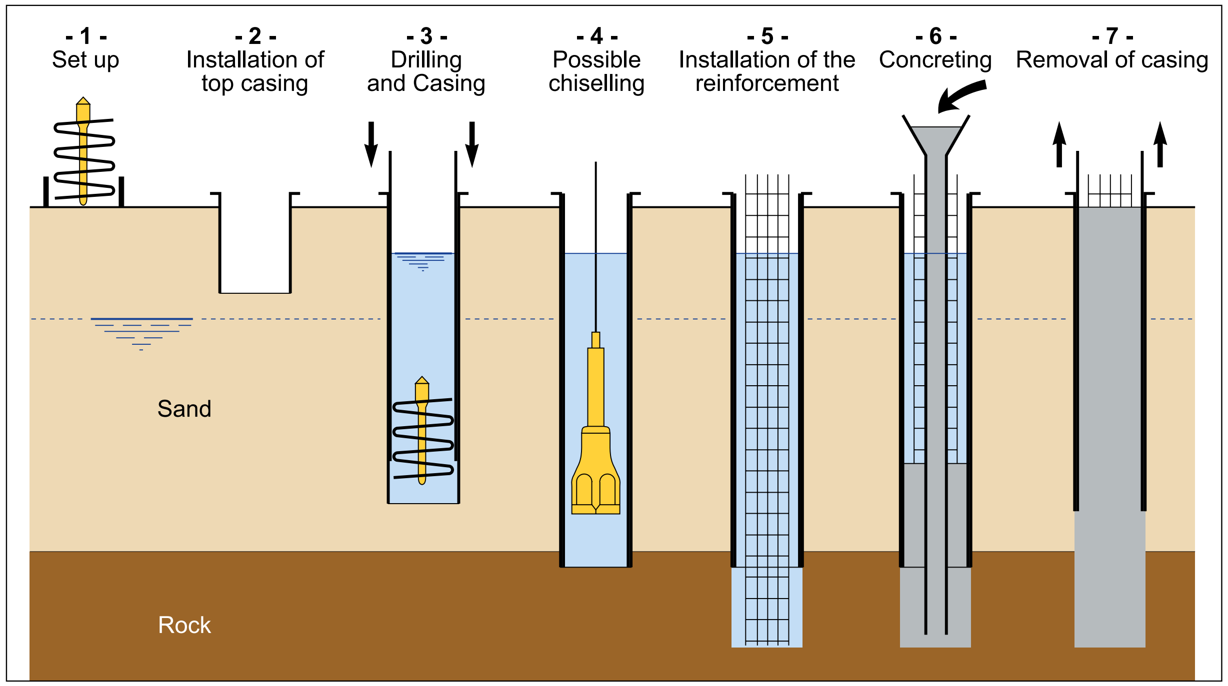
- independent and placed prior to installing a precast reinforced concrete or steel section,
- incorporated into the drilling tool, in which case the reinforcement is placed after concreting as in the case of continuous-flight auger piles.

The compressive strength of the concrete is usually between C20/25 and C30/37 (cylinder/cube).

Type II cements are recommended. The additives in these cements reduce the heat of hydration, improve the workability and the durability of concrete used for deep foundations. The cement content can vary from 325kg/m³ to 450kg/m³, depending on aggregate size and concreting conditions.



STARSOL® pile



Cased piles

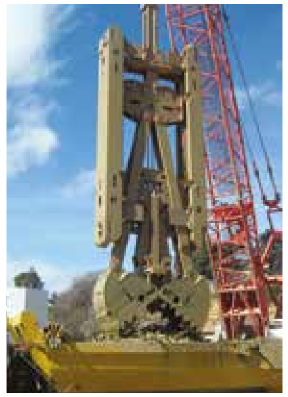
DEEP FOUNDATIONS

Soletanche Bachy has developed highly sophisticated machines with excellent productivity and on-board electronic systems providing high reliability of construction for piles and barrettes:

- HYDROFRAISE®, a milling-type cutter to excavate barrettes up to 2.40m thick in all soil types including rock. The machine has continuous, real-time monitoring of excavation process using ENPAFRAISE. This information allows the machine to correct its trajectory to remain within tolerance,
- KS, rectangular hydraulic grabs designed for cohesive and non-cohesive soils for excavating barrettes up to 1.50m thick and fitted with the SAKSO system for monitoring and correcting trajectory to remain within tolerances.
- STARSOL®, a continuous-flight auger drill for constructing piles up to 1.50m in diameter and 35m deep, fitted with an extendible tremie pipe and continuous monitoring of drilling and concreting, through the ENBESOL® version of SYMPA, to ensure correct construction of the pile in dry or wet conditions.

Soletanche Bachy has developed high-performance piles, such has the grooved T-PILE®. These new cost-efficient pile concepts are intended to improve the load bearing capacity to cost ratio by reducing the amount of concrete required. They have their own specification.

HYDROFRAISE® EVOLUTION 3 Machine for very deep diaphragm walls or barrettes



M8 new-generation grab

STARSOL® - Continuous earth auger with integrated temie pipe and continuous recording of parameters

T-PILE® - Grooved piles for increased performance

< ENPAFRAISE (for recording hy settings)

2.3. Micropiles

Micropiles are drilled piles with a diameter less than 300mm and displacement piles with a diameter less than 150mm. The load bearing element of a micro pile consists of steel bar, a steel tube, or an H-type profile that is:

- either embedded into the ground using a cement grout, mortar or micro-concrete for load transfer (drilled micropiles),
- or by direct contact with the ground (displacement micropiles).

A characteristic of micropiles is that their bearing capacity is assumed to be derived exclusively from lateral friction since the end bearing is frequently negligible.

Lateral friction can be vastly improved by adding pressure grouting, either a tightening exercise called Global and Unique Injection (GUI) shortly after cementing the steel reinforcement in the ground, or by an IRS type injection (repetitive and selective grouting under pressure) with sleeved casings. These two improvement methods can also be combined.

Construction methods for micropiles are similar to those of piles in general. However, drilled micropiles are constructed using light drilling machines that allow continuous direct and/or reverse circulation drilling with fluids such as air, water, bentonite slurry, polymers, or cement grout.

This foundation technique is generally used for repairing existing foundations, and for strengthening foundations of existing structures, as the equipment is light and able to work within the existing structure.
Lost-bit drilling, in which the drill string is left in place to become the load bearing element after cement grouting of the annulus, is a solution that requires skill and care during construction.

Micropiles can also be used for building new structures and micro soldier pile walls.

The very high slenderness ratios of these foundation elements sometimes requires checking for buckling in poor soils and the addition of reinforcement to increase the inertia.

During underpinning works of building foundations, the partial or total transfer of anticipated loads can be carried out in advance by jacking in order to limit the effects of differential movements over time due to the fact that the vertical deformation of micropiles is greater than that of bored piles with an equivalent bearing capacity.

3. Design principles

The following Eurocodes are the general reference documents for the design of deep foundations:
- EN1990: 2002 Eurocode : Basis of calculation for the structures,
- EN1991 Eurocode 1: Actions on structures,
- EN1992 Eurocode 2: Design of concrete structures,
- EN1993 Eurocode 3: Design of steel structures,
- EN1994 Eurocode 4: Design of composite steel and concrete structures,
- EN1997-1 Eurocode 7 - Part 1: Geotechnical design - General rules,
- In1997-2 Eurocode 7 - Part 2: Geotechnical calculations. Site survey and testing,
- EN1998 Eurocode 8: Structural resistance to earthquakes.

The national application document NF P 94 262 - Pile foundations clarifies section 7 of Part 1 of Eurocode 7 concerning the design itself.

The design of deep foundations must be carried out to ultimate and service limit states for the loads arriving from the structure resulting from the loads to which it is subjected.

Additional checks may also be required for deep foundations that need to allow for:
- poor ground conditions,
- earthquakes.

3.1. Determination of the embedment length of piles

The determination of the embedment length of piles subjected to axial loading is based on the geotechnical characteristics of the soil defined by one or more of the following:
- in situ pressuremeter tests, static and/or dynamic penetrometer tests,
- in situ static and/or dynamic axial loading on test piles,
- in the laboratory on the basis of triaxial tests on «intact» samples.

Foundation behaviour is characterised by an elasto-plastic relationship between the axial load at the pile head and displacement at the pile head, from which two load parameters can be identified:
- creep load,
- limit load.

Soletanche Bachy SETPILE software calculates pile settlement using several methods, including the Frank-Zao method.

3.2. Negative skin friction

The load bearing capacity of a pile and/or and group of piles must take into account negative friction induced by the settlement of compressible soils surrounding the piles under the effect, for example, of a surcharge related to backfilling or a drop in the level of the water table due to dewatering.

Forces related to negative friction can be calculated using a method based on load at failure such as the one proposed by Combarieu.
These settlements can also be associated with horizontal movement of the soil under the influence of a surcharge, leading to lateral forces on the foundations of which due consideration is needed in design.

DEEP FOUNDATIONS

3.3. Ground heave

Ground heave can result from either of the following:
- a swelling phenomenon, e.g. a soil sensitive to water,
- the installation of displacement piles through deformable soil that can produce substantial tensile forces in neighbouring piles.

When displacement piles are installed in soft and plastic clays, this heave may also be accompanied by a lateral displacement of the ground that may damage neighbouring piles. In these circumstances, specific installation procedures should be used.

3.4. Determination of pile resistance to lateral forces

The behaviour of piles loaded horizontally varies considerably according to their slenderness and fixed end conditions at the head of the pile.
For short piles, rupture occurs mainly by plastification of the soil, whereas for long piles it mainly occurs within the pile itself. In such situations, either a stability calculation or a calculation of the bending moment resistance must be performed respectively.

The coefficient of sub-grade reaction method allows for simple mathematical solutions in the case of long piles subjected to a unit load for a cœfficient that is:
- constant with the depth,
- increasing with depth (piles of offshore structures),

This allows the assesment of:
- the displacement at the pile head and along the length of the pile,
- the rotation at the pile head,
- the maximum bending moments developed in the pile.

Soletanche Bachy HOLPILE software can calculate the stresses on a pile taking into account any lateral deformation of the soil.

3.5. Pile Groups

The group effect refers to the interaction between closely installed foundations and must be taken into account when the distance between them is less than 3 diameters. Generally, this effect increases horizontal and vertical displacements and reduces load bearing capacity. Terzaghi's equivalent pile method and similar approaches can be used to model this interaction.

Soletanche Bachy's PICASSO software can be used for the design of a group of vertical and/or inclined piles subjected to forces of any kind.

3.6. Calculation of the structural capacity of the pile

The structural capacity of a pile is checked against the stresses caused by pre-defined loading cases. It takes account of the materials making up the pile and the applicable standards or codes.

Soletanche Bachy's PROVERB software is used for checking sections subjected to loading of any kind in accordance with the provisions of various reinforced concrete codes (BAEL BSI, EC2).
For cast-in-place reinforced concrete piles, the minimum reinforcement section must meet the criteria described in the following table:

Nominal section of a pile: Ac	Section area of longitudinal reinforcement: As
Ac ≤ 0,5 m^2	As ≥ 0,5% Ac
0,5 m^2 ≤ Ac ≤ 1,0 m^2	As ≥ 0,0025 m^2
Ac > 1,0 m^2	As ≥ 0,25% Ac

The longitudinal reinforcement must contain at least four bars of a diameter greater than or equal to 12mm with a spacing between the bars of between 100mm and 400mm.
Recommended diameters for transverse reinforcement are given in the table below:

Stirrups and hoops	≥ 6mm and ≥ 1/4 of the diameter of longitudinal reinforcement
Wires or welded mesh	≥ 5 mm

The concrete cover to reinforcement must not be less than 75mm.

3.7. Internal instability

Buckling can be examined for long piles and micropiles for soils defined as soft.
Souche charts or suitable software can be used.
Soletanche Bachy PARIS software has this capacity through its buckling module.

3.8. Earthquake

The design of deep foundations under earthquake conditions is covered in Eurocode 8 - Part 5.
It must take into account:
- the inertia of the superstructure,
- dynamic lateral forces due to movement of the soil during earthquakes against which the ultimate capacity of the foundations needs to be checked.

The piles must behave elastically but with the possibility of the formation of plastic hinges.

4. Monitoring and Checking of Construction

Two stages can be distinguished in the monitoring process:
- during installation using the on-board computer on the drilling machine,
- after installation to confirm the quality of a foundation or detect faults. These checks serve to correct poor quality construction.

4.1. Monitoring and Checking during Construction

Soletanche Bachy is aware of the need to provide high quality deep foundations in all circumstances. With the participation of its staff and the technical competence of its engineers, it has developed rigorous and real-time monitoring tools for the construction process in order to take immediate action in the event of changing conditions.

These tools help the operator to immediately correct any departure that may compromise the quality of the foundation.
SYMPA or «Modular and versatile system for data acquisition» is the control system for these various tools.
On-board systems for each type of machine used for the installation of deep foundations have been developed around SYMPA:
- ENPAFRAISE, installed on the HYDROFRAISE®, enables visual monitoring of the hydraulic parameters of the machine. It helps to position the cutter in three dimensions and to correct departures from the theoretical position of the barrette.
- SAKSO, installed on the hydraulic grab KS, has the same functions but also provides complete control of the entire excavation cycle including the slew necessary to deposit the cuttings from the grab.
- ENBESOL®, installed on the continuous flight auger STARSOL®. It monitors and assists with concreting of the pile as in addition to the drilling.
- ENPASOL®, installed on small diameter drill rigs to record the drilling parameters during the construction of micropiles.

DEEP FOUNDATIONS

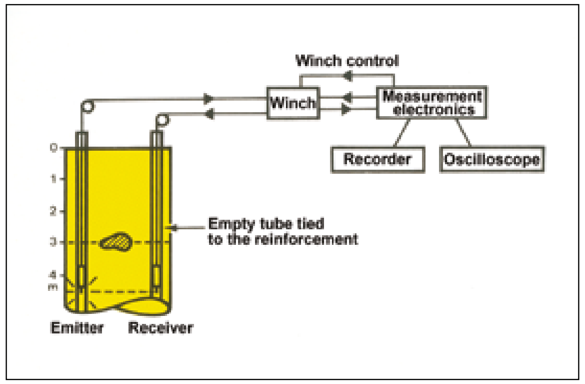
4.2. Monitoring and Checking After Construction

These tests can be either destructive or non destructive.

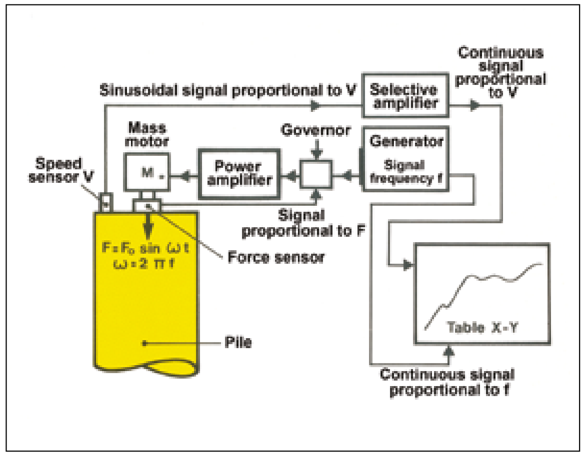
4.2.1. Non-destructive tests

Non-destructive tests of piles are essentially based on four standardised inspection methods:
- the sonic coring method,
- the reflection method,
- the parallel seismic method,
- the impedance method.

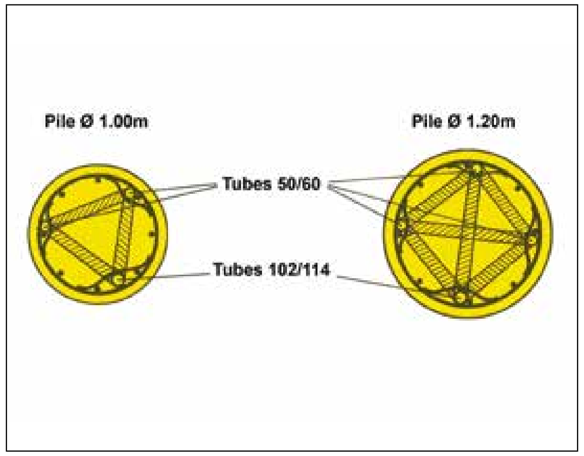
The parallel seismic method is the least used as it requires the drilling of additional bore holes alongside the pile.
All these methods give the length of the constructed pile and detect abnormalities in the pile shaft by measuring the wave velocity in concrete.



Sonic coring - Measurement principal



Mechanical impedance method - Diagram of the apparatus



Sonic coring - Tube layout
Only the hatched zones are investigated

4.2.2. Destructive tests

A diamond coring drilling tool, with continuous sampling, should be used when there are doubts about the quality of the concrete of the pile.
Core drilling is essentially used to visually inspect the quality of the contact of the concrete with ground at the pile toe.
Core drilling is used to improve this contact by injecting cement grout under relatively high pressure.

4.2.3. Loading tests

Load testing for deep foundations can be divided into static tests and dynamic tests.
Static load tests are those involving compressive axial loads, tensile loads or lateral loads.
Dynamic type tests can also be used for determining the load bearing capacity of the foundation, but these are high-energy tests and are used mainly for driven piles.
They can be used for in-situ concrete piles, providing precautions are taken to avoid damage to the head.

5. Project references

5.1. Piles


HONG KONG - AIG Tower Furama, Bored Piles


HONG KONG - 402 - Steel H Piles


FRANCE - Paris - Farman
Seine-Ouest district, Starsol® piles


NEW CALEDONIA - Prony
Down-the-hole hammer (DTH) drilling


UNITED KINGDOM - London - CTRL 105
St Pancras Railway Station, Bored piles

DEEP FOUNDATIONS


MACAO - Wynn Resorts Diamond Suite Hotel,
Bored piles with plunged columns


URUGUAY - Quai TCP
Piles over water


HONG KONG - 402
Reverse circulation pile drilling machine

5.2. Micropiles


FRANCE - Paris - Raynouard street


UNITED STATES - New York - World Trade Center

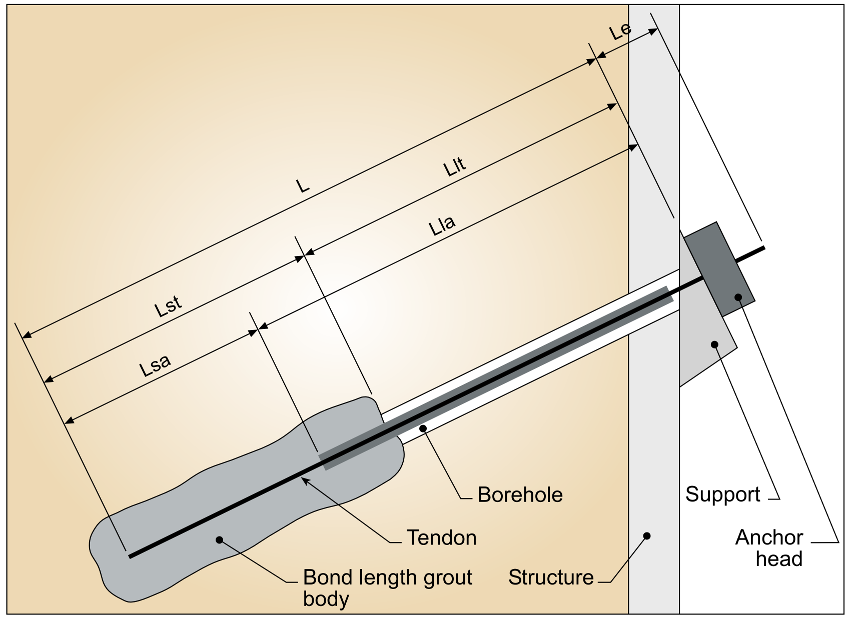

FRANCE - Marseille - Grand Littoral

GROUND ANCHORS

1. Definitions

A ground anchor is a load transfer system designed to transfer the forces applied to it to a competent stratum. An anchor is generally said to be temporary if it has a lifespan of under 18 months and permanent if the lifespan is over 18 months.

An anchor comprises three parts:
- The head, transmitting the anchor force to the structure via the bearing plate.
- The free length of tendon, from the head to the near extremity of the bond length.
- The bond length, which is the length of tendon through which the tensile force is transmitted to the surrounding ground through the bond grout.



Le: External section

Lst: Bond length to ground

Llt: Free length to ground

Lsa: Bond length of tendon

Lla: Free length of tendon

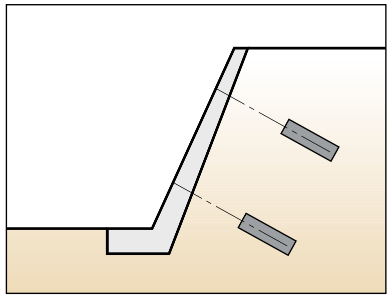
At: Tendon cross-section

There are «active» and «passive» soil anchors:

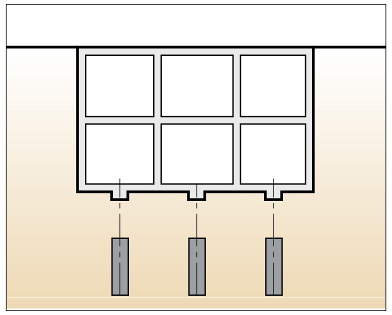
- A **passive anchor** is only tensioned by the structure itself applying load to it. It dœs not usually have a free length. Generally speaking, the armature consists of a steel bar or sometimes a bar of composite material.

- An **active anchor** is pre-tensioned, which reduces displacement of the structure. The armature is usually made of steel cables as used for pre-stressing.

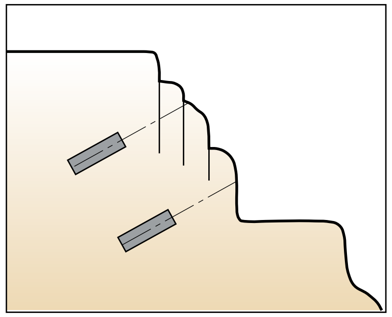
2. Fields of application



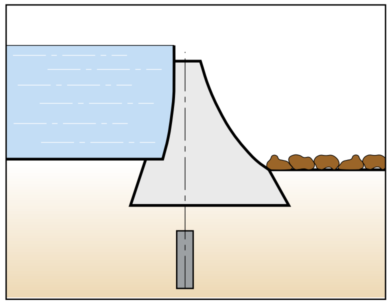
< Supporting excavations
Diaphragm walls - Sheet pile walls - Retaining walls - Underpinning walls - Berlin-type walls and similar.



Resisting tensile loads >
Ramps below groundwater - Prestressing of tension piles - Anchoring of slim structures (pylons, tower-blocks, stacks...) - Resisting cable or shroud loads (suspended bridges, pylons, etc.).



< Pinning - Nailing
Fissured rock, cliffs, scree slopes - Stabilisation of landslides - Strengthening tunnels - Penstock reaction blocks.



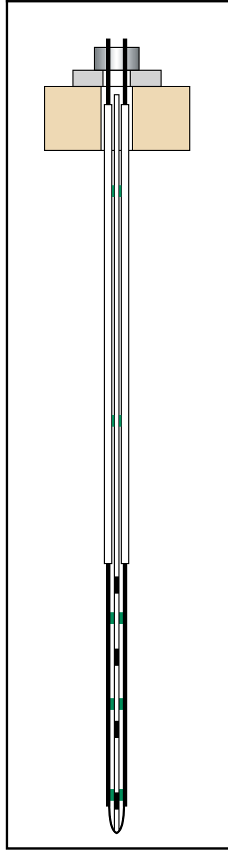
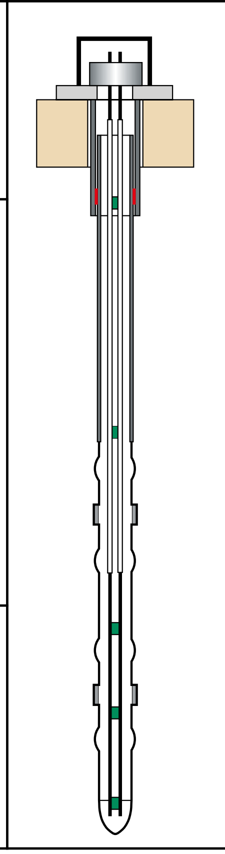
Miscellaneous >
Take up of arch thrust - Structure post-tensioning - Enhancing dam stability

3. Protection against corrosion

The type of protection depends on the anchor service life and the aggressivity of the environment.
The protection is applied over each of the three parts of the anchor.

STANDARD RULES (TA. 95)				**NF.EN.1537**	
Service life / Environment	Under 9 months	9 to 18 months	Over 18 months	Temporary anchors under 2 years	Permanent anchors over 2 years
Non-agressive	P0	P1	P2	The basic protection is similar to P0 but can be enhanced. The service life can be extended beyond two years if planned at the outset.	The basic protection is similar to P2, but with a minimum cover of 20mm between the anchor casing and the ground.
Moderately agressive	P1	P2	P2		
Agressive	P2	P2	P2		

Examples

TEMPORARY ANCHOR (P0)		**PERMANENT ANCHORS (P2)**	
	**Non-aggressive environment and atmosphere, short-term** **Anchor head** **A non-fluid anticorrosion coating.**		**Anchor head** **A rigid, painted head-cover, connected to the bearing plate. The head-cover is filled with an anticorrosion product.**
	Free length **The tendon is protected by a casing which is blocked off at either end. The casing must allow the tendon to extend freely during pre-tensioning.**		**Transition zone** **A trumpet tube is connected to the free length and filled with an anticorrosion product.**
			Free length **Each tendon is encapsulated within a flexible casing, filled with grease. A tube around all the tendons is filled with a dense cement grout.**
	Bond length **The tendons must have a grout cover of at least 10mm relative to the borehole wall.**		**Bond length** **A steel TAM at least 3mm thick, filled with a dense cement grout A minimum grout cover of 20mm between the anchor tube and the ground with a minimum grouting pressure of 0.5MPa. Minimum grout cover between the armature and the inside of the grout tube of 5mm.**

NB: the P1 protection provides intermediate protection between P0 and P2 (the bond length does not have the protection of P2)

4. Installation

The sequence of operations for installing a soil anchor is as follows:

• Drill borehole, diameter 100-200mm, depending on the dimensions of the anchor body, at the appropriate angle, using a drill rig and drilling fluid to suit soil conditions.

• Clean borehole, replace drilling fluid with grout, usually a high-cement-content mix (water/cement ratio between 1.7 and 2.3)

• Insert the ground anchor by crane, from drum or even by manhandling.

• Once the grout has set, the bond length may be pressure-grouted with cement grout. Various grouting systems are used to suit ground conditions and the degree of bonding enhancement required. The most common method is the tube à manchettes sleeved grout pipe (see the Grouting chapter). The TA 95 guidelines provide for two main methods:

- IRS (Injection Repetitive Selective) (Selective Repetitive Grouting)
- IGU (Injection Globale Unique) (Single hit overall grouting)

• A period of 2 to 5 days, depending on ground and grout type, is left between the final grouting and anchor tensioning.

• The anchor head protection is added after tensioning is validated

5. Design

The cross-section of the tendon, the length of the anchor and the length of the free part must be determined.

5.1. Tendon cross-section (At)

This is based on the maximum measured tension (P) remaining in the anchor to provide structural stability under a serviceability limit state.

	STANDARD RULES (TA 95) *	
TEMPORARY ANCHOR	Under 18 months	At ≥ 1.33 P / $Pt_{0.1}k$
	Over 18 months	At ≥ 1.67 P / $Pt_{0.1}k$
PERMANENT ANCHOR	At ≥ 1.67 P / $Pt_{0.1}k$	

Ptk: Characteristic tendon tension - $Pt_{0.1}k$: yield point at 0.1% of tendon strain.
Under seismic action, application of AFPS 90 : Cross-section ≥ 1.11 P / $Pt_{0.1}k$

** These figures may be amended by application documents and by European standards specific to certain categories of structures.*

5.2. Bond Length

Bond length is determined by:
- reference to past experience,
- full scale tests,
- a theoretical assessment.

Theoretical Assessment

The most commonly used method in France is the Bustamante method (Bull.liaison lab P. and Ch 140 - Nov-Dec 1995), given in appendix 3 of the TA.95.
In this method, ground anchorage capacity is presumed to be proportional to:
- to the anchorage length in the ground (Ls),
- the equivalent drill-hole diameter ($D_s = \alpha$ Dd),
- the ultimate unit lateral friction of the ground (q_s).

The estimated ultimate tensile capacity (Tu) of an anchor is calculated using the formula
$Tu = \pi \alpha\ Dd\ Ls\ q_s$

q_s is read off existing graphs according to the type of soil, its compacity and the grouting method to be employed

α is an enhancement cœfficient, the value of which depends on the type of soil and which is also highly dependent on the grouting method proposed.

When this assessment method is used, a safety factor of 2 is applied to the ultimate capacity.

It should be borne in mind that the value given is an estimate only, and the final capacity should be determined by the results of the trial anchors.

As a general guide, typical figures are:
- loose sand and gravel: 20 - 40KN/m,
- dense sand and gravel: 60 - 120KN/m,
- stiff clay and silt: 20 - 60KN/m,
- hard clay and silt: 40 - 100KN/m,
- weathered chalk: 50 - 80KN/m,
- sound chalk: 100 - 150KN/m,
- rock: 150 - > 250KN/m.

These values are representative for bond lengths of between 5 and 15m.

5.3. Free length

Assessment of the free length is based on 3 main criteria:
- position of the anchorage stratum,
- the minimum length of the tendon to allow the preload, (including allowance allowing for mechanical losses) to be locked in,
- the overall stability of the ground mass taking the load (Kranz method*)

* See paragraph 5.4 of the chapter on the «diaphragm walls».

6. Tendon capacity

6.1. Strand anchors

Units	1T15	2T15	3T15	4T15	5T15	6T15	7T15	8T15	9T15	10T15	11T15	12T15
ext strand Ø	Toron T15,7 - Feg = 1650 MPa - Frg = 1860 MPa *											
Steel cross-section in mm²	150	300	450	600	750	900	1 050	1 200	1 350	1 500	1 650	1 800
Breaking load in kN (Frg)	279	558	837	1 116	1 395	1 674	1 953	2 232	2 511	2 790	3 069	3 348
Yield point in kN (Feg)	248	496	744	992	1 240	1 488	1 736	1 984	2 232	2 475	2 723	2 970
Permanent anchor in kN (Ts = 0,60 Feg)	149	298	446	595	744	893	1 042	1 190	1 339	1 485	1 634	1 782
Temporary anchor in kN (Ts = 0,75 Feg)	186	372	558	744	930	1 116	1 302	1 488	1 674	1 856	2 042	2 228
Anchors complying with EN 1537	181	362	544	725	906	1 087	1 268	1 449	1 630	1 812	1 993	2 174

** This type of strand, under the European standard, is equivalent to the 0.6" 270 kpsi strand of the ASTM standards.*

6.2. Anchor rods

Steels	Diameter in mm		Cross-section mm²	Weight kg	Failure load Frg		Elastic limit Feg		Geotechnical		
	Nominal	External			N/mm²	kN	N/mm²	kN	Test 0,90 Feg	Temporary 0,75 Feg	Final 0,60 Feg
GEWI	16	18,00	201	1,61	550	111	500	101	90	75	60
	25	27,90	491	3,85	550	270	500	245	221	184	147
	32	36,00	804	6,50	550	442	500	402	362	302	241
	40	44,20	1 257	9,95	550	691	500	628	565	471	377
	50	55,60	1 963	15,41	550	1 080	500	982	884	736	589
GEWI Plus	18	21,00	254	1,96	800	204	670	170	153	128	102
	25	28,00	491	3,85	800	393	670	329	296	247	197
	28	32,00	616	4,83	800	493	670	413	371	309	248
	43	48,00	1 452	11,51	800	1 162	670	973	876	730	584
	63,5	70,00	3 167	24,86	800	2 534	670	2 122	1 910	1 591	1 273
DYWIDrill (Self drilling)	R25N	25,00	290	2,30	690	200	520	150	135	113	90
	R32N	32,00	430	3,40	650	280	530	230	207	173	138
	R38N	38,00	770	6,10	650	500	520	400	360	300	240
	R51N	51,00	1 070	8,40	750	800	590	630	567	473	378
	T76S	76,00	2 750	22,00	690	1 900	550	1 500	1 350	1 125	900
FREYSSIBAR + HSA	27	30	560	4,70	1 070	599	870	487	438	365	292
	32	36	816	6,82	1 070	873	870	710	639	533	426
	41	45	1 306	10,83	1 070	1 397	870	1 136	1 022	852	682
	51	56	2 030	16,84	1 070	2 172	870	1 766	1 589	1 325	1 060
	75	81	4 465	36,25	1 070	4 778	870	3 885	3 497	2 914	2 331
FREYSSIBAR + HTSR	27	30	560	4,70	1 200	672	1 050	588	529	441	353
	32	36	816	6,83	1 200	979	1 050	857	771	643	514
	35	39	975	8,08	1 200	1 170	1 050	1 024	922	768	614
	41	45	1 306	10,83	1 200	1 567	1 050	1 371	1 234	1 028	823
	51	56	2 030	16,84	1 200	2 436	1 050	2 132	1 919	1 599	1 279
	75	81	4 465	36,25	1 200	5 358	1 050	4 688	4 219	3 516	2 813

NB: for laid anchors, smooth bars are used. An example is given below

Steels	Diameter in mm		Cross-section mm²	Weight kg	Failure load Frg		Elastic limit Feg		Geotechnical		
	Nominal	External			N/mm²	kN	N/mm²	kN	Test 0,90 Feg	Temporary 0,75 Feg	Final 0,60 Feg
FILTARO (upset-end anchors)	60	76	2 827	22,2	800	2 261	500	1 413	1 271	1 059	847
	70	90	3 848	30,21	800	3 078	500	1 924	1 731	1 443	1 154
	90	105	6 361	49,94	800	5 088	500	3 180	2 862	2 385	1 908
	100	115	7 853	61,65	800	6 282	500	3 926	3 533	2 944	2 355
	125	150	12 271	96,33	800	9 816	500	6 135	5 521	4 601	3 681

7. Tests

There are 3 types of tests:

Tests to failure

Tests to failure are carried out either to determine the ultimate resistance of the ground, or to test a new kind of anchor. The precise nature of the tests is determined contractually on a case-by-case basis.

Control tests

These are non-destructive tests, and tested anchors can therefore be used in the structure. The aims of a control test are as follows:
- to confirm the behaviour obtained in the tests to failure,
- to determine the critical creep load when no test to failure is performed.

Generally at least 3 control tests must be performed for each structure.

Acceptance tests

Every prestressed anchor in a structure must undergo an acceptance test.

The aims are as follows:
- show that a proof load can be borne by the anchor
- make sure that the actual lock-off load (Po), excluding friction, is in line with the design lock-off load.

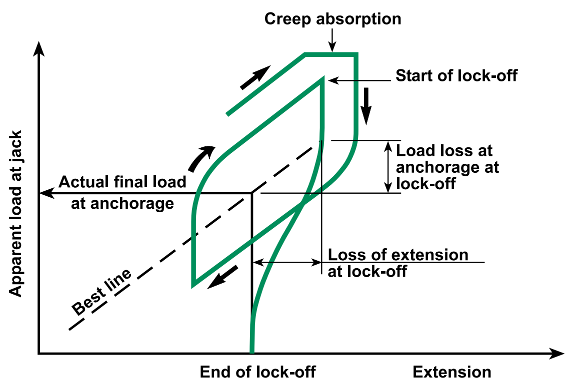
Design lock-off load (P):
| Po-P | ≤ Max (50 KN ; 5% P)

The cyclic method used by Soletanche Bachy for tensioning is described in the following paragraph.

8. Tensioning using the cyclic method

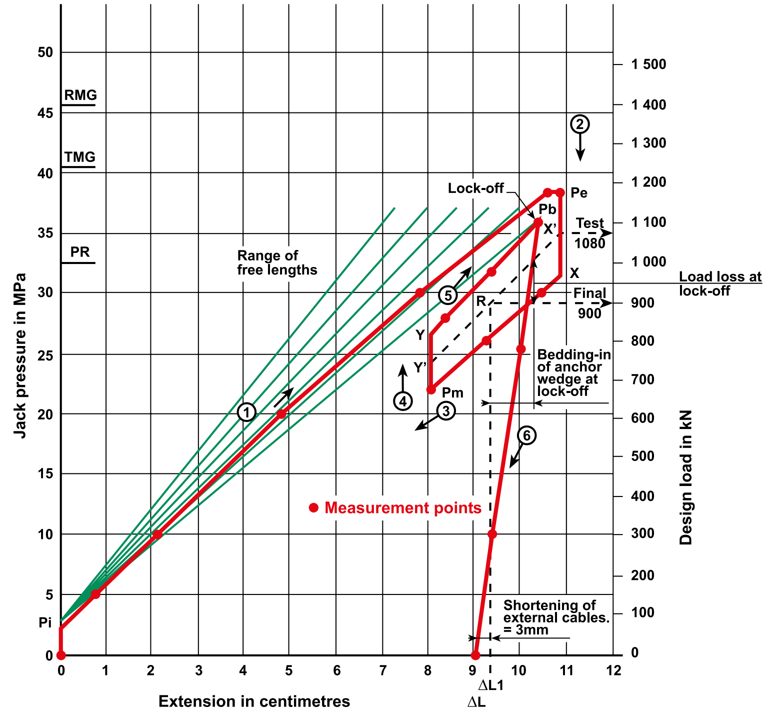
Description

A conventional cycle comprises:
- one load increase (in 3 or 4 stages),
- one partial load decrease: ideally with minimal shortening of the strand (to assess the friction in the system) (1 stage),
- further load decrease with shortening of the strand (in 2 - 3 stages),
- one reloading without extension of the strand (to assess the friction in the system) (1 stage),
- one reloading with extension of the strand, up to lock-off pressure (2 - 3 stages),
- measurement of residual deformation. (See figure)



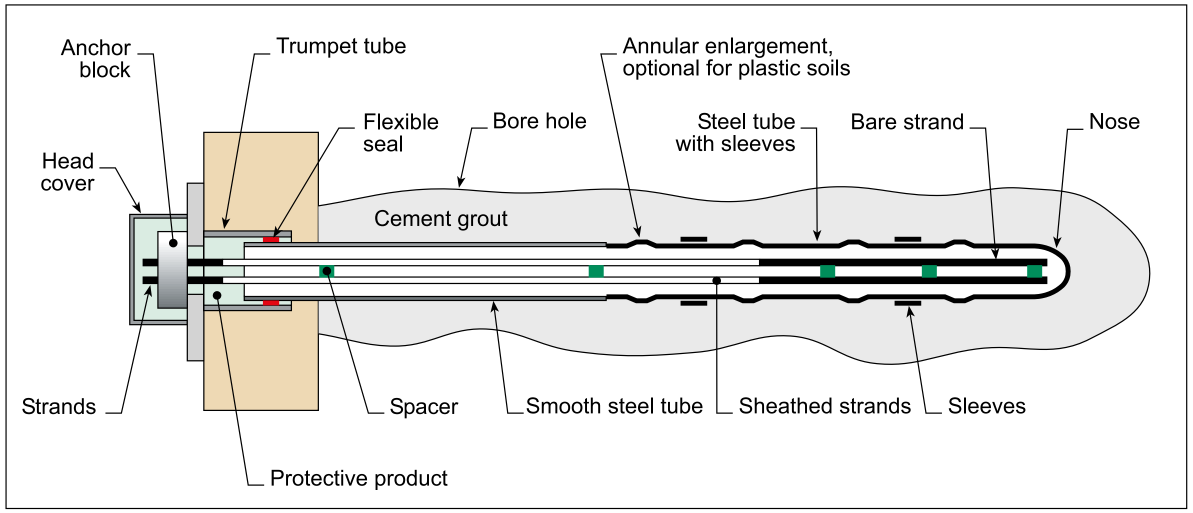
Interpretation

Monitoring the pressure in the loading jack allows the following assessments to be made:
- initial pressure, representing friction in the head at the outset,
- maximum test pressure; the Pi-Pe segment must fall within the range corresponding to the design free length,
- constructed point : represents two times the friction in the head (jack + head + pressure gauge),
- the mid point of the segment PeX gives the maximum anchorage test load,
- lowest point of unloading cycle Pm,
- constructed point Y and mid point of PmY,
- pressure at lock-off; segments XPm and YPb must be essentailly parallel; the segment Y'X' represents the true values of the loads for the measured extensions, i.e. excluding all friction,
- ΔL1 is set a few millimetres above measured ΔL, to factor in the extension of the part of the cables between the lock-off head and the jaws of the jack,
- point R constructed from X'Y' and ΔL1, gives the actual final anchor load, or residual load; the graph also allows to determine the load loss and bedding-in of the bearing plate at lock-off.



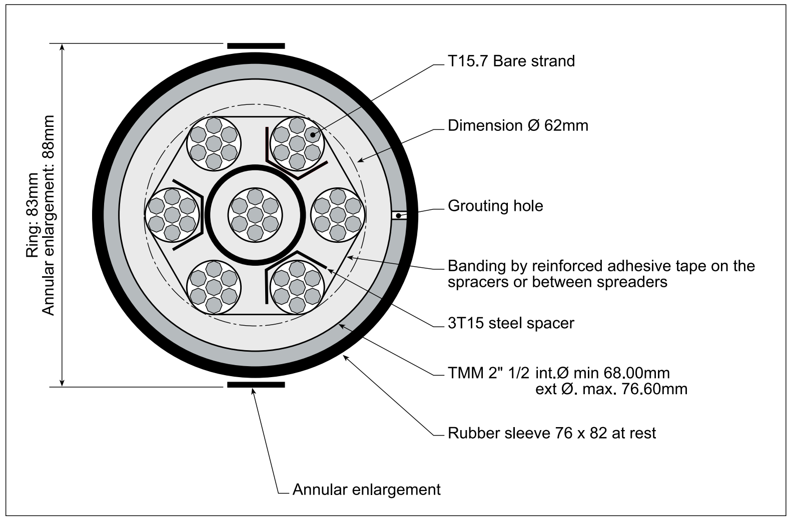
9. Description of the MMT anchor

The MMT anchor (Metal Manchette Tube) is a permanent (P2) active strand anchor, protected against corrosion; capacity can be adjusted according to needs, from 200KN to over 2000KN.

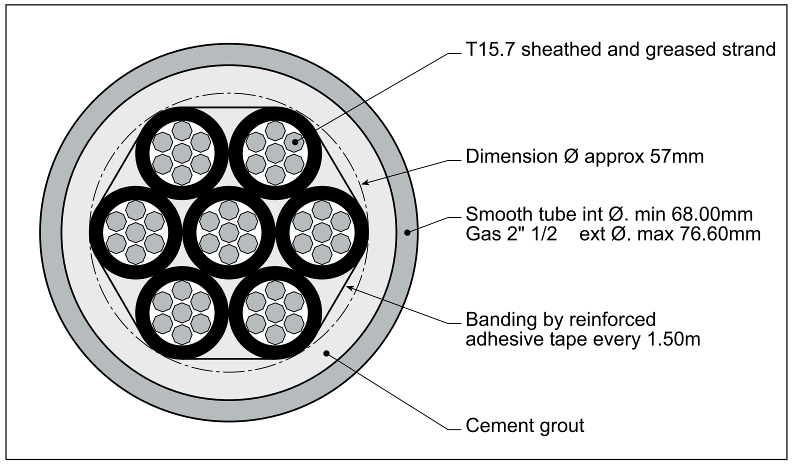


The MMT steel tube is filled with cement grout with a high cement/water ratio, following the final pressure grouting of the anchored zone.

Cross sections and dimensions



Anchorage section >



< Free length section

Other types of anchors are also possible.

10. Standards

NF EN 1997-1 standard: Geotechnical design,
NF P 94-282 standard: Retaining structures,
NF - EN 1537 standard: Execution of special geotechnical work - Ground anchors,
TA 95 guidelines,
NF P 94-153 standard: Ground anchor static test.

MONACO - Testimonio (2004 - 2005)
Berlin-type wall, shotcrete, diaphragm wall, temporary and permanent anchors

FRANCE - Cruseilles A41 (2007) Troinex retaining wall

FRANCE - Besançon - Tunnel du Bois de Peu (2004) Nailed wall

UNITED STATES - Gilboa dam (2006) Stabilisation

ARGENTINA - Potrerillos hydroelectric project (2000)
Support of North West embankment slope of the future Cacheuta power station

SPAIN - Valencia - Corte Inglés (2001)
Department store

GROUND WATER LOWERING - CUT-OFFS

1. Introduction

Problems related to groundwater control are among the most complex encountered in geotechnical engineering. They are influenced by:
- soil heterogeneity,
- anisotropy,
- the conditions in which the aquifers are recharged.

Knowledge of a large number of parameters is required for an adequate understanding of such a complex system.

In addition to any theoretical approach, experience is fundamental to the grasp of the mechanisms involved. However extensive the geotechnical investigation, and however advanced the expertise, a degree of uncertainty will nevertheless remain.

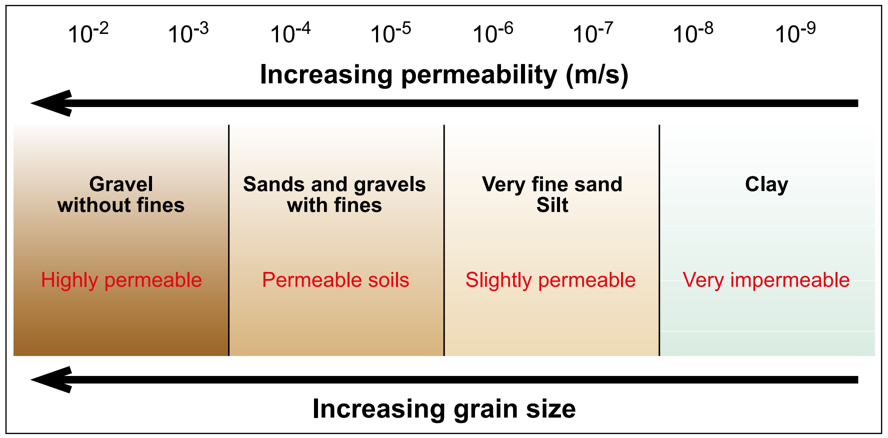
At the start of each project where ground water is present, the following issues must be addressed:
- the aquifers that may affect the works,
- the pumping discharge rate and the method by which the will the water be extracted,
- the acceptability of the effects of drawdown on the surroundings,
- the need to introducing barriers into the soil to alter the flow paths,
- the means that need to be put in place to ensure that the work proceeds as planned during the entire period of the dewatering.

2. Design procedure

2.1. Geotechnical site survey

As for any geotechnical project, a survey is required to understand the effects of groundwater lowering by reference to all aquifers that may influence the project. These aquifers are characterised by the following parameters:
- power (conductivity/transmissivity),
- nature (unconfined/confined),
- permeability,
- recharge conditions.

The permeability of a stratum can be measured in different ways. Those given below are in order of increasing reliability:
- in the laboratory from core samples taken from bore holes on-site. These can be used for direct measurement (permeameter) or indirectly by reference to particle size distribution as given by Hazen and Kozeny.
- in situ, including:
  - spot tests: «Lugeon» tests for rock and «Lefranc» tests for alluvial soils,
  - large-scale pumping tests. This involves the installation of one or more deep wells and associated piezometers to provide a good overall picture of the hydraulic behaviour of the soils encountered.

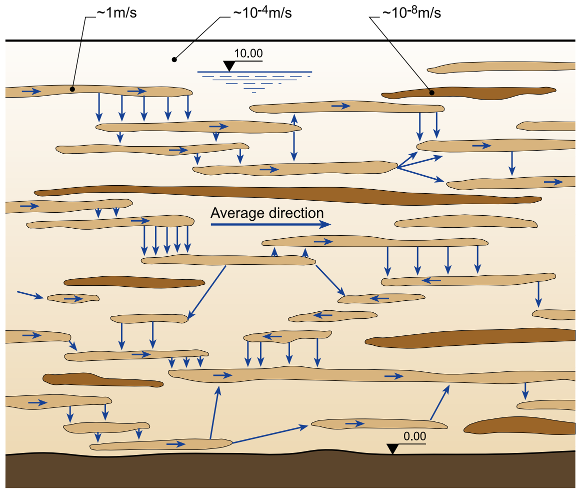


Permeability scale

The shortcomings of spot testing (in the laboratory and in-situ) are as follows:
- a legitimate doubt regarding the extent to which the results are representative of the whole site,
- a failure to take into account the anisotropy of the soil.

Pumping tests can overcome the limitations of spot testing, and can provide as accurate a picture as reasonably possible of the strata influencing the project. The Figure opposite illustrates the 'scale effect', which cannot be detected with spot testing.

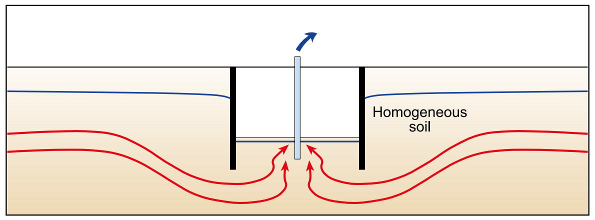
To extract useful information from the pumping test, the following must be installed:
- pumping wells,
- true piezometers (as opposed to stand pipes) with isolated pressure measurement points located such as to adequately measure the effect of pumping on the different aquifers.



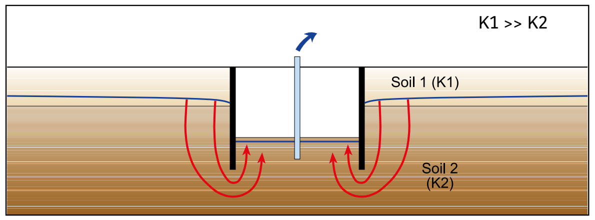
Example of scale effect for permeability

2.2. Water flow types

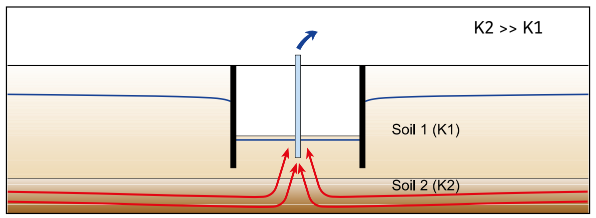
Once the different strata and their permeabilities have been identified, it is possible to construct the flow net associated with the proposed dewatering. There are three basic types of flow (see Figures). Specific site conditions could produce a combination or superposition of more than one of these types.



Radial flow



Downward vertical flow - cofferdam type
(corresponds to the case treated by the Davidenkoff chart)



Upward vertical flow

2.3. Defining the Dewatering Project

From the technical options available, the choice will be made by reference to the following factors:
- the hydrology,
- the geometry of the works,
- the external environment,
- the contractual framework (e.g. maximum discharge rate).

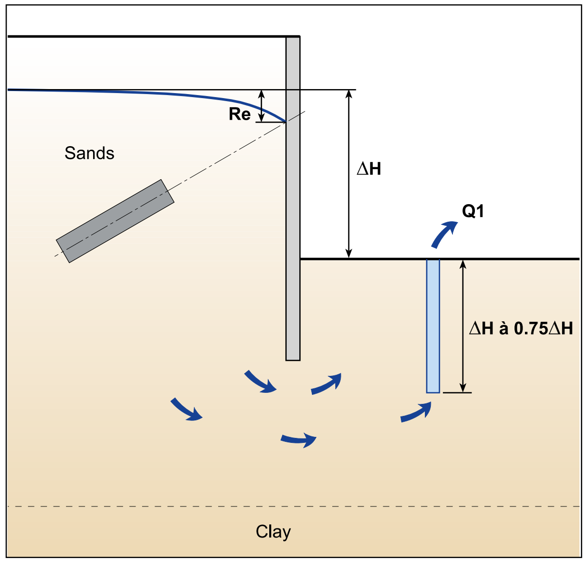
These choices must take into account the risks described in Paragraph 3 and be used to determine the requirements in terms of:
- possible cut-offs: walls, grouting,
- pumping installation (wells, wellpoints, etc.) and monitoring devices (piezometers), (Refer to the tables at end of Paragraph 4, indicating the scope of the different methods).

2.4. Examples of approaches for deep, retained excavations

We give below options that can be adopted to control ground water during bulk excavation in different ground conditions for deep retained excavations:
- in sandy soils,
- in rocky-like conditions,
- in competent soil with a more permeable soil stratum at depth.

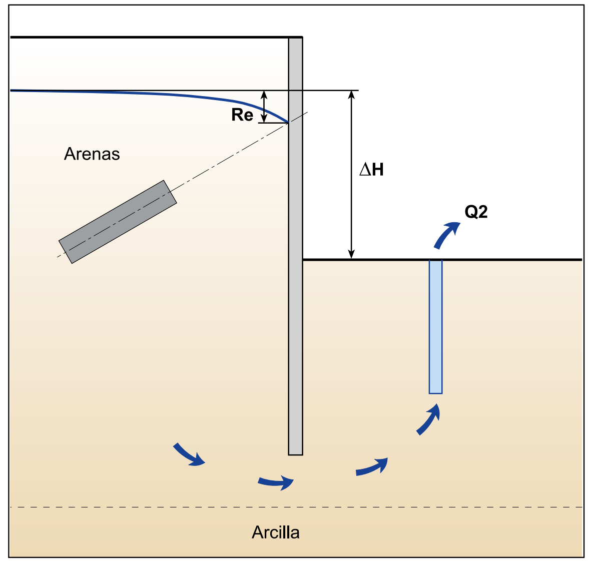
2.4.1. Excavation in sandy soils

Solution 1
Short wall and pumping



This solution is possible:
- if the discharge rate is environmentally acceptable,
- if the ground water can be effectively extracted (extremely difficult for silts and very fine sands),
- if the external drawdown, Re, is permissible.

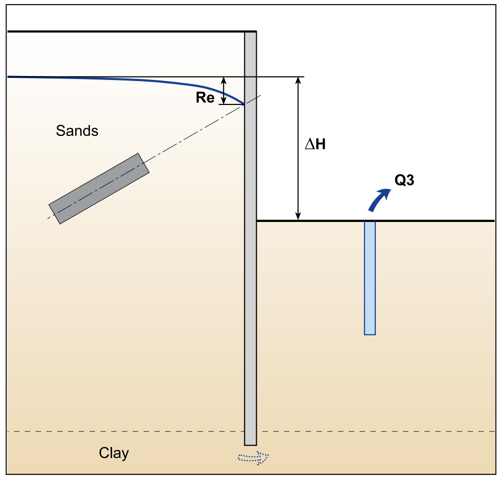
Solution 2
Extending the wall downwards



The deepening of the wall increases the head loss: Q2 < Q1

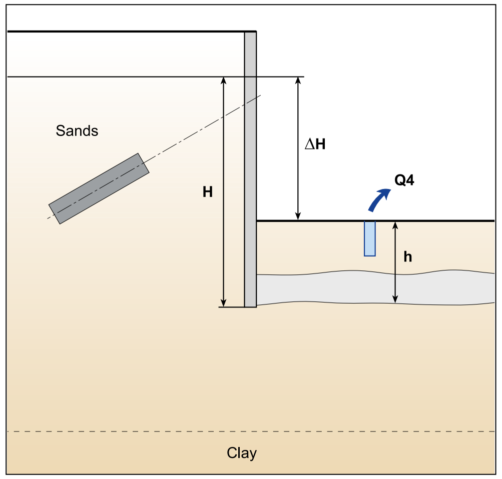
Note: this deepening of the wall can be replaced by a grouted cut-off.

Solution 3
Embedment of the wall in clay



Embedding the wall into clay will help to reduce the pumping rate since clay is far less permeable than sand.

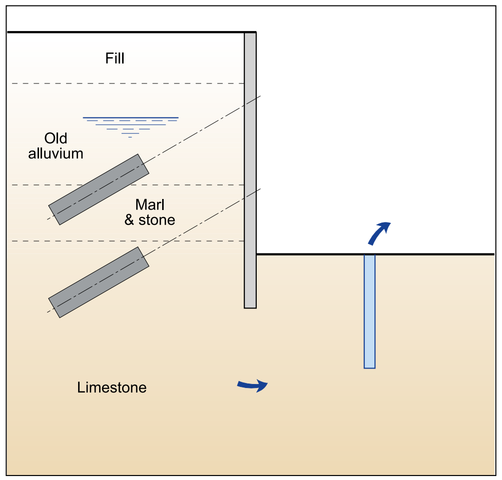
Solution 4
Short wall and grouted mat



This solution should only be used if there is no aquaclude at a reasonable depth.
In this configuration the head loss is concentrated across the height of the grout raft due to its low permeability.
The uplift force exerted on the underside of the raft is counterbalanced by the weight of the ground above.
A safety factor of 1.05 is required.
The hydraulic gradient across the grouted raft is generally limited to between 3 and 5, depending on the nature of the ground and the type of grouting employed.

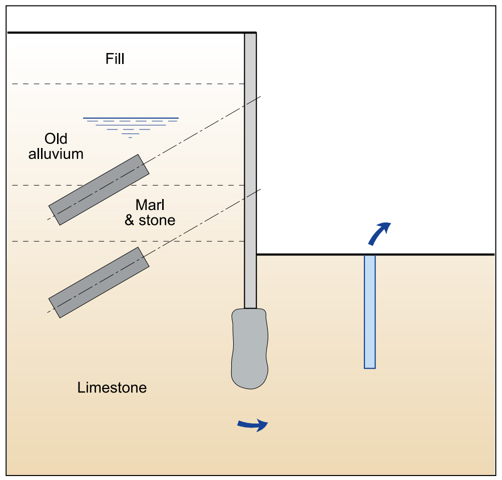
2.4.2. Excavation in rock-like conditions

Solution 1
Pumping only



This solution presents a risk of high inflow if wash-out of the fissures occurs in the fractured rock.

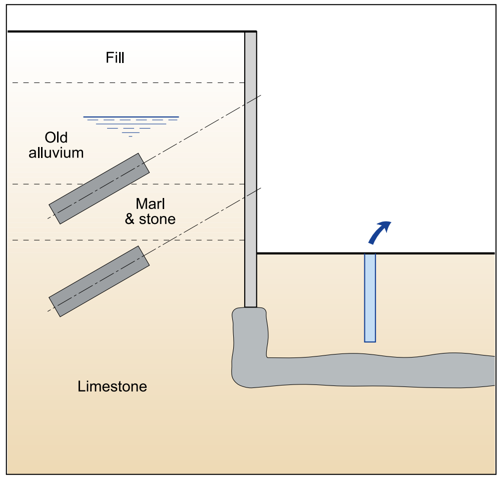
Solution 2
Pumping + grouted cut-off



This is the most usual solution:
- The lateral flow has been addressed; but the rate of flow from the base remains uncertain.
- The ground around the retaining wall embedment is protected, hence reducing the risk of wash-out of the fissures.

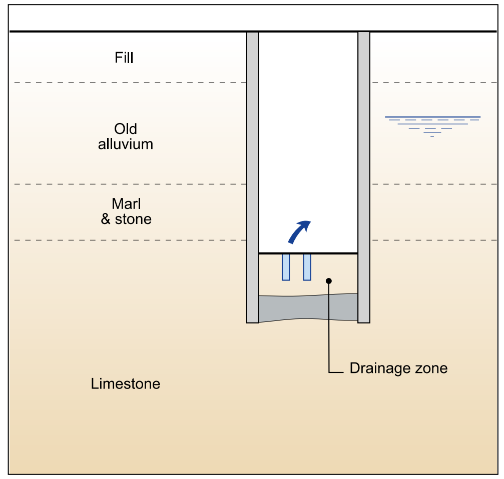
GROUND WATER LOWERING - CUT-OFFS

Solution 3
Pumping and grouted cut-off and grouted mat



With this solution, flow from the both sides and the base are addressed.

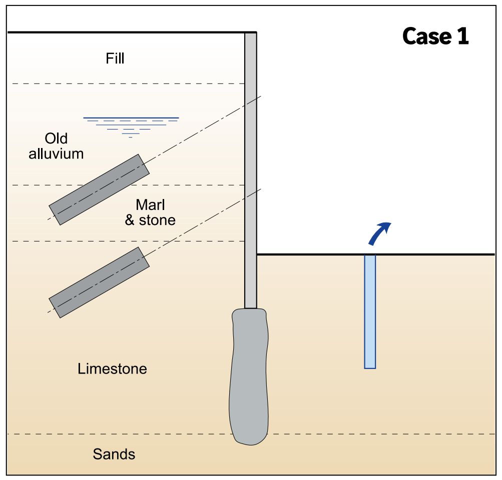
Solution 4
Alternative for «narrow» excavations - arching effect



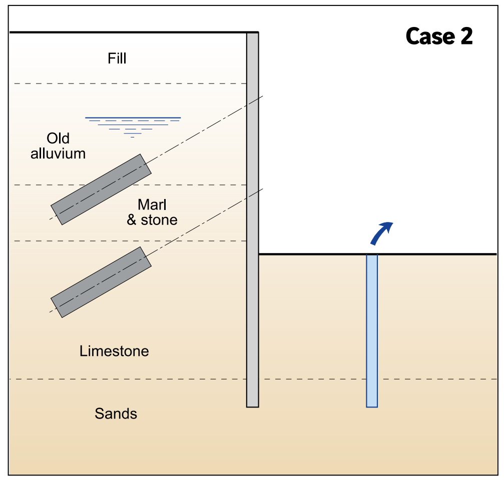
The stability of the grout raft is provided by the arching effect. The hydraulic gradient across the grouted raft needs to be carefully considered.

2.4.3. Base of the excavation in competent ground with a more permeable stratum at depth

The stability of the prism of ground between the base of the excavation and the top of the permeable layer under the effect of the hydraulic uplift forces acting at the interface must be checked.



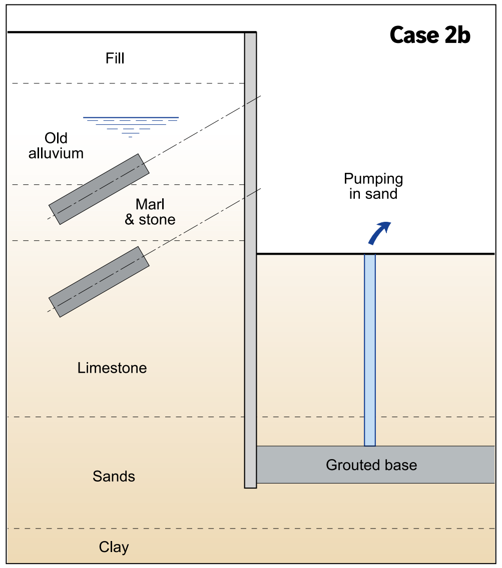
This case is applicable where the hydraulic uplift force at the base of the less permeable ground is counterbalanced by the weight of the overlying ground.

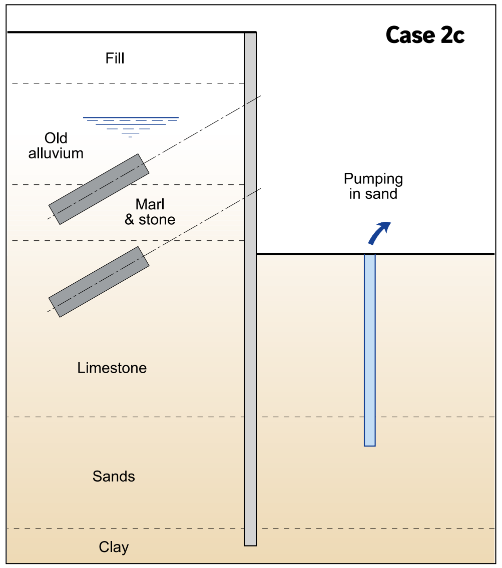


The depth of the prism of less permeable ground is insufficient to provide stability against the full hydrostatic uplift pressure below. The dewatering wells must extend into the underlying sand to reduce this pressure in order to avoid the danger of failure of the overlying prism and erosion of the underlying sand. This precaution inceases pumping rates.

Note: This source of instability can also occur where, for instance, a clayey seam exists at shallow depth within a sandy stratum below excavation level. In these instances it may be necessary to install relief wells to relieve the piezometric head below the seam.

The flow rate can limited by:
- installing a grouted mat in the sandy stratum,
- extending the walls down to an underlying aquaclude if such exists.

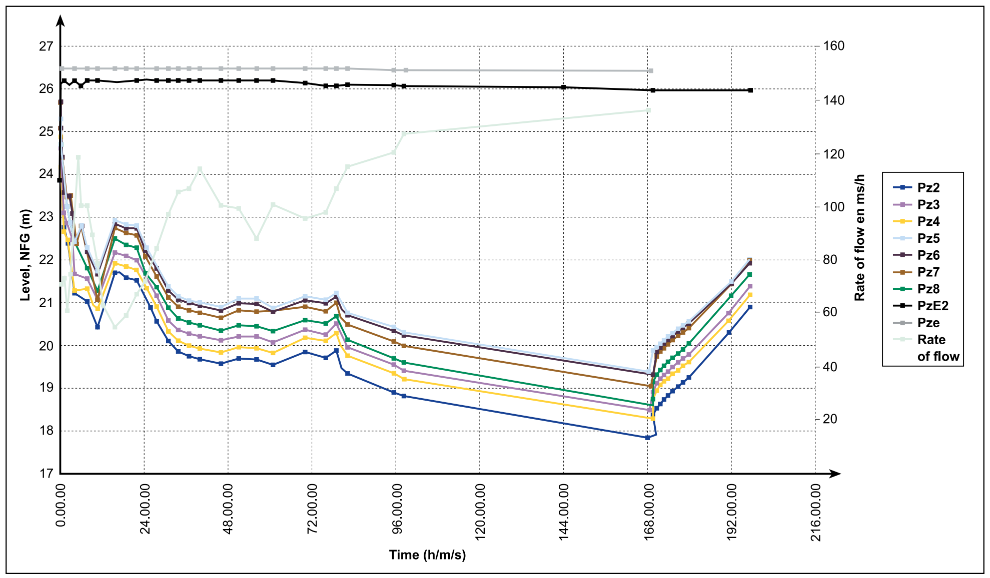




2.5. Commissioning and monitoring

When the works are completed, and before commencement of bulk excavation, a pumping test is carried out.

The pumping test can be interpreted:
- in steady state (constant flow and drawdown). Where the permeability is low or the excavation large, full steady state cannot be reached during a test of realistic duration and an upper bound figure for the specific discharge (flow/drawdown) is assessed from what is termed pseudo steady state.
- in transient flow in which the final phase of the test is to stop pumping and to observe the reaction of the piezometers.



Example of plot produced from a pumping test under transient flow.

The conclusions to be drawn from the results of the test will be very much dependent upon the basic philosophy underlying the original design; the cost/benefit of building in or not a degree of conservatism, the degree to which contingency measures have been anticipated and the management of risk generally. In any event they will include the following:
- whether the work has been carried out in accordance with requirements (e.g. check for continuous embedment of the wall in an aquaclude),
- whether the pumping rate required to keep the excavation dry and its impact on the surroundings are broadly in line with expectations,
- establishment of a value for the specific discharge.

For a given excavation, the flow/drawdown ratio or «specific discharge» should be constant. It is essential to take regular measurements during the entire pumping period. In the case of permanent sub-slab pumping, this period is the life span of the structure.

The loss of material through wash-out or erosion will be reflected by an increased specific discharge.

More radical conclusions or options may arise if the dewatering scheme was designed with this in mind such as the potential benefits of modifying aspects of the project. For example:
- installation of a grouted base or cut-off,
- reviewing the choice between permanent pumping and a tanked base slab designed for hydraulic uplift,
- change in the approach to earthworks. For instance, if the anticiapted flow rate is deemed excessive and no viable solution exists to reduce it, an option may be to carry out the earthworks and concrete the mat foundation (reinforced or not) under water. This method is best suited to relatively shallow excavations. It is often necessary to anchor the slab with micropiles to resist hydraulic uplift forces.

2.6. Behaviour of the structure in service conditions (Choice of base slab design)

The choice of base slab is dependent upon the final dewatering rate:
- a drained base slab with permanent dewatering. In this case the system will require maintenance throughout its life (the cleaning of drains etc.,) and its behaviour needs to be monitored (specific discharge, piezometry).
- a tanked base slab designed to withstand uplift forces. In this case:
  - if the weight of the structure is less than the uplift force, it is possible to anchor it down by using micropiles, piles or vertical tie-rods to mobilise the weight of the underlying ground.
  - even if the dewatering of the excavation during construction is carried under the protection of a grouted mat, when pumping stops the full uplift forces will act on the structure and not the mat.

3. Subjects of concern related to water flow

All water flows result in a seepage force proportional to the gradient.

3.1. Stability of the base of the excavation

Upward water flows are especially dangerous since the seepage force reduces the effective stress, and hence the resistance of the soil to shear. This is extremely detrimental to the performance of retaining walls which rely on passive restraint for their stability.

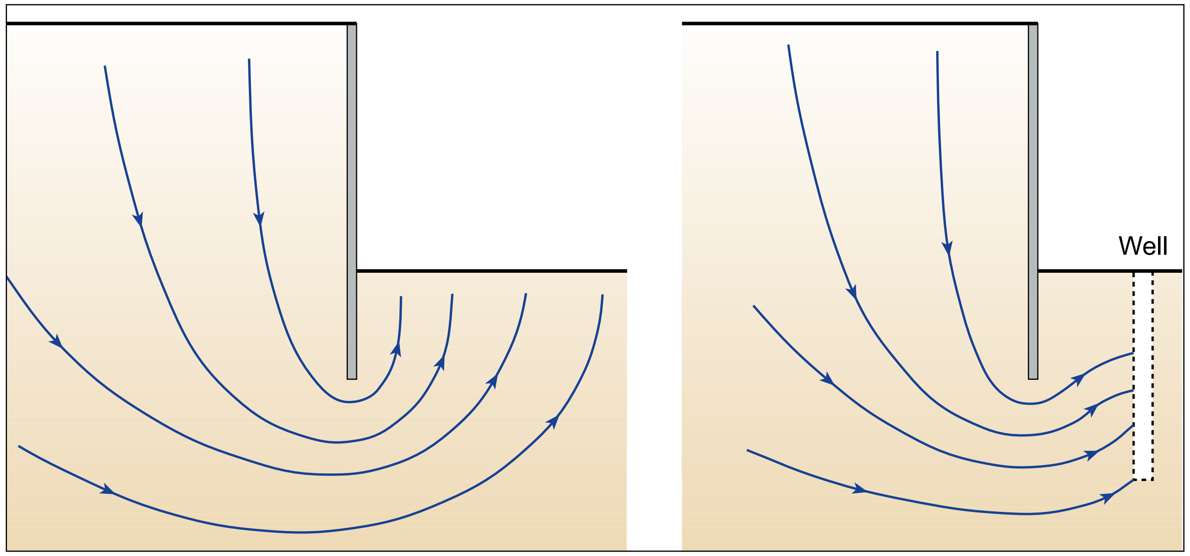
The limiting cases are as follows:
- in granular soils: reduction of the vertical effective stress to zero when the piezometric pressure equals the total vertical stress. This leads to boiling or quicksand conditions,
- in cohesive soil: the occurrence of hydro-fractures in the soil mass.

Water flow and hydraulic gradients beneath the base of the excavation must therefore be understood as far as is possible and, in any case, controlled.

This is generally achieved by the use of cut-offs and deep wells with adequate filters.

Whatever the system - filtered wells, wellpoints, drains, or trench drains - their purpose is to direct the flow lines in order to avoid uncontrolled flow at the base of the excavation.
The effectiveness of the system is dependent upon the reliability of the installation (management of the clogging of wells in particular) and the risk of unexpected pump stoppages (power failure for instance).

The design of the installation is usually based on representative pump tests and the use of finite difference or finite elements simulations.
The need to keep the water flows under control is especially important in sandy and silty soils susceptible to piping erosion. However, it is of concern for all soils types wherever a retaining wall is dependent on passive restraint at the toe for its stability or where there is an issue of base heave.



Role of deep wells

Practical aspects
The objective of a designed dewatering system, as opposed to pumping directly from the bottom of the excavation (surface drainage by sump or shallow trench as used in straightforward earthworks) is to fulfil three conditions:
- no loss of ground,
- dry conditions at excavation level,
- stable base and slopes.

The water is extracted by pumping through filters. The most common methods are by wellpoint or by deep well.

3.2. Liquid Piping or Blow-out

Even at hydraulic gradients below the critical level that result in boiling or quicksand, the progressive wash-out of fines can occur. This can result in liquid piping or blow-out. This localized phenomenon depends on the grain size distribution and can change with time. It causes disruption to the internal structure of the soil. It can occur at locations such as a bore hole that has not been properly grouted up on completion.

In the presence of a blow-out, special precautions must be taken during any pumping to avoid worsening the situation. Uncontrolled pumping will remove more fines increasing the gradients and seepage forces, creating a vicious circle.
Liquid erosion or blow-out constitutes significant risk to the surroundings: sink holes, subsidence, etc.

One of the following measures can be taken:
- pumping after ensuring that an adequate filter, such as sand bags, is in place,
- if the flow rate is unmanageable, allowing the water to rise to the level of the water table to stop the flow and therefore the phenomenon. The problem is then dealt with by other methods (grouting, diving operations etc.).

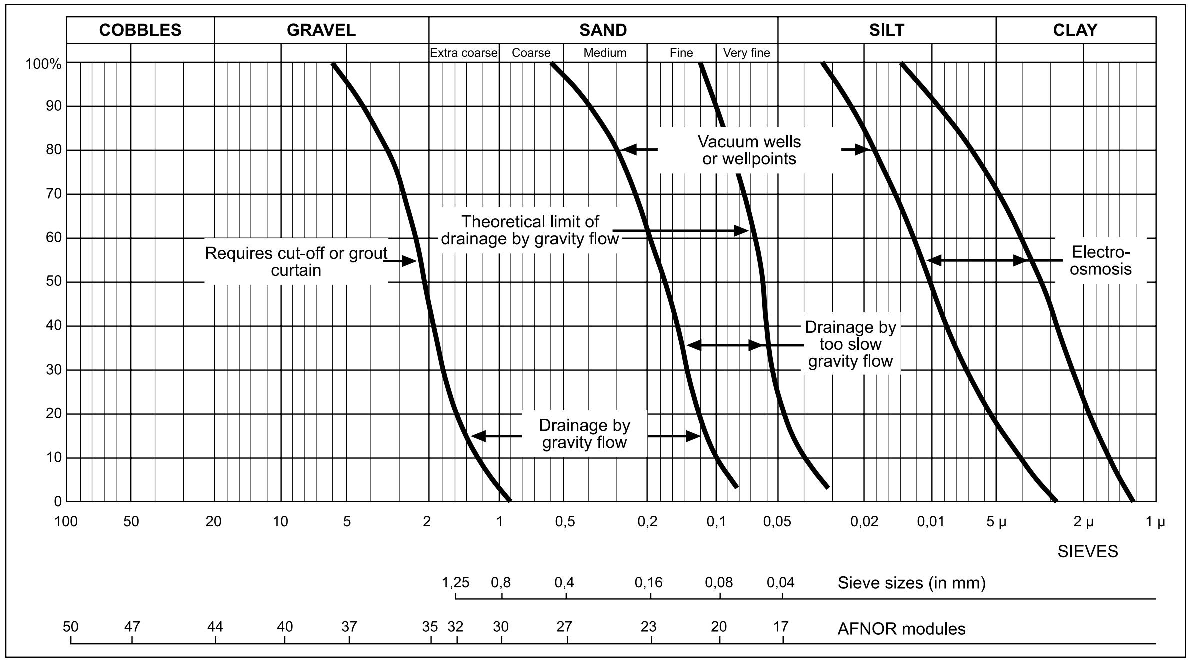
A contingency plan needs to be in place and rapid action is required in the event of an occurrence.

The same phenomenon can occur with a badly designed well equipped with a filter system that allows fine soil particles to pass. It is possible to detect a developing problem by regularly measuring the specific discharge of the excavation and the clarity, or otherwise, of the discharge water. The specific discharge should not increase with time.

4. Groundwater lowering and cut-off methods

4.1. Drawdown: Field of application of the methods

Groundwater lowering techniques are applicable for fairly homogeneous soils with permeabilities greater than approximately 10^{-5}m/s.
They are as applicable to urban excavations with flow rates of a few tens of m^3/h, and to large excavation related to major civil engineering projects (e.g. dams), where the flow rates can reach several thousand m^3/h.
For a typical urban excavation the wells are placed approximately every fifteen meters and can each pump approximately $30m^3$/h.
The diagram below shows the area of application of different extraction methods as a function of grain size distribution.



Groundwater lowering techniques as a function of soil permeability (source: Moretrench Corporation)

The following table describes the various extraction methods

EXTRACTION METHOD	AREA OF APPLICATION	ADVANTAGES	DISADVANTAGES
SUMPS	• Gravels, coarse sands • Dewatering of excavation	• Basic equipment	• Danger of erosion by removal of fines from the ground • Instability of slopes and base of excavation
DEEP WELLS WITH SUBMERSIBLE PUMPS	• Coarse-to-fine silty sands, gravel, fractured rock • Deep excavations • De-pressurising of confined aquifers	• Stability of slopes and base of excavation • No limit to achievable drawdown • Possibility of slotted casing over a large height • Can be placed outside the working area • No noticeable noise if electrically powered • Substantial discharge/well if needed	• High cost of installation • May require substantial installation for handling and treatment of discharge • 24-hour supervision required • 24 hr power required • Standby generators required • Running cost proportional to the duration
VACUUM WELLS	• Sand and gravel, silty sand, fractured rock • Soils with relatively high permeability	• Stability of slopes and base of excavation • Running costs lower than for a well-point installation of equivalent discharge	• Installation can be expensive • 24-hour supervision required • Requires several stages for major drawdown
WELLPOINTS, VACUUM OR NON VACUUM	• Medium to fine sands, silty sands • Dewatering of excavations • De-pressurisation of confined aquifers	• Stability of slopes and base of excavation • Quick and easy to install in suitable soils • Inexpensive • Suitable for a «rolling» site (e.g. pipe laying)	• Difficult to install in coarse gravel, pebbles or cobbles • 24-hour supervision required • Requires more than 1 level for a drawdown greater than 5.50m
PANEL DRAIN	• Water extraction uninterrupted in plan for applications such as stabilisation of landslides • Allows pre-existing hydrogeology to be re-established around a major underground structure by providing effective communication and exposure between the upstream and down stream profiles. • Can be used for the installation of filter gates to treat leachate leaving a confined polluted site	• Possibility to hydraulically isolate short sections of works in progress, hence negligible risk of aggravating an overall stability problem during construction due to drilling fluid surcharge pressure. • Substantial depths possible - up to 20m approximately	• High cost of construction
DRAINAGE GALLERY	• Where high flow rates are anticipated • Stabilization of landslides (with radial drains) • Treatment of any aquifer layer overlying a relatively impermeable bedrock in which a gallery can be constructed	• No limit to drawdown • Levels and flows are easily controlled • Very flexible alignment to optimise performance • Radius of action can be increased by the installation of radial drains	• High initial cost of installation • Risk of blow-out during construction

Note: Automatic recording of piezometric levels and flows is being increasingly used for test pumping and the monitoring of drawdown. Sensitive projects will require an alarm system and automatic surveillance fitted to the pumping equipment.

4.2. Cut-offs

There are two categories:

- grouted raft or curtain,
- positive cut-off walls constructed driving sections into the ground or by replacing excavated soil with a low permeability material.

POSITIVE CUT-OFFS

METHOD	AREA OF APPLICATION	ADVANTAGES	DISADVANTAGES
SHEET PILING	• Poor soil types except scree, coarse gravel, fine sands	• Process widely known and used • Rapid installation • Option to leave as an integral part of structure or to recover after use	• Sheet pile driving is difficult in gravels, fine sands and scree • Noise and vibrations during installation • Expensive if sheet piling not recovered • Not entirely watertight, possibility of windows
CAST-IN-SITU OR PRECAST DIAPHRAGM WALLS	• All soil types • Building with several basement levels • Underground car parks • Pumping stations • Locks, canals • Retention for all types of excavation	• Can be incorporated into the permanent structure • Economical for circular structures (diaphragm walls) • Can be embedded into rock • Little noise or vibration • No problems with corrosion • Can be installed in a tight working area • Can be installed close to existing foundations	• High cost if its only purose is a cut-off
THICK CUT-OFF WALLS (Slurry wall, plastic concrete wall)	• All soil types • Cut-off set back from excavation • Cut-off below dams • Confinement of polluted site	• Speed of construction and low cost • Can be embedded into rock • Considerable depth possible (above 50m) • Low permeability ($\leq 10^{-7}$m/s) • Flexible, can accommodate ground movements	
THIN WALLS	• Silt, sand, gravel	• Speed of construction and low cost • High level waterproofing ($\leq 10^{-7}$m/s) • Flexible, can accommodate ground movements	• Depth limited to 25m • No embedment into rock
SECANT PILES	• All soil types • Buildings with several basement levels • Underground car parks • Pumping stations • Locks, canals • Retention for al types of excavation	• Can be installed in a tight working area • Little noise and vibration • Can be installed close to existing foundations	• Technical difficulties beyond 20m depth • Bending moment resistance lower than for a diaphragm wall • High cost

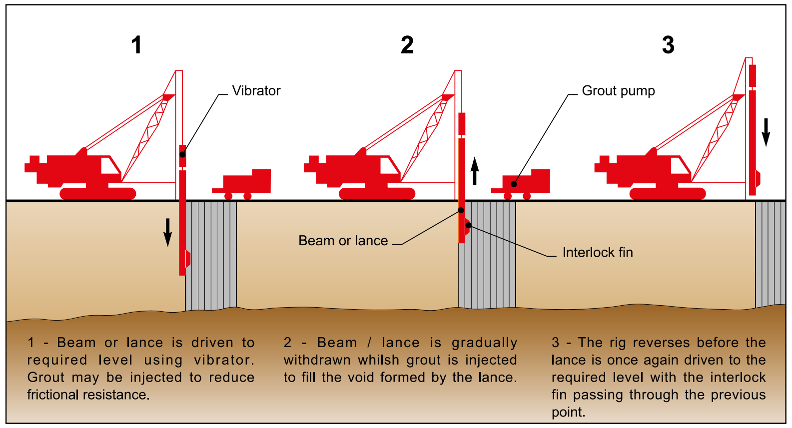
GROUT BASED METHODS

METHOD	AREA OF APPLICATION	ADVANTAGES	DISADVANTAGES
PERMEATION GROUTING	• Rock, permeable soils	• Small diameter drilling • Flexibility, adaptability	• Limited to soils with $k \geq 10^{-6}$ • Result > 10^{-6} - 10^{-7} m/s • Durability of chemical grouts
JET GROUTING	• Soils and weathered rocks	• Small diameter drilling • Flexibility, adaptability	• Problems with tolerance on verticality • Substantial quantities of spoil produced

Thin cut-off wall

The construction of a thin cut-off wall is carried out by vibratory driving of a steel «H» section fitted with grouting tubes. During removal of the section, grout is introduced out at the toe of the section. The imprint of the «H» section is thus filled with grout to form one panel of the wall.

A continuous waterproof wall is formed by repeating the process. In some soils, it is possible to combine the vibratory driving with high pressure jet of grout to assist with penetration of the section.



Vibwall installation sequence

FRANCE - Strasbourg Rhine embankment

5. References

FRANCE - Le Havre - Port 2000

FRANCE - Concarneau dry-dock

INDIA - Chasma Dam

INDIA - Teesta Dam

GROUTING

1. Principle

Grouting involves the injection of a pumpable material (the slurry or grout), which will subsequently stiffen, into the voids in the ground or in man-made material such as masonry, in order to strengthen it or to reduce its permeability.
If the grout material can fill the voids in the ground, the cracks within rock, solution cavities without appreciable movement of the surrounding material the process is referred to as fissure or permeation grouting. If the surrounding material is displaced by squeezing or hydrofracturing, it is referred to as compaction grouting or solid grouting or hydrofracture grouting. This is covered in a separate chapter.
Grouting during which the soil is displaced in a strictly controlled way can be used to prevent potential damage to structures due to nearby excavation (galleries, tunnels, major urban excavations, etc.). This is referred to as compensation grouting and is covered in a separate chapter.

Example of permeation grouting

PARIS - Meteor Project

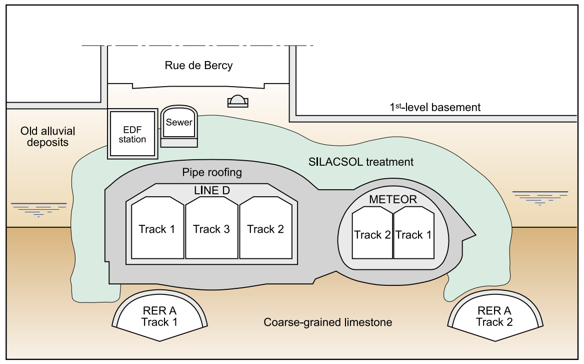
Example of hyfrofracture grouting

TAIWAN - Tzechiang Tunnel

2. Applications

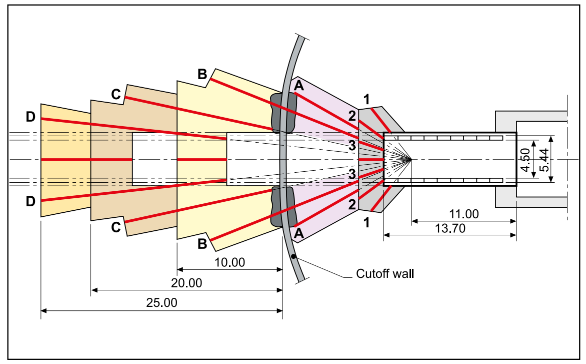
Excavations have always provided a major field of application for grouting, both for strengthening the ground and for reducing its permeability.

Tunnel: treatment before excavation



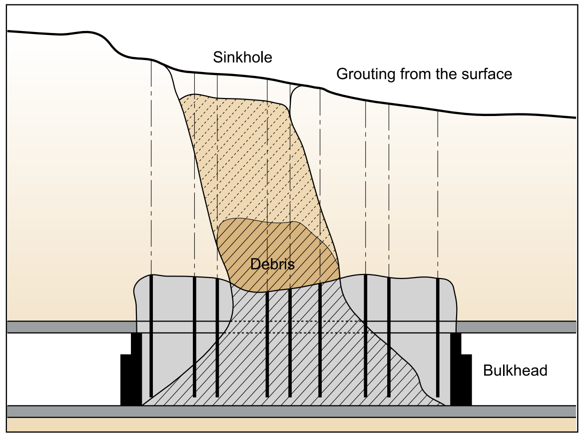
PARIS - D3M10 lines

Tunnel: Advance stagegrouting



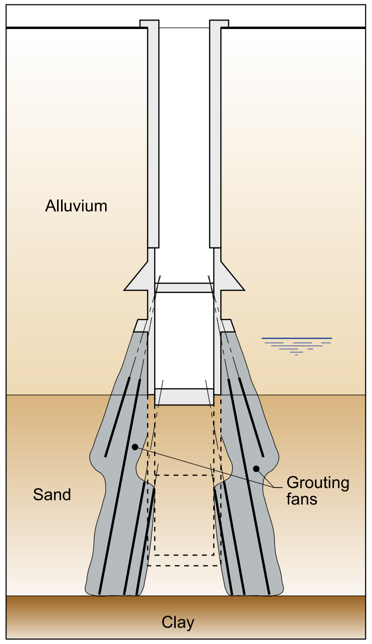
FRANCE - Sangatte - Channel Tunnel

Tunnel: repair works



Consolidation of a sinkhole

Deep shafts

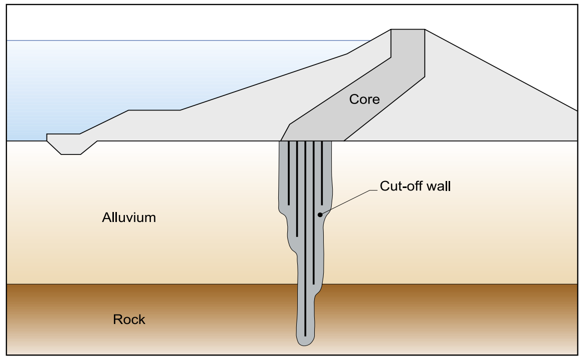


Consolidation / permeability reduction for shaft excavation >

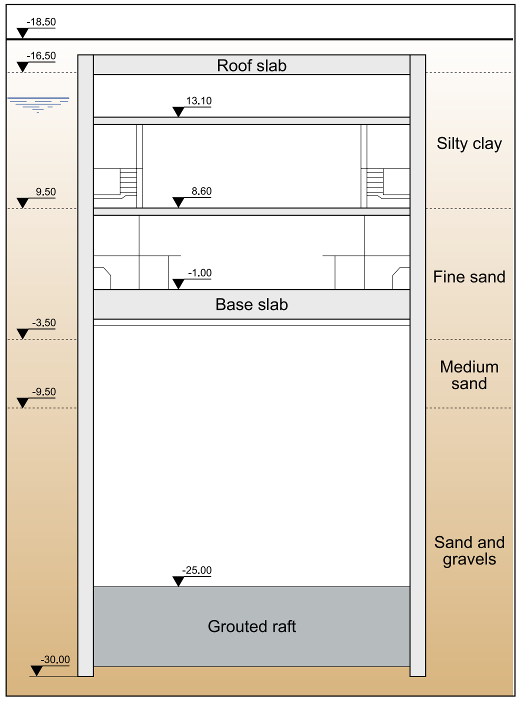
Grouting is the conventional method for achieving the water tightness of dam foundations.

Deep excavations often require a watertight base to keep pumping within acceptable limits.

Dam founded on alluvial soil

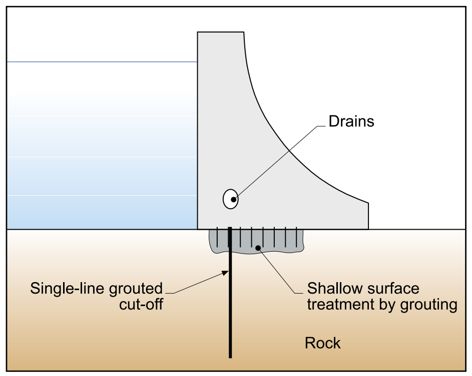


Multiline grouted cut-off in alluvium

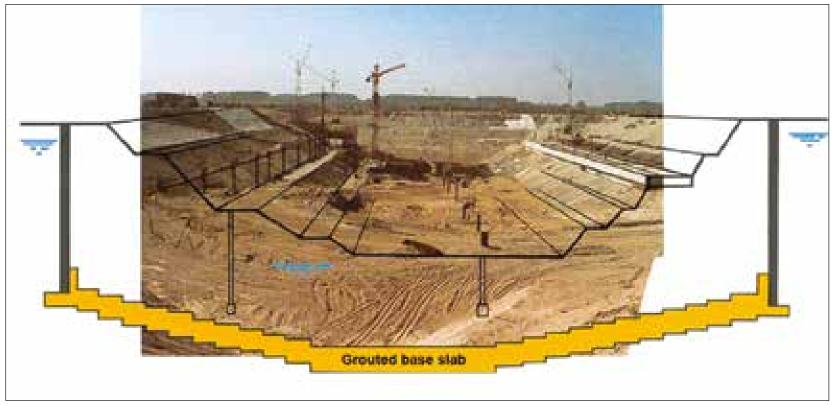


Egypt - Cairo metro
Grouted base slabs for station construction

Dam founded on a rock substratum

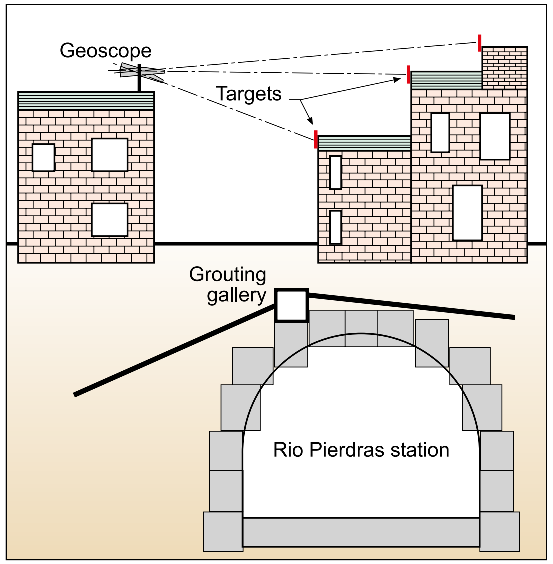


< Single-line grouted cut-off in rock

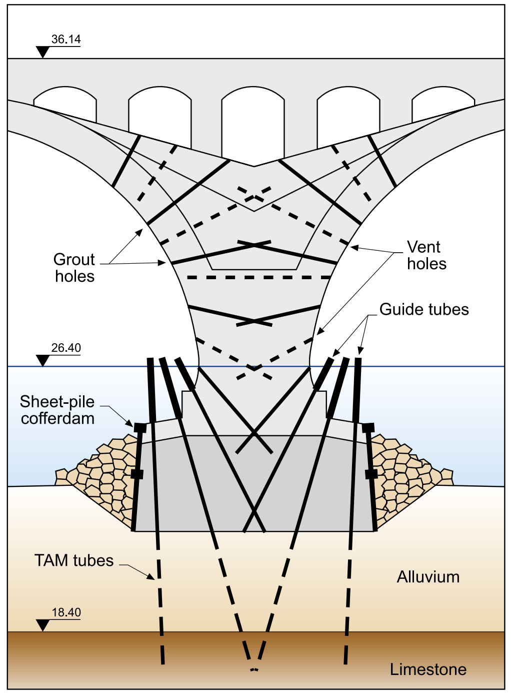


SLOVAKIA Gabçikovo
Grouted curtain in Danube alluvial sediments

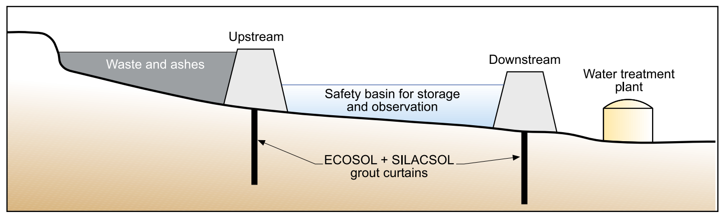
Grouting is also used for protecting structures or for reinforcing foundations, for filling abandonned quarries and for containing materials and soil that are potentially harmful to the environment.



PORTO RICO - Compensation grouting: Compensation of settlement brought about by the construction of the Rio Piedras station



FRANCE - Paris - Bercy bridge
Reinforcement of the foundation of the bridge piers and masonry



JAPAN - Containment by grouting
Containment of a reservoir of polluted water to protect the aquifer

< FRANCE - Maintenance of the Paris Metro
Maintenance of the tunnels and stations outside operating hours

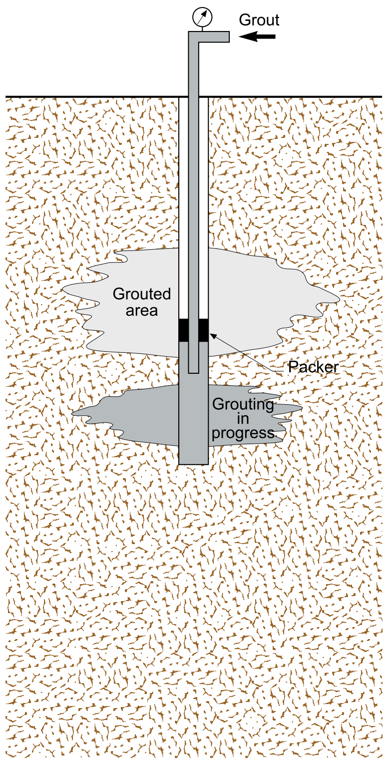
3. Techniques used

The techniques used vary according to the objectives of the treatment and the type of ground.

3.1. Drilling

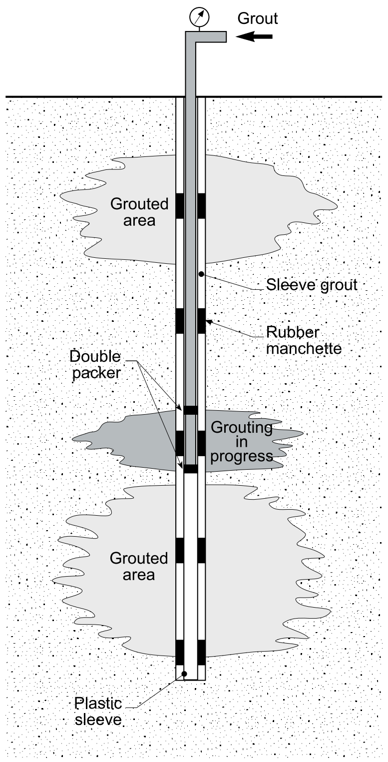
In rock, the borehole walls are stable and the open-hole grouting technique can be used. In loose soils, a sleeve pipe (tube à manchette) will be used, into which the grout packer is introduced.

OPEN-HOLE DOWNSTAGE GROUTING



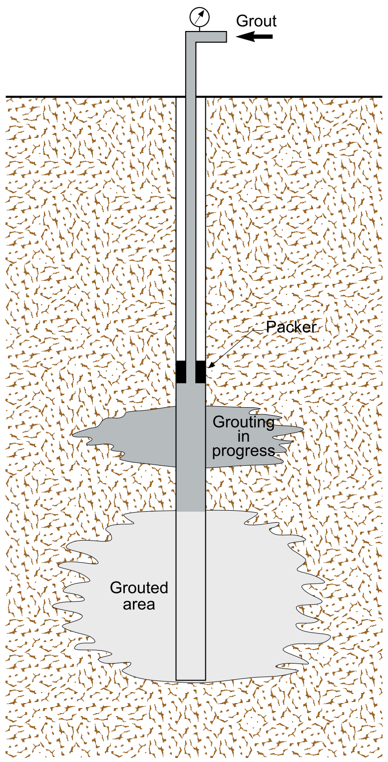
Highly fissured or open rock

TUBES A MANCHETTES GROUTING



Double packer grouting

OPEN-HOLE UPSTAGE GROUTING



Stable borehole along the entire length

The sleeved tube (tube à manchettes) is a tube with a smooth internal surface and with perforations every 30 to 40cm. These perforations are covered by rubber sleeves, called «manchettes», which act as non-return valves. The tube is sealed into the grout hole with a weak bentonite-cement slurry (the sleeve grout) to prevent the subsequent grout from travelling along the annular space.

Grout-hole layout

The spacing of the grout-holes depends on the type of soils, the grout used and the objectives of the treatment: the finer the soil grain-size, the smaller the distance between the grout holes. The table below shows the layouts in various situations:

STRUCTURE	GROUND TYPE	GROUT HOLE LAYOUT
Grout curtains	Alluvium	2 rows of grout holes minimum spacing between grout holes: 1 to 3m
	Rock	1 to 3 rows of grout holes spacing between grout holes: 1.5 to 6m
Mass grouting	Alluvium	Grout hole layout : 1 x 1 to 3 x 3m
	Rock	Grout hole layout : approx. 3 x 3
Impermeable foundations	Alluvium	Grout hole layout : 1.5 x 1.5 to 3 x 3m
	Rock	Grout hole layout : approx. 3 x 3

3.2. Grouts

There are several types of grouts:

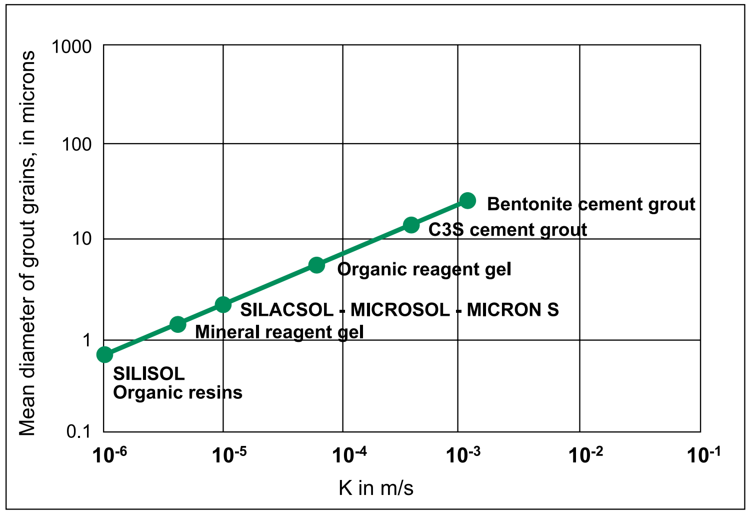
- liquid grouts: their ability to penetrate is a a function of their viscosity, and the change in viscosity over time.

- suspensions: in addition to viscosity, these grouts possess rigidity or cohesion, which restricts their radius of action. The voids or pores that can be sealed with these grouts depend on the size of the grains in suspension. Broadly, it is considered that there should be a minimum ratio of three between the size of the void and the grain size of the suspension.

The stability of a suspension (decantation, pressure filtration) is an important grouting parameter. An unstable grout behaves in the same way as hydraulic fill where the water, which provides the mobility of the mix, progressively bleeds out.

- Mortars: mortar grouts have high rigidity and are used for filling large voids and cavities, or for grouting where soil displacement is the objective: solid or compensation grouting

Grout penetrability versus soil permeability is shown in the chart below:

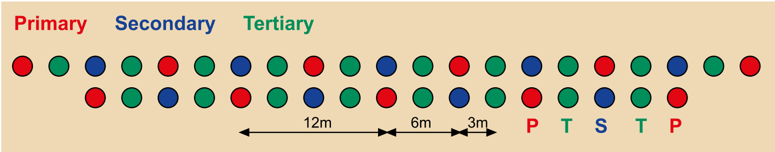


Grout penetrability limits based on soil permeability

Our Materials Laboratory is able to design grouts upon request for specific purposes and has developed substantial expertise in fine-powder grouting and sorbing grouts (ECOSOL® and PETRISOL).

3.3. Rock grouting

The most widely used method in rock grouting is «split hole grouting», whereby primary boreholes are drilled, with the spacing of 6m for example, followed by secondary boreholes (with the same spacing as the primary ones), followed by tertiary boreholes (with half the spacing) until the desired permeability objectives are achieved. These are generally expressed in Lugeon units (for measuring rock permeability). The Lugeon test is a standard test for measuring the amount of water injected into a segment of the borehole under steady pressure. In terms of hydraulic conductivity, a Lugeon unit is approximately equivalent to 1 to 2.10^{-7}m/s.

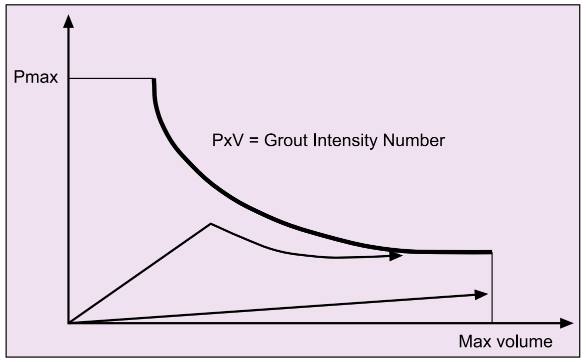


Borehole gradient and orientation are adjusted to suit the rock fissuring

The final spacing between boreholes depends on the rock characteristics and the objective to be achieved. It can vary from 0.75 to 3m.
Stage heights can vary between 3 to 5m but can be as much as 10m.

The Grout Intensity Number (GIN) method proposed by Prof. Lombardi is becoming increasingly widespread particularly since the process has become computer-controlled (see chapter 4). This method balances the pressure and quantity parameters needed to obtain local radii of action that are virtually the same, regardless of the degree and extent of fissuring.

GIN method
The pump flow rate is automatically adjusted to follow the GIN curve as closely as possible.



3.4. Alluvial grouting

The sleeved tube (tube à manchettes) method is generally used for alluvial grouting, with 3 manchettes per meter. Grouting is usually performed in two stages: in the first stage, bentonite cement grout is injected to fill the larger voids. This is followed by the injection of a liquid or ultrafine suspension grout with higher penetrability. The principle parameter to be determined is the amount of grout required to achieve optimum filling of the pores.

3.5. Grouting volume

The chart below gives indicative grout percentages of volume required according to ground and treatment type:

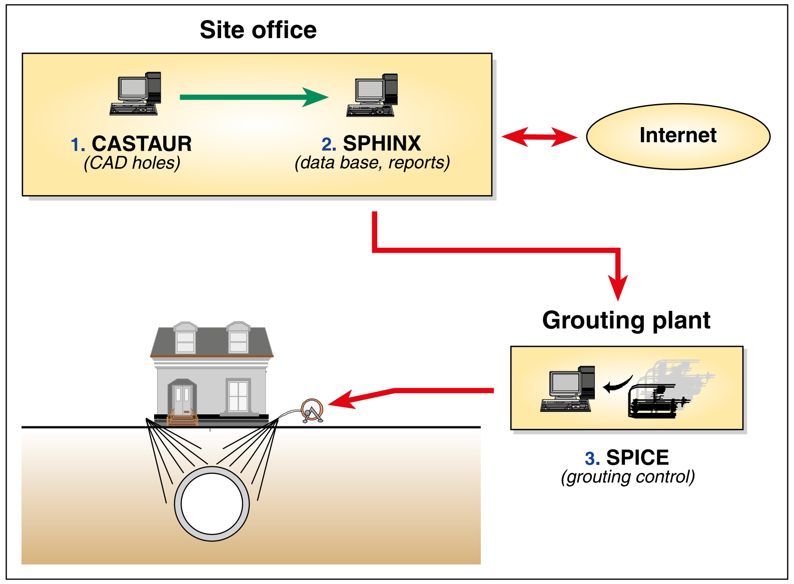
Sands and gravels	25 - 35% soil volume
Fine sand	35 - 45% soil volume
Fisured rock	5 - 15% soil volume
Base slab in chalk	8 - 25% soil volume
Hydro fracturing	10 - 20% soil volume

Range of grouting volumes

4. Grouting control system

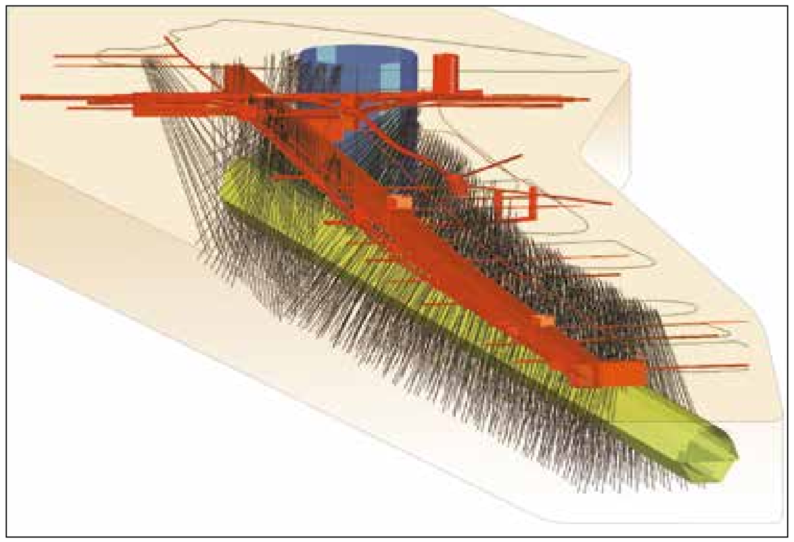
Good quality control during grouting is a key factor for the successful completion of the process. Soletanche Bachy has its own in-house control system, called SPICE (or GROUT I.T.® in the USA), the development of which started in the 1980s. The system is essential for the management of the vast amount of data required and generated at every stage of the process: the setting-up stage (grout hole geometry and the calculations of the volumes of grout needed) the acquisition and injection settings, controlling the grouting plant (monitoring the pumps, acquiring flow-rate and grouting-pressure data), and tracking quality and production.

The control-system software package developed by Soletanche Bachy comprises a suite of interactive programmes:
- CASTAUR groups together all the site investigation data and determines the location of grout holes.
- SPICE is installed in the grouting plant and controls all grouting operations, including the electro-hydraulic grout pumps.
- SPHINX organises all the grouting data and presents them in a graphic format.



Grouting control system

The advanced features of this package make it enormously helpful in the grouting process:
- **Safety**: data collection and subsequent control of operations in the field are extremely precise and rigorous.
- **Quality**: powerful synthesis and analytical tools produce fast, automatic reports and charts ;
- **Performance**: higher production rates, and efficient control that is able to keep up.



< CASTAUR
3D modelling of grout holes in a particularly complex location: the green shows the grouted soil surrounding the tunnel in its construction phase. The red area shows existing parts of the system and the blue area the access shaft. The black lines are the boreholes.

CASTAUR - Grout hole pattern

The CASTAUR program is used to create a three-dimensional model of the grout hole pattern, factoring in multiple constraints: drilling machine geometry, site location characteristics, hidden obstacles (foundations, utilities, etc.), grout volume, geology, etc.

CASTAUR is used to define and optimise the geometric arrangement of grout hole fans and manchettes in the most complex situations. It is also a highly versatile program, able to react very quickly to any changes that may take place during the project.

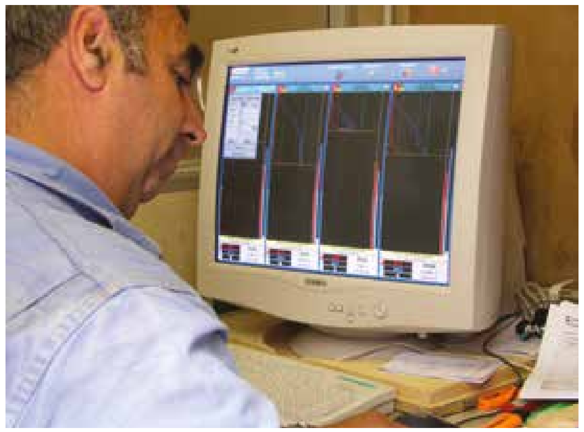
Computerised grouting plant

SPICE - Automated grout plant control

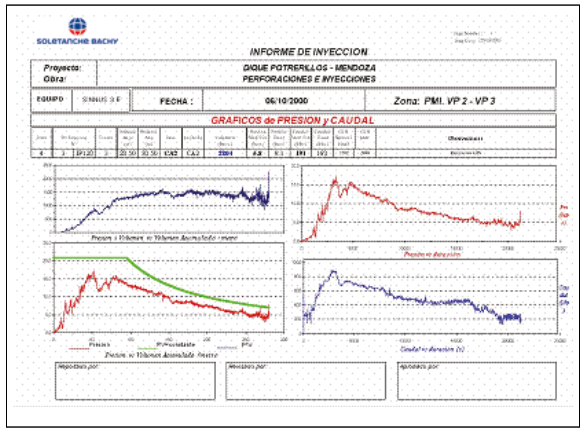
SPICE system features:

- each grout pump has pressure and flow sensors connected to an electronic control box which is programmed for rapid acquisition of signals and precise regulation of grout flow,
- the SPICE program is installed on an industrial PC which supervises the grouting sequence and the start and end of each grouting pass, in accordance with preset criteria: volume, pressure, Lugeon or GIN criteria, etc.

SPICE is of major value for site management: ongoing supervision of pumps, control of grout flow, strict compliance with stop criteria, reliability of recorded data.
A plant with 12 grouting points can be supervised by a single employee, while the time spent in switching from one grouting stage to another is reduced.



SPICE
Grouting plant computerised control system



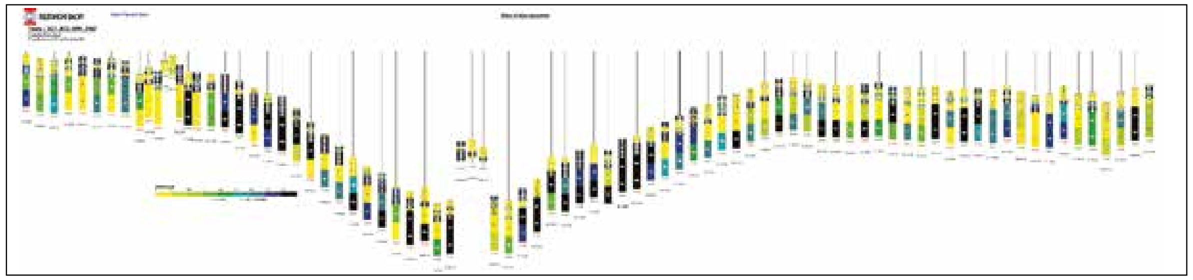
GIN grout volume monitoring

SPHINX - Grouting data management

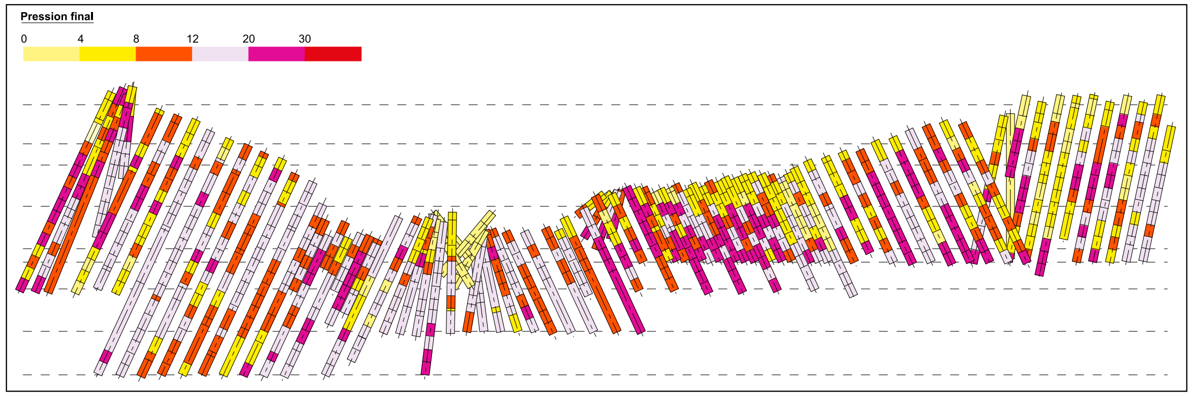
SPHINX initially creates grouting instructions based on grout-hole geometry (defined using CASTAUR) and the sequence of operations. It is subsequently used for acquiring and validating data from each workstation which were recorded by SPICE in the grouting plants.

SPHINX records all the data in a database and subsequently allows multifaceted analysis of the data:

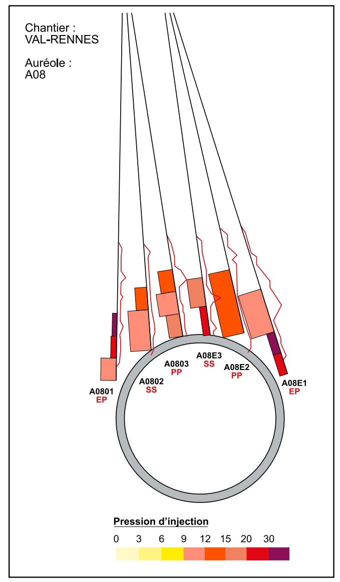
- production reports or reports specified by the contract,
- multi-criteria analyses by fan, by zone, by grout hole in the form of graphs giving a comprehensive overview of non-grouted zones, zones to be re-grouted and the general status of the works.



SPHINX
Graphic display of grouting results (St Ferréol dam)



Symvoulos - Upstream line



Val-Rennes
grouting rue d'Orléans

JET GROUTING

1. Principle

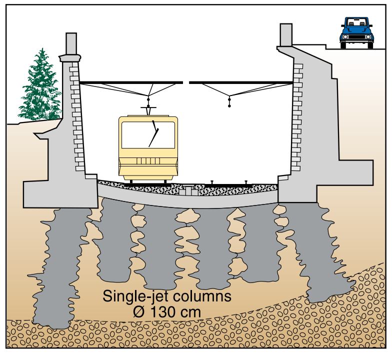
JET GROUTING is a construction process that uses a high-pressure jet of fluid (generally 20 - 40 MPa) to break up and loosen the soil at depth in a borehole and to mix it with a self-hardening grout to form columns, panels and other structures in the ground. The parameters for the JET GROUTING process and the final strength of the treated soil depend on a number of characteristics, such as the soil type, the technique used and the objective to be attained.

In granular soils, the high-pressure jet breaks up the grains through erosion, while in a cohesive soil, such as clay, the jet breaks the mass up into manageable fragments. High pressure is needed to produce the kinetic energy required for the jet which is generated through a small-diameter nozzle. Waste material from the process (a mix of soil, water and binder) is recovered at the surface before being taken away for disposal.

2. Applications

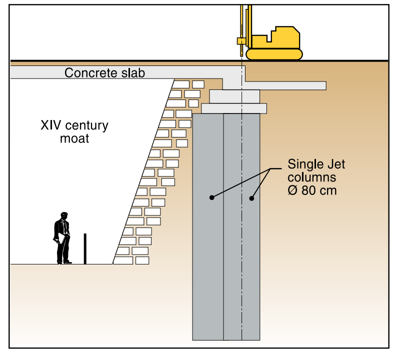
The process can be used in all loose or soft-rock soils to reinforce them or, in certain cases, to reduce their permeablity. Application include underpinning buildings, dam cut-off walls, retaining walls, pipe roofing for tunnels or reinforcement of side-walls, reinforcing quay walls, etc.

When JET GROUTING is used to reduce the permeability of the soil, it is sometimes necessary to use an additional grouting treatment, depending on the final result required.

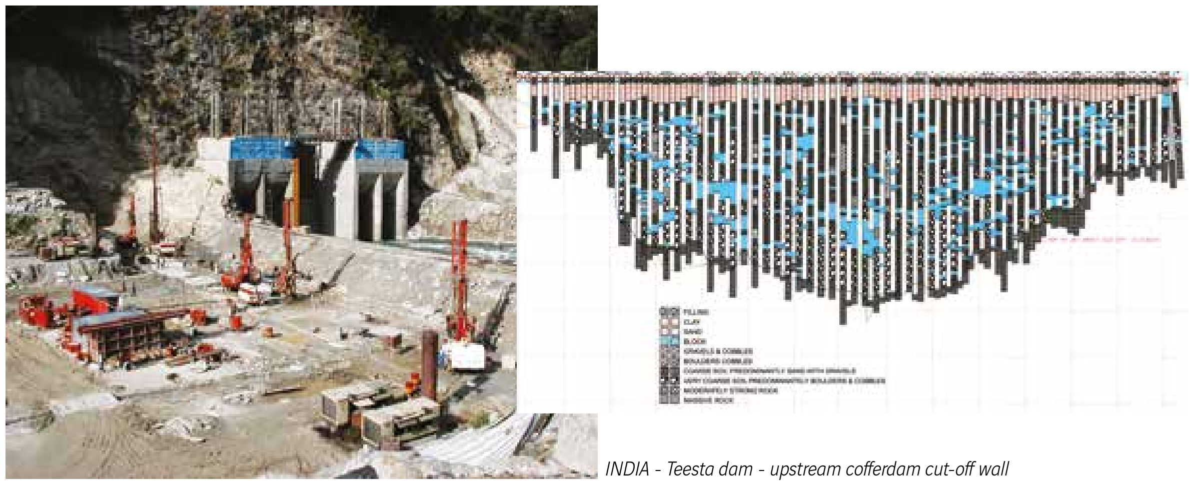


FRANCE - Paris - RER C

FRANCE - Concarneau - Quay wall



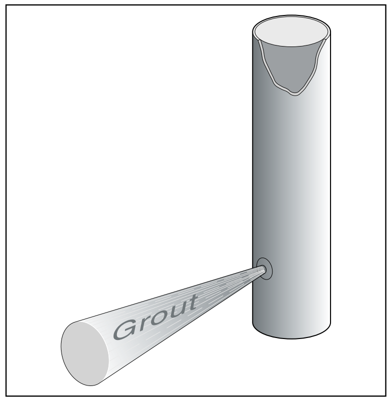
FRANCE - Paris - Louvre Museum



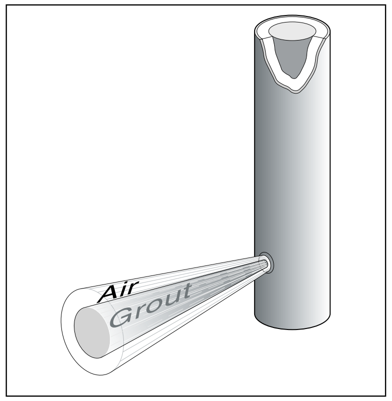
INDIA - Teesta dam - upstream cofferdam cut-off wall

3. Techniques employed

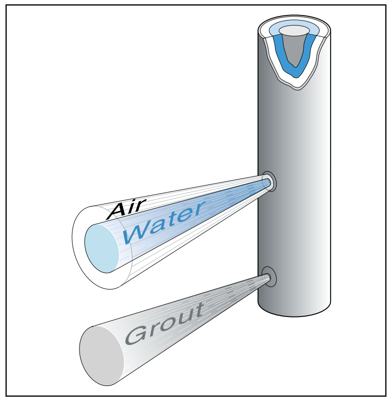
As mentioned above, jet grouting consists in drilling a borehole, then pumping high pressure fluid through nozzles at the bottom of the drill string, at preset raising and rotation speeds, to obtain the required soil-cement structure.
There are three main techniques:



SINGLE FLUID JET GROUTING
Grout is pumped at high pressure through a set of nozzles located just above the drill bit. The jet of grout breaks up and binds the surrounding soil.

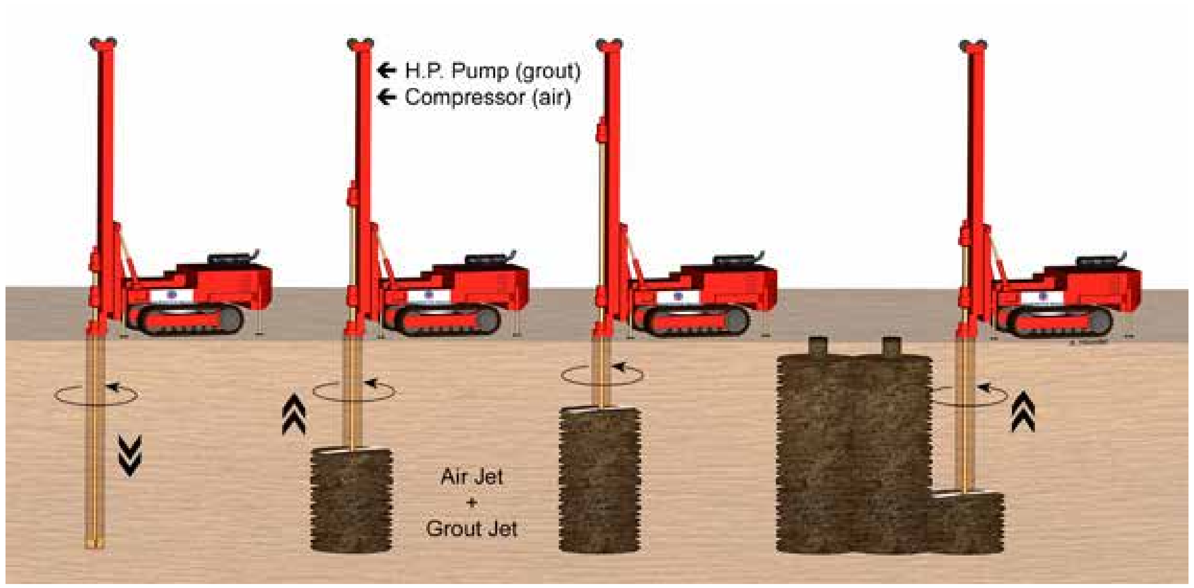


DOUBLE FLUID JET GROUTING
Grout is pumped at high pressure, surrounded by a concentric jet of compressed air, which enhances its erosion efficiency.



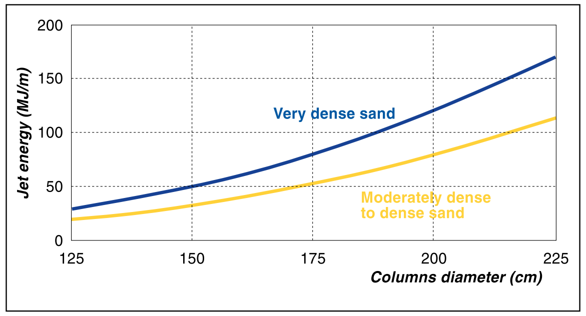
TRIPLE FLUID JET GROUTING
The soil surrounding the drill string is broken up by a high-energy jet of water surrounded by a concentric jet of air, while the binder is injected through a second nozzle.

The column or cylinder is the most commonly used soil-cement element:

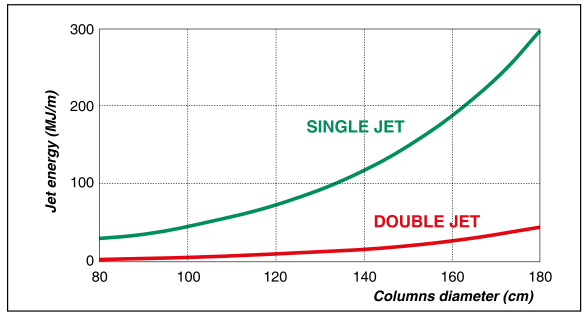


The method used depends on the type of soil and the required diameter and strength. Use of the technique requires specialist knowledge. For a given soil, the result depends on the energy, E, used per metre:
E = P x Q / V (P : jet pressure, Q : jet flow rate, V : raising speed)

Examples of jet energy vs. Column diameter:



Double flow jet in sand of varying densities



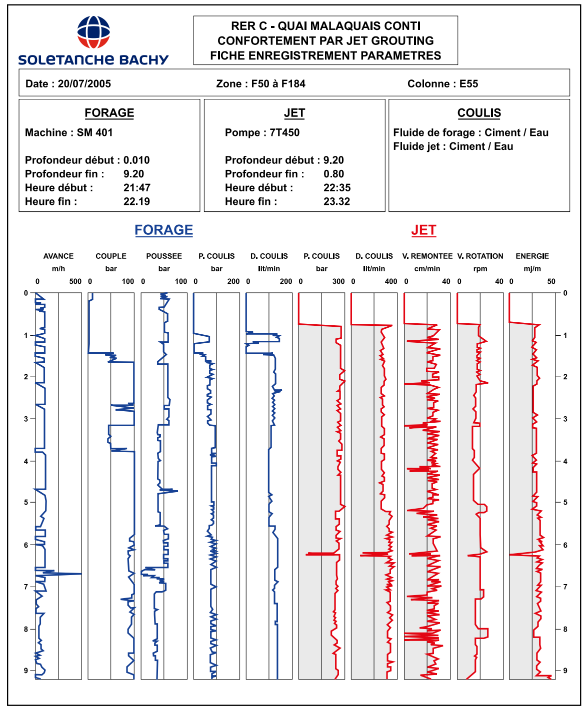
Single and double flow jet in soft, peaty clay

For a given jet energy, the choice of parameters (pressure and flow rate of the various fluids, rotation and raising speed, diameter and number of nozzles) will depend on the particular features of the job in hand, the capacity of the high-pressure pump, the experience of the engineers and analysis of the jet-grouted test columns.

4. Monitoring

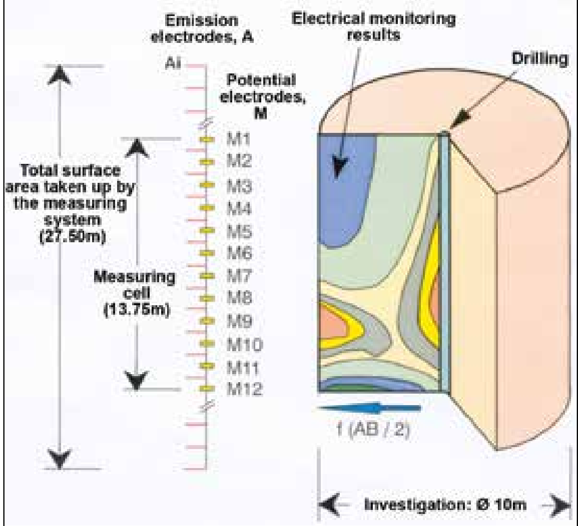
The process is monitored by the SYMPA system, which logs the drilling, control system and JET GROUTING parameters.

DENMARK- Comet
Drilling rig fitted with the SYMPA system



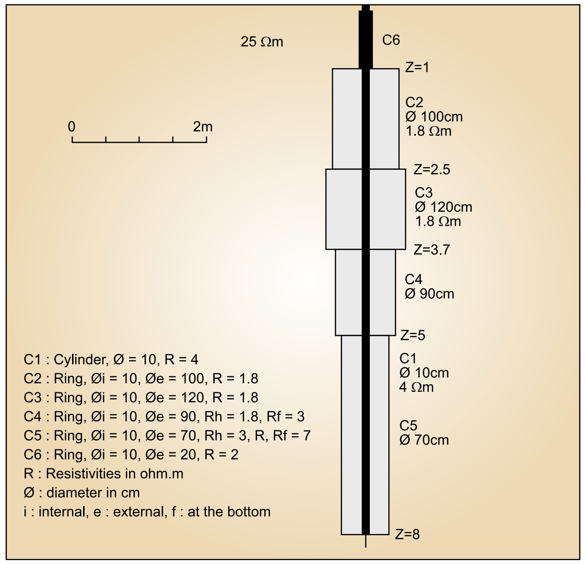
FRANCE - Paris - RER C
Reinforcement by Jet Grouting parameter log file

Column diameter can be measured by the CYLJET electrical cylinder (developed by Sixense Geophysics). A measuring probe is sent down through a tube placed in the column, either in the fresh grout or through a re-drilled bore hole.



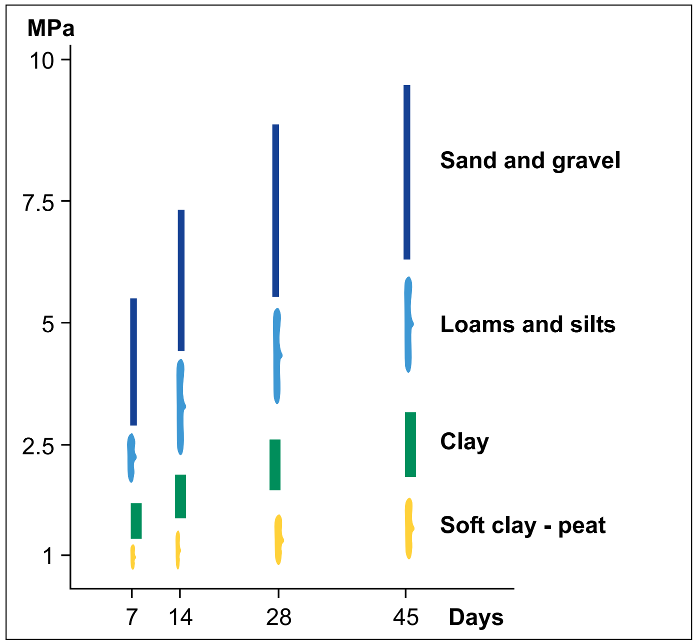
1 - Des électrodes sont introduites dans un tube crépine
2 - Un courant électrique est envoyé dans le terrain
3 - Des différences de potentiel sont mesurées

Sixense Geophysics electrical cylinder method



SWITZERLAND - Geneva
Jelmoli: measuring result

The characteristics of the final composition of the column can be checked by coring (the results can show substantial dispersion) or can be found by correlation with the characteristics of the spoils recovered at the surface when the columns are installed.

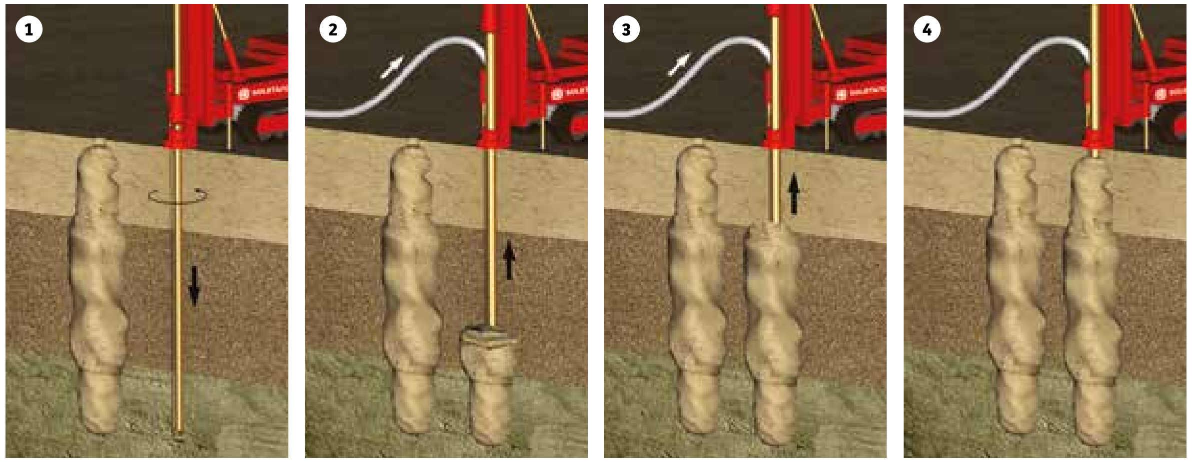


Resistance according to soil type

COMPACTION GROUTING

1. Principle

Compaction grouting is a process employed for increasing the density of the soil by injecting a stiff, mortar-like grout under high pressure through cased boreholes. The grouting is usually carried out bottom-up, in successive stages of about 1m.
As the grout is pumped in, it gradually forms a bulb which pushes the surrounding soil to the side, thereby increasing the relative density of the soil. The degree of densification depends on the type of soil treated and the grid pattern for the injection points.
Injection rates generally vary from 4 to 6m^3 per hour, reducing to 2m^3 per hour in particularly sensitive conditions.
Injection pressure is generally in the range of 1 to 4MPa.



Placement of mortar

2. Applications

Compaction grouting is used for treating a wide variety of loose soils (Pl under 0.7MPa), with relatively good drainage.
Compaction grouting can be performed at depths ranging from 2 or 3m, right down to several tens of meters.

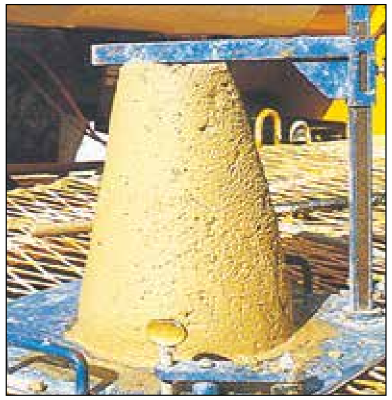
The work can be carried out from the surface, from an existing basement or locations with limited headroom. It is also possible to drill through hard material to gain access for treatment of low strength strata beneath.

The only requirement is sufficient room to be able to drill holes of approximately 120mm diameter.

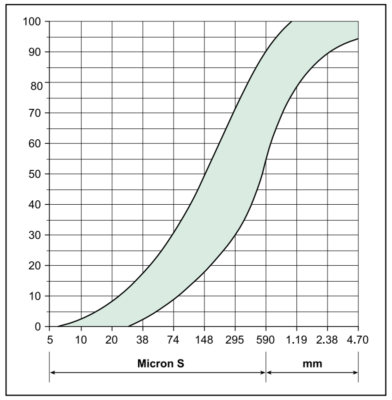
3. Grout material

The mortar grout must meet the following requirements:
- it must be pumpable
- it must not cause soil fracturing
- it must not «bind up» leading to refusal of grout before the injection process is complete.

The grout must therefore have an appropriate slump and grading. The main constituent is a sandy material, often with added fines (cement, fillers, etc.). The usual slump value is less than 10cm.



Checking the slump



Example of suitable grading curve for mortar

Vue of mortar bulb in fine sand

4. Process parameters

The key parameters for compaction grouting are the grid pattern, the injection pressure and the grout take.

Grid pattern

The grid pattern is devised such that each drill hole nominally treats a given area in plan. The grid can be square or triangular and generally makes the distinction between primary and secondary (and sometimes tertiary) holes. The grid is determined by the type of treatment required (localized or en masse) and the radius of influence (Ri). The radius of influence is the distance from the center of the drill hole to the furthest point at which there is a change of void ratio as a result of the treatment.

The table below gives an idea of the possible range of radii of influence «Ri».

Soil type	Radius of influence «Ri»
Clays	0.2 to 0.3m
Silts	0.5 to 1.0m
Sands or gravels	1.5 to 3.0m

Grout take « τ «

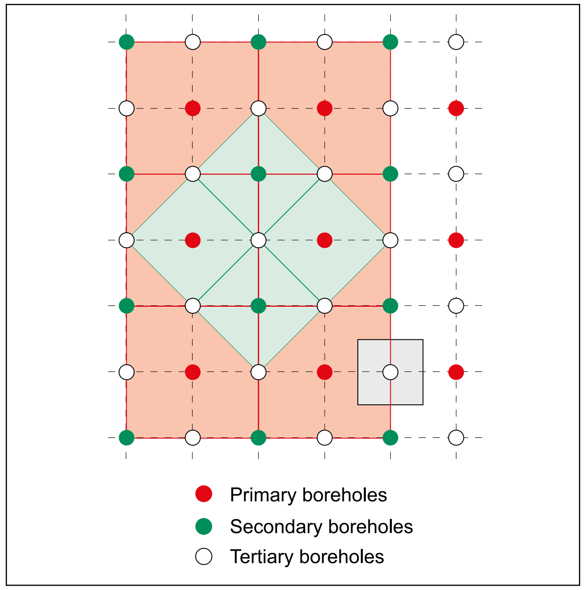
Grout take is the volume of mortar injected expressed as a percentage of the volume of soil treated.

$$\tau = \frac{Vi}{Vt}$$

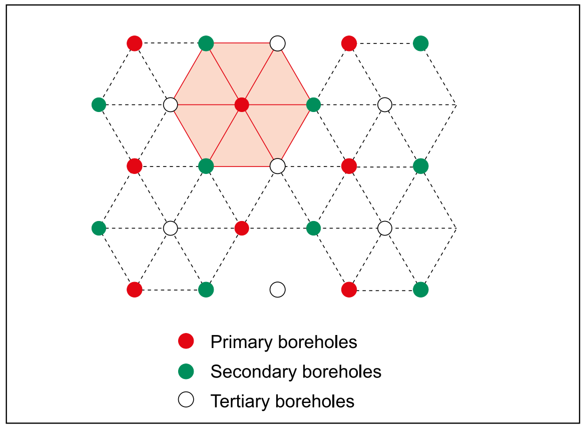
« τ « can also be expressed by reference to the void ratios before (e_o) and after (e_f) treatment.

$$\tau = \frac{\Delta e}{(1 + e_o)(1 + e_f)}$$

e_o and e_f can be determined from D_{ro}, the initial relative density of the soil, and D_{rf}, the final relative density, which can be estimated from in-situ SPT or CPT tests.



Square grid pattern



Triangular grid pattern

COMPACTION GROUTING

Injection pressure

The injection pressure is dependent on the specific site conditions: presence of buildings, civil engineering structures, open site, treatment depths, etc.

Generally speaking, pressure is prescribed by depth in stages at 1 bar (100Kpa) per meter of depth measured from the bottom of the stage.

Most compaction grouting is performed using grid patterns of 4 to 9m^2 with grout take varying from 2 to 6%.

In the particular case of sinkholes, grout takes are highly variable and have been known to be as high as 14%.

5. Controls

Controls during the works

- grout test and slump monitoring
- drilling parameters by reference to the automatic recording results
- injection parameters by recording injection pressure and rate, as well as the total volume injected.

Controls after the works

The results of the treatment are generally analysed by reference to pressuremeter or penetrometer tests.

These tests should be carried out as late as possible in the treated areas, especially in soils with poor drainage. Meaningful analysis is dependent on the same type of tests having been carried out before commencement of the works.

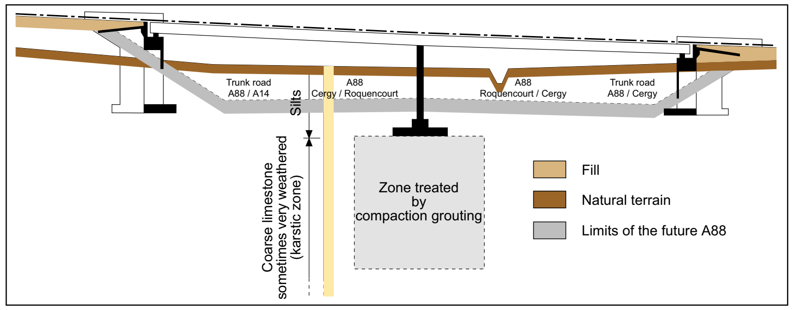
6. Plant and equipment

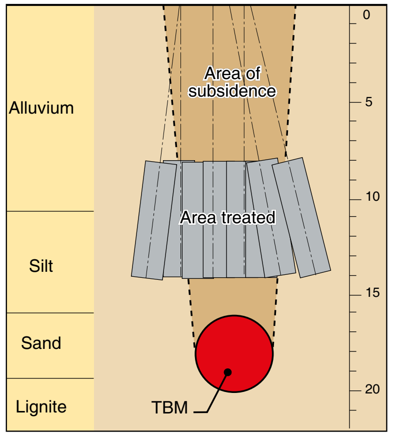
- KOS (Putzmeister) - type mortar pump or similar
- drill rig of any type, including crane-mounted leader
- mortar storage facility, with conveyor belt if necessary
- accessories for recording of parameters

FRANCE - Béziers - A9 highway

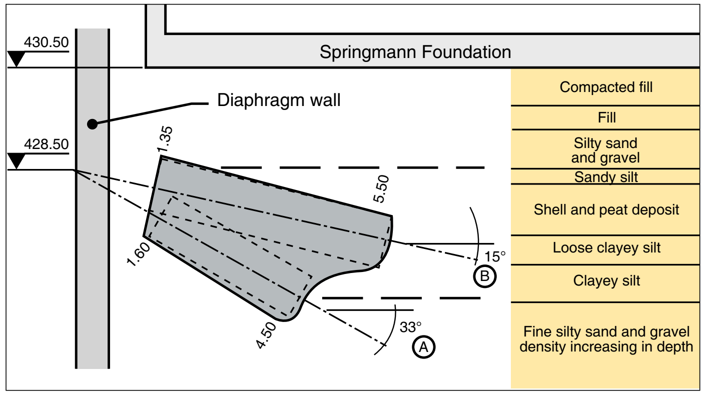
7. Examples of compaction grouting

Consolidation of soil under a bridge pier
Orgeval A14 (new structure)
Treatment of coarse-grained limestone, sometimes highly weathered, with silty or sandy backfill.

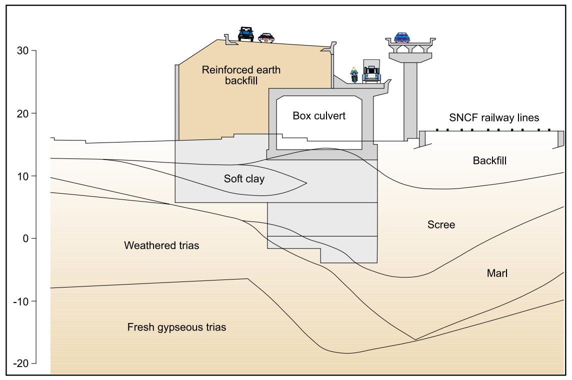




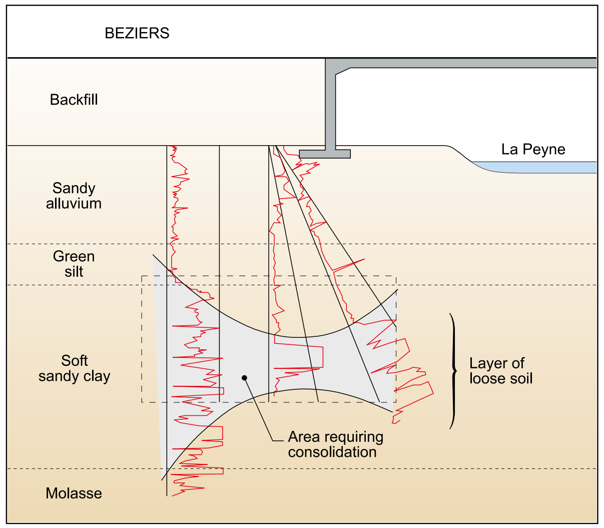
Treatment of a sinkhole directly above a tunnel boring machine (TBM), prior to resumption of tunneling
Treatment of very loose silt, following the appearance of a sinkhole.



Consolidation of soil under a building Neufchâtel
Treatment of loose silt prior to digging a trench under the building.



St Lambert - Nice - Traffic intersection
Treatment of an area of backfill and scree under a traffic intersection.



Pezenas - Treatment of backfill
Treatment of a soft sandy clay area.

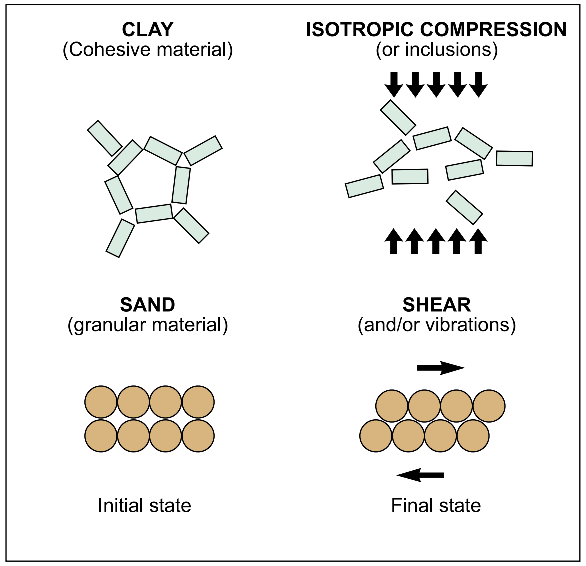
SOIL IMPROVEMENT AND SOIL REINFORCEMENT

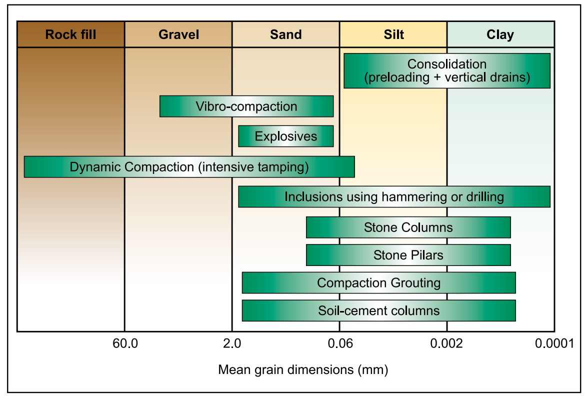
1. Principle - Domain of application

Soil-improvement techniques involve changing soil characteristics by physical action, such as vibration, or by the inclusion or mixing into the soil of a stronger material.
The aim of this process is as follows:

- to increase the load-bearing capacity and/or the shear strength,
- to reduce both absolute and differential settlements or in certain cases, accelerate them,
- to mitigate the risk of liquefaction in the event of an earthquake or major vibrations.

The scope of application of the various techniques depends mainly on the type and particle size of the soils that require improving.





Improvement methods depending on particle size

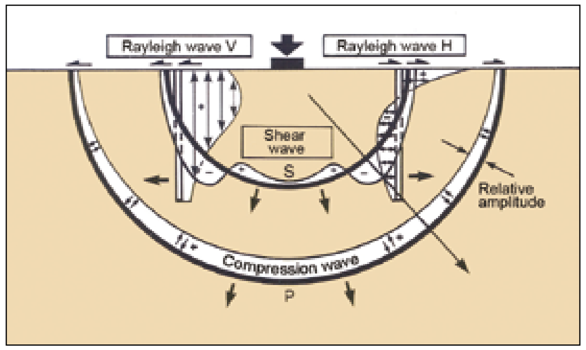
2. Dynamic compaction (high energy tamping)

The principle consists of releasing, in repeated free fall, a weight of several tons from a height of ten meters or more.

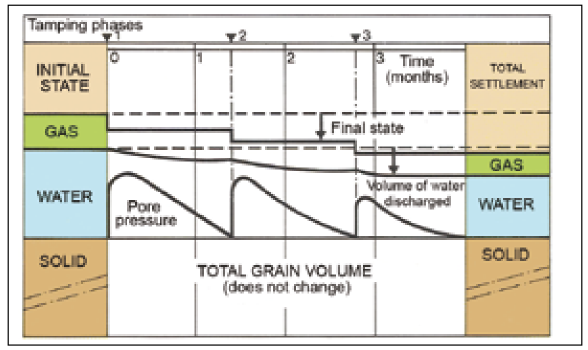
The impact creates various wave trains:

- A relatively fast compression wave **P** (3000m/s), moving in the liquid soil phase and causing an increase in pore pressure, and a dislocation of the granular structure.
- A slower moving shear wave **S** in the solid soil phase.
- A double shear wave, propagating under the soil surface (Rayleigh waves).

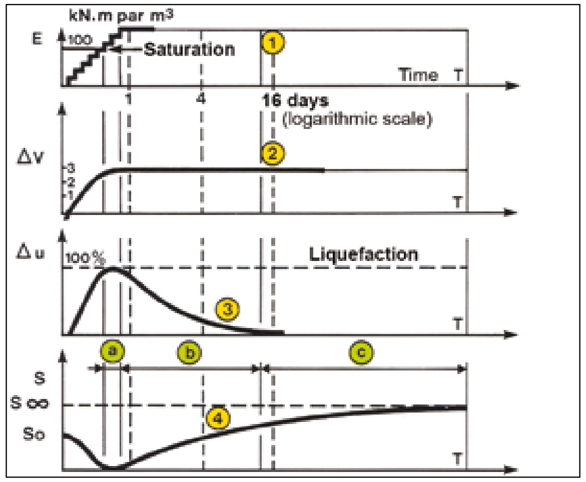
Shear waves have the effect of rearranging the soil particles into a more compact configuration.



Wave propagation under impact

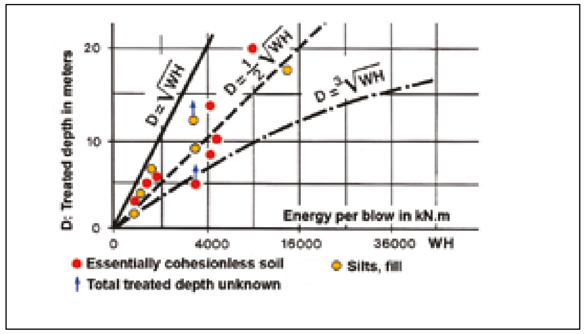


Evolution of gas and liquid phases during a dynamic consolidation operation



1. Energy input in kNm per m^3.
2. Volume change with time.
3. Relationship of the pore pressure u to the liquefaction pressure versus time.
4. Change in load-bearing capacity of the soil versus time:
   a. Liquefaction phase
   b. Phase of pore pressure dissipation
   c. Phase of thixotropy recovery

L. Menard showed the role played by the gaseous phase during the process. The soil can be likened to a stack of «hydropneumatic» capacitors. Under the effect of impact, the gas is compressed into micro-bubbles and subsequently forces out the water. The water escapes through drainage paths in the form of vertical hydrofractures caused by alternating compression-depression in the soil mass. Substantial geysers can sometimes be observed. The almost liquefied soil passes through a period during which its mechanical strength is very low. The effectiveness of the treatment can therefore only be assessed once the excess pore pressures have dissipated.



Relationship between impact energy and depth treated

W kN	H m	D m
150	10	6,1
150	20	8,6
150	25	9,7

< With a standard weight of 15t and common drop heights of 10 to 25m, depths of 6 to 10m can be treated.

2.1. The process of dynamic compaction

The stages of the process are as follows:

- choice of unit impact energy (weight of the mass W and drop height H),
- trials to establish the number of drops, N, per point of impact,
- trials to establish the initial grid M (generally between 0.7 and 1 times the depth to be treated)
- treatment by a first pass according to the determined grid with monitoring of the results obtained,
- treatment by a second pass, with monitoring of results obtained (after possible modification of the parameters W, H, N and M),
- and so on, until the final pass, called the ironing pass or continuous pass.

The allowable bearing capacity following soil improvement depends on the total applied energy and the nature of the compacted soil. The usual upper limits for bearing capacity are:

Silt	Sand
200 kPa	350 à 400 kPa

In general, the soil modulus can be improved by a factor of 2. Absolute settlements are reduced proportionately.

CZECH REPUBLIC - Spolana
First pass

Dynamic compaction rig in operation

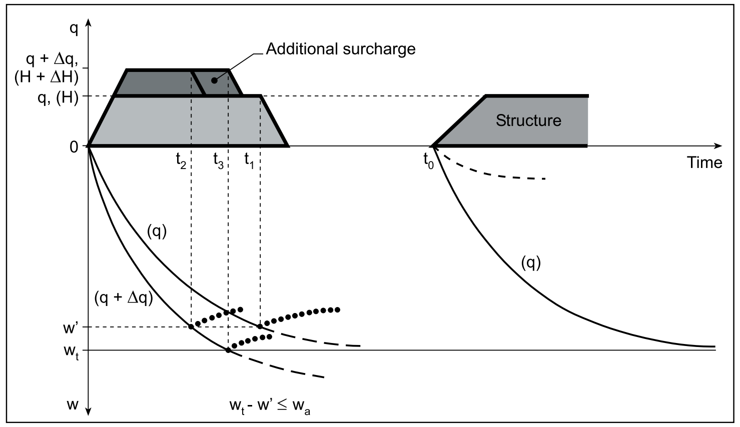
GERMANY - Hailer
Landfill compaction of household waste
25 ton weight dropped 25m with automated crane

3. Vertical drains

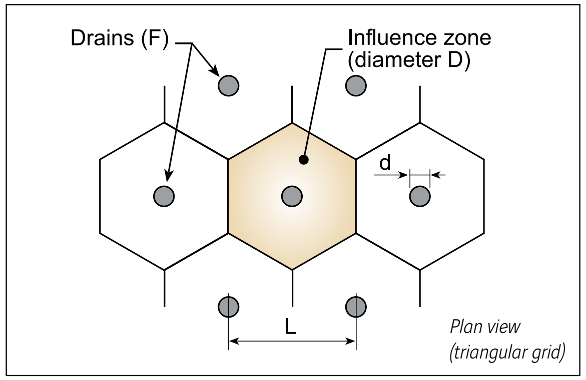
3.1. Theory behind the Method

This method consists of placing vertical permeable elements in compressible soils with low permeability using a close and regular grid layout. The resulting reduction in flow path length accelerates the dissipation of excess pore pressures and hence substantially reduces the time required for consolidation.

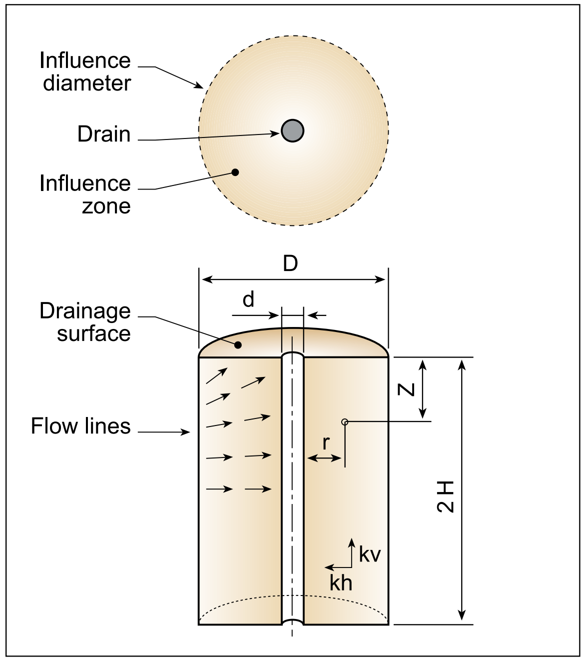
The method is often used in connection with the placement of fill on soft soils. It is usually combined with a preload equal to or greater than that of the future construction load.
The figure below shows that if the expected long term settlement (sometimes over several decades) due to the load «q» exerted by the structure is equal to w_t, then it is possible to achieve a settlement w' (almost equal to w_t) over a much shorter period t_1 (usually between 2 to 6 months), using a grid of vertical drains and a load level q provided by an embankment of height H. At the end of preloading, a slight rebound occurs (dotted line in the figure) and the construction of the structure will result in a settlement equal to the rebound plus the difference between w_t and w'. To restrict delayed settlement to the rebound value, a surcharge, in the form of an additional embankment of height ΔH, should be applied for an additional time period t_3, which is less than t_1. If the consolidation time under the additional surcharge is limited to t_2, delayed settlements of the structure equal to the rebound value plus the difference w_t - w' are to be expected.



The theoretical calculation of the settlement duration for vertically drained soils is based on Terzaghi's and Barron's theories; the hexagonal shaped zones of influence being replaced by equivalent cylindrical zones.



Sketch of vertical sand drain layout



Sketch of zone of influence of a vertical drain >

Design charts based on this theory allow the grid layout to be determined.

3.2. Installation

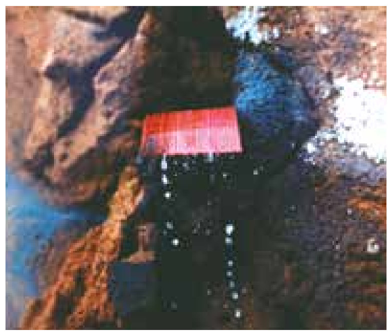
There are two types of vertical drains: sand drains, usually made by drilling in a diameter between 30 and 60cm, and flat prefabricated drains (band drains) with equivalent diameters of to 5 to 6cm, These drains are pushed or vibrated into the ground via a hollow mandrel.

TYPE	DIMENSION width x thickness (mm)	MATERIALS		SECTION
		CORE	FILTER	
DRAINS PLATS				
KJELLMAN DRAIN	100 x 4	Cardboard		
GEODRAIN	100 x 4	Polyethylene LD	Cellulose, cellulose fiber or polyester	
ALIDRAIN	100 x 6	Plastic	Cellulose paper	
COLBOND	var : x 4	Nylon	Nonwoven polyester	
ROPLAST	100 x 3	Celluloid	Felt	
MEBRA-DRAIN	100 x 3	Polypropylene	Typar	
P.V.C.	100 x 1^{5}	Microporous PVC		
DRAIN PIPES				
SOIL DRAIN	Ø 50 to 200 mm	Polyester	Felt	

Installation of prefabricated band drains


Mandrel driving leader with top vibrator

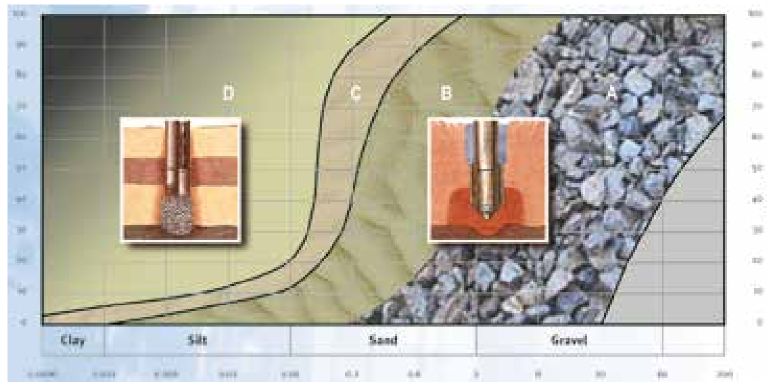

Placing of anchorage shoe for a band drain


Band drain after installation

4. Vibroflotation (vibro-compaction and stone columns)

4.1. Principles - Scope of application

The Vibroflotation method involves the use of vibration at depth to improve the soil (vibro-compaction) and/or to install columns with enhanced mechanical properties (stone columns).
The field of application of each technique is directly related to the particle-size distribution of the soil to be improved:



The soils of zones A and B are granular with a percentage of fines (≤ 0.06mm) of less than 12%. They can easily be compacted by vibration to relatively high densities.

To the right of zone A, the soil may be too coarse for the vibrator to reach the required depth. Pre-drilling or the use of powerful vibrators may be necessary.
In zone D (more than 20% fines), permeability is too low for the compaction to work. Vibro-compaction would not therefore be effective and stone columns are needed.
In the intermediate zone C, the soil is too impermeable for vibro-compaction to be fully effective, but the installation of stone columns in silty sand will allow water to escape through the neighboring columns already in place and thus improve compaction.

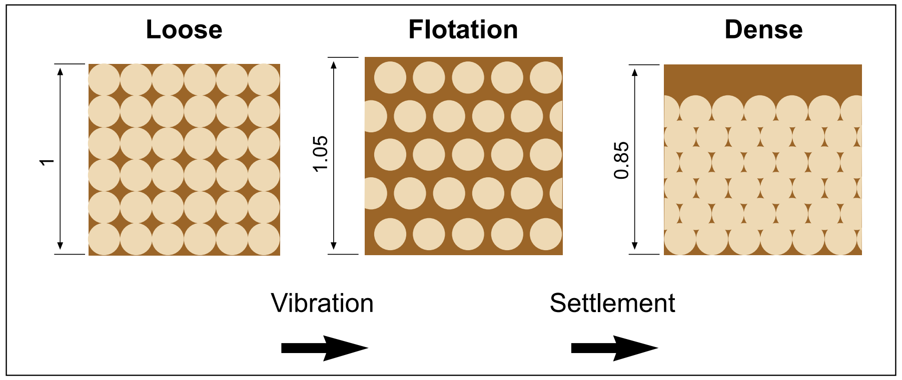

Vibro-compaction


Stone column

4.2. Vibro-compaction

4.2.1. Principle

A vibrator is inserted vertically into the soil following a regularly spaced grid. The vibrations passing through the soil provide a transient state of liquefaction, allowing a rearrangement of the soil particles into a denser configuration:



4.2.2. Effects and Design

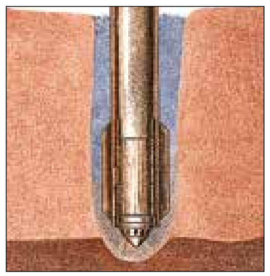
Vibro-compaction results in soil settlement of about 4 to 8%, sometimes even more; the effects on the soil are as follows:

- Decrease in void ratio,
- Increase in density,
- Increase in the cœfficient of earth pressure at rest (K_0),
- Decrease in permeability (usually in the ratio of 2 to 5),
- Increase in the angle of internal friction of 5 to 10 degrees,
- Increase in the modulus of deformation by a ratio of approximately 2 to 4.

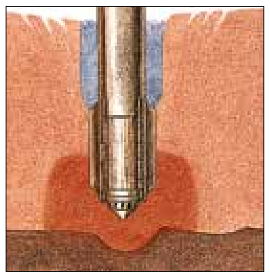
The design is developed around the above principles and consists of determining the relative density required to obtain the specified geotechnical characteristics.

4.2.3. Implementation

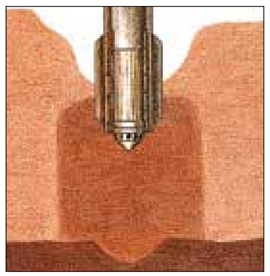
Vibro-compaction is generally performed using a triangular grid layout. The distance between grid points varies from 2.5m to 5.5m depending on the type of soil and its initial density, the result to be obtained, the type of vibrator used (power, amplitude of vibration, eccentric force) and the detailed compaction procedure (height of successive passes, criteria for completion of each pass such as amperage or hydraulic pressure).



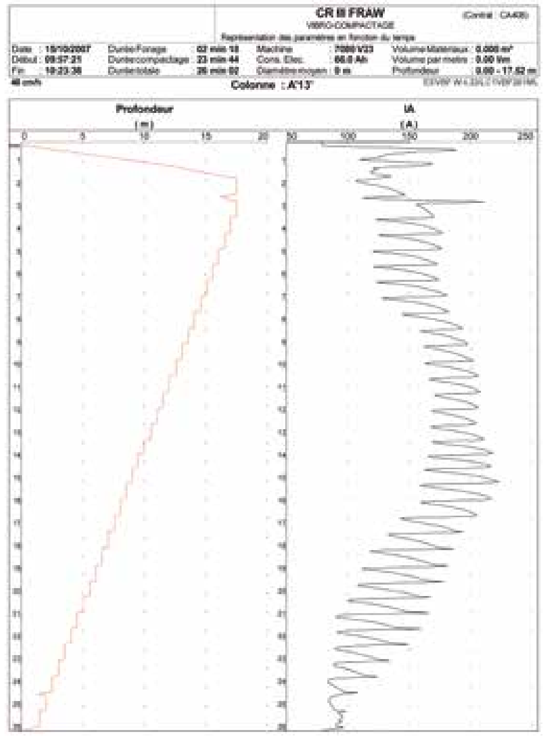
PENETRATION
The vibrator penetrates the soil to the desired depth with the assitance of vibration and water or air jetting.



COMPACTION
The vibrator is raised in 50cm stages. The existing sand or gravel drops towards the tip of the vibrator.



END OF PROCESS
The process is completed by placing back fill at the surface or by simply allowing the existing ground level to drop

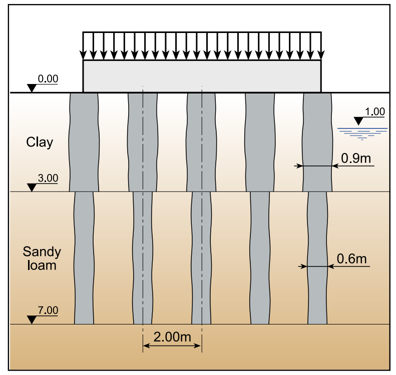


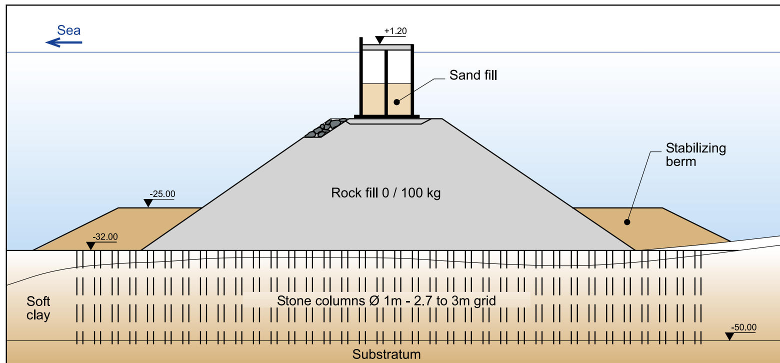
Logging of installation (electrical vibrator)

4.3. Stone columns

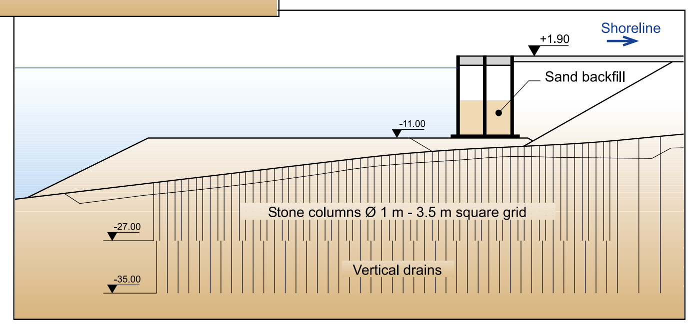
4.3.1. Principle

Stone columns installed using the same equipment as for vibroflotation, allow silts and clayey soils to be drained and reinforced. They can be installed on land (right hand figure) or over water (figures below).





GREECE - Patras Port Expansion - Phase II
Foundation treatment by stone columns



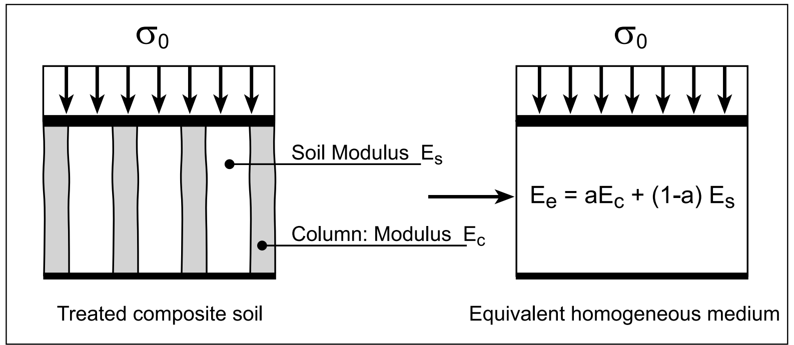
4.3.2. Effects and Design

Stone columns have the following effects on the treated soil:
- introduction of porous elements with good engineering properties on a regular grid,
- increase in the modulus of deformation of the whole treated mass,
- increase in the average angle of internal friction and in overall shear strength,
- increase in the cœfficient of earth pressure at rest (K_0),
- significant increase in the rate of consolidation, with most of the settlement occurring during the first weeks after construction.

There are many design methods for stone column reinforcements. Most of them are only valid for loads spread over a large area. Methods confirmed by experience are preferable.

4.3.3. Homogenisation, or smear method

The modulus for the equivalent homogeneous medium is calculated on the basis of the percentage of incorporation «a» (ratio of column section to the treated soil surface) and the ratio of column modulus to undisturbed soil (often taken as equal to 8 or 10).



Note: The calculation methods advocated by the «Recommendations on the use of stone columns» published by COPREC and SOFFONS are based on this method.

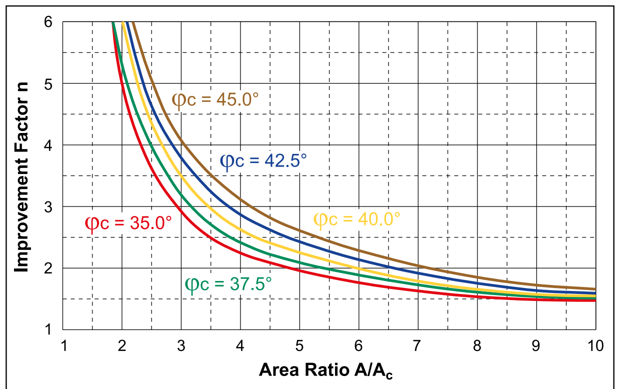
(Source: A. Dhouib and F. Blondeau, «Colonnes ballastées [Stone columns]», ENPC publications, figure 3.4 page 136)

4.3.4. Priebe's method

Priebe's method is used for determining a settlement reduction factor or improvement factor «n», the ratio of settlement in the unimproved soil to that of the improved soil. This is dependent on the angle of internal friction of the column material (ballast) and the A/A_c ratio («area ratio»), which is simply the inverse of the percentage of incorporation: $a = A_c/A$.

When the compressibility of the material forming the column is taken into account, this leads to an improvement factor n_1 which is lower than n, whilst when the confinement provided by depth is taken into account this leads to an improvement factor of n_2 which is higher than n.

Priebe's method is also used for the empirical calculation of strip footings or individual footings on stone columns.



The friction angle of the homogenized mass is finally calculated as follows:

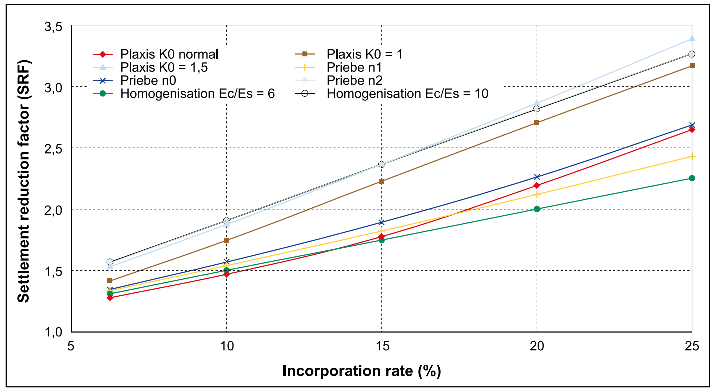
$\operatorname{tg}\phi_e = m \operatorname{tg}\phi_c + (1\text{-}m) \operatorname{tg}\phi_s$, where :

$m = (n_1\text{-}1)/n_1$

ϕ_e = angle of internal friction of the equivalent homogeneous medium

ϕ_c = angle of internal friction of the column material

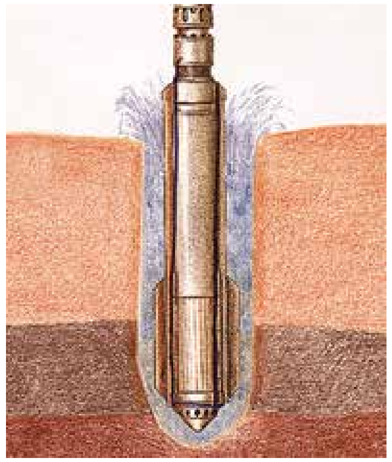
ϕ_s = angle of internal friction of the soil

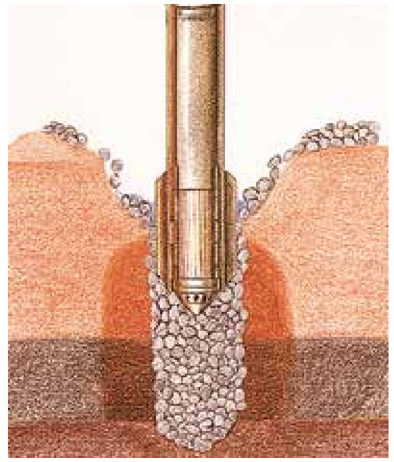


Comparison of the improvement factors versus the percentage of incorporation by different methods (homogenization, Priebe, finite elements) and different assumptions

4.3.5. Installation

The wet top-feed process

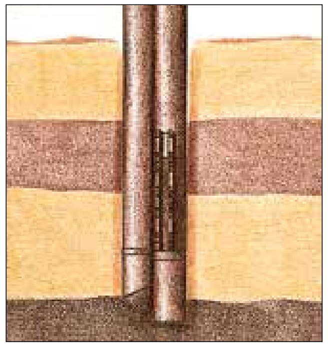




The vibrator is introduced into the soil to the required depth whilst water jetting creates an annular space around it.

The ballast is placed from the surface down the annular space and compacted by the vibrator in upward stages resulting in forced lateral displacement of the surrounding ground.

The dry bottom-feed process

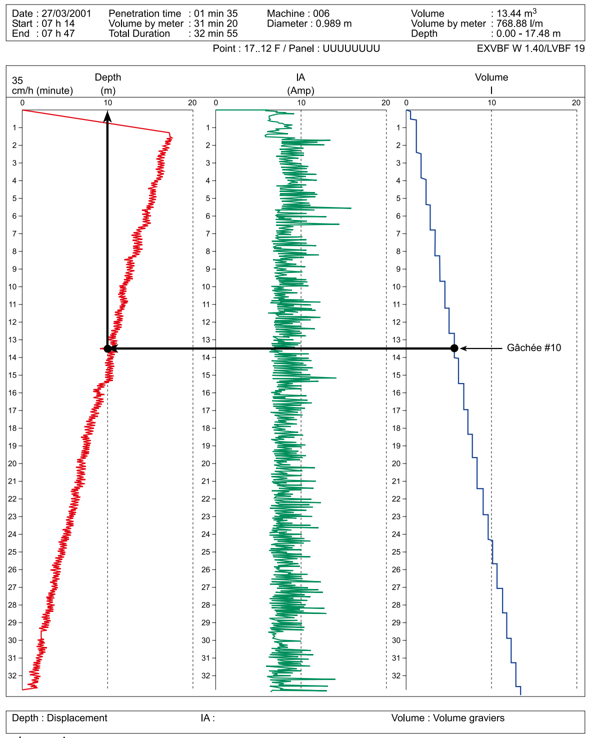


The vibrator penetrates to the desired depth under the action of vibration and air jetting.

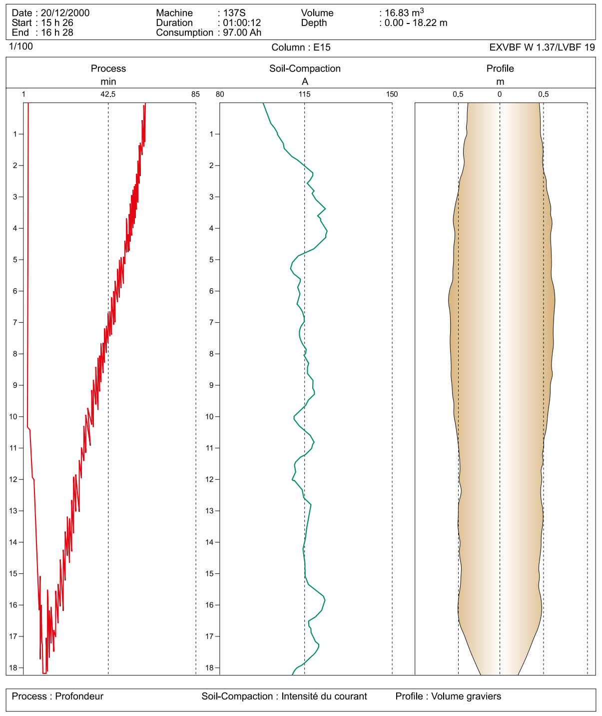
The column is installed by placing the ballast via a lateral tube alongside the vibrator.

The diameter of the columns varies with soil strength. The treatment is completed by leveling and compacting the ground surface.

Logging of parameters



Log v time



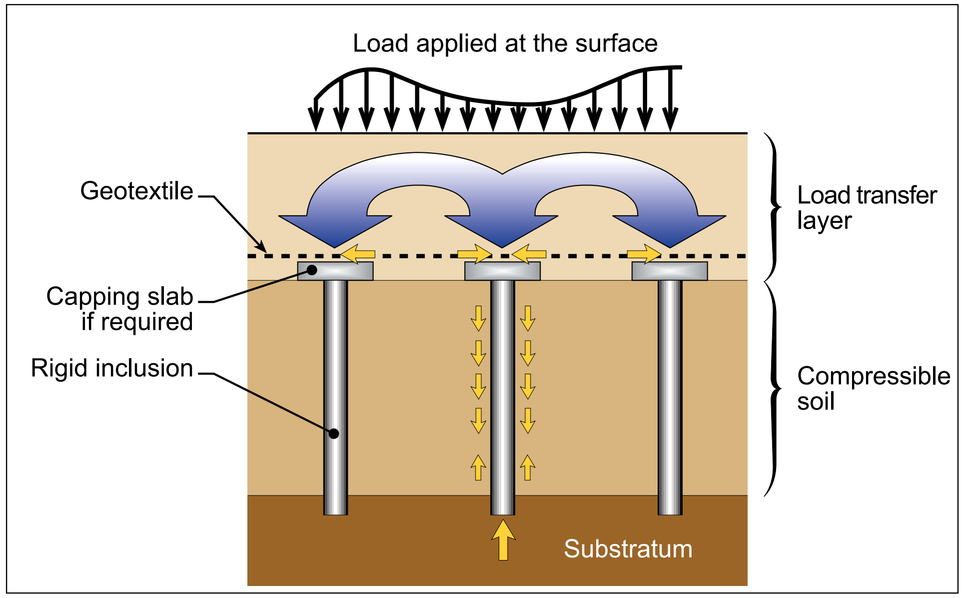
Log v depth

STITCHER stone column rig with COBALT recording system

5. Rigid inclusions

5.1. Principle of the method

Reinforcement by rigid inclusions combines a grid of vertical inclusions, extending down to a load bearing stratum, with a backfill layer of frictional soil forming the load distribution platform. The purpose of this system is to transfer vertical loading applied at the surface to the load bearing stratum without causing unwanted settlement in the compressible layer. The inclusions comprise structural elements possessing their own strength and low deformability compared with the compressible soil through which they pass. Inclusions may be topped by a slab or an enlarged section at the head if required. Layers of horizontal geotextile or wire mesh can be used to reinforced the load distribution blanket. This blanket consists of a granular material (alluvium gravel or quarry-run) or soil treated with hydraulic binders. Structural loads are supported by this blanket via shallow strip foundations, isolated footings or on mat foundations depending on the structure. Base slabs or ground slabs are supported on grade and also act as the load distribution blanket.



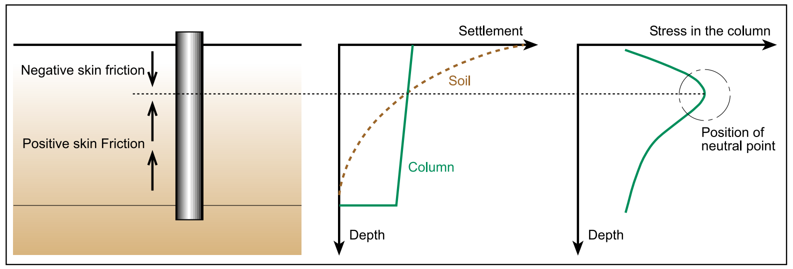
Principle of load transfer

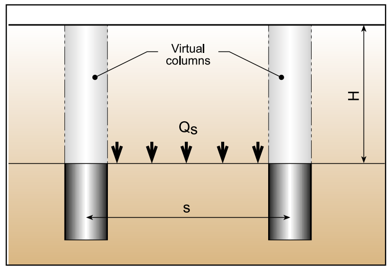
5.2. Design principles

There are many analytical approaches which differ in the load transfer model adopted. The most popular method in France is the one developed by O. Combarieu and is based on load transfer in the load distribution blanket and in the soil by negative friction. The method is based on two assumptions:

1 - An arching effect develops in the distribution blanket as soon as the compressible soil settles more than the inclusions. There is hence a virtual extension of the inclusions within the height of the blanket. These virtual inclusions are also subjected to negative skin friction.

2 - The compressible soil subjected to overburden stress will also load the inclusions by negative friction, increasing the total load transfer to them.





Negative friction method by O. Combarieu

Other methods develop the arching effect only within the blanket and the distribution of stresses between inclusions at the base of the blanket (Terzaghi's method (1943), Marston and Anderson's method (1913), etc.).

Today, numerical methods are considered to be the most reliable way of studying the behavior and interaction of the soil/inclusion/platform/structure system.

The design validation approach is based on the following steps:

1 - Calculation of settlements.
2 - Verification of the maximum stresses in the inclusions (neutral point).
3 - Choice of the strength of the constituent material.
4 - Check of the load bearing capacity of the inclusions.
5 - Check of resistance to horizontal forces.
6 - Check against punching shear in the load distribution platform.

5.3. Installation

The rigid inclusion is installed by drilling or driving.
The following types of device are examples of those that can be used:

- soil mixing tools (see dedicated chapter),
- displacement auger, hollow stem auger (HSA),
- vibratory driven tube with capped end.

The diameter of the tool varies according to the objectives of the treatment. When the tool has reached its final depth, concrete or grout is pumped in.

5.4. Examples of application

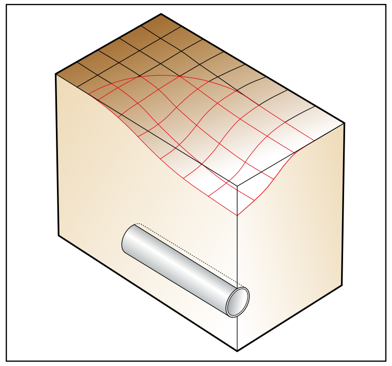
On the left, the rigid inclusion rig and on the right, the stone column rig

Installation of rigid inclusions with vibratory driven pipe

COMPENSATION GROUTING

1. Principle

Tunnel driving inevitably causes a shallow bell shaped settlement that is a potential source of substantial damage to existing structures.
Compensation grouting is an active technique used to counteract relaxation of the ground resulting from tunnel excavation. This is achieved by injecting precise quantities of grout between the tunnel roof and the structures exposed to the danger.



The key factors for success are as follows:
- precise measurement of the quantities of grout injected,
- perfect synchronisation of the grouting exercise and the advancement of the excavation: the grouting should precede the anticipated settlement,
- precise, real-time monitoring of the grouting process and displacements of the soil and structures.

2. Applications

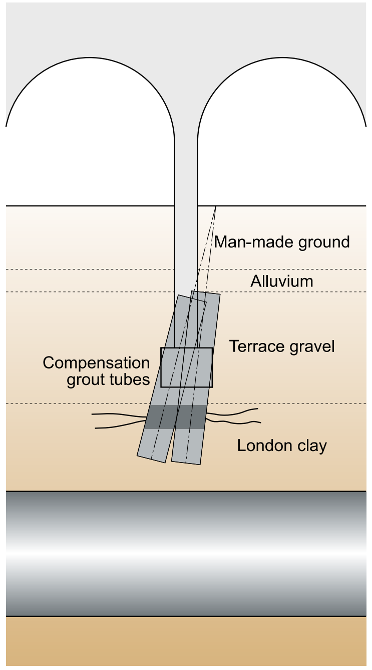
Compensation grouting is a method which can be used whenever the construction of a tunnel is likely to cause movement of a sensitive structure, irrespective of the tunneling method. (tunnel boring machine, NATM, etc.).
The method is subject to certain geotechnical constraints:
- there must exist above the tunnel roof a suitable stratum in which the compensation grouting can be carried out,
- the method cannot be used in soft clays since the compensation effect will not be sustained over time,
- structures supported on piles are, generally speaking, far more difficult to protect with compensation grouting.

The technique is a perfect fit with the observational method: compensation grouting can comprise the mainstay of the contingency plan to protect adjoining structures during tunnel construction.

3. Techniques used

Compensation grouting can be carried out either via subvertical drilling (as in the Jubilee line) or subhorizontal drilling (as in Station Rio Piedras).
The first stage of grouting is carried out before tunneling work begins, to tighten the ground around the drill hole.

Compensation grouting must be very accurately monitored, as only small quantities of grout are used, with low grout pressure and flow rate. SPICE, Soletanche Bachy's in-house control system, is ideally suited to this technique. Measuring instruments must be set up all around the site to ensure comprehensive data is provided on displacement of the ground and the structures that need protecting. The GEOSCOPE automatic surveillance system, developed by Sixense Soldata, can be linked to SPICE in order to allow the data to be accessed from the grouting plant.



A purpose-developped module of SPICE can be used for prediction of settlement as a function of tunnel advancement. The module uses a simplified model for calculating settlements, based on calculations by finite elements, continually adjusted on the basis of observations made during the tunneling.

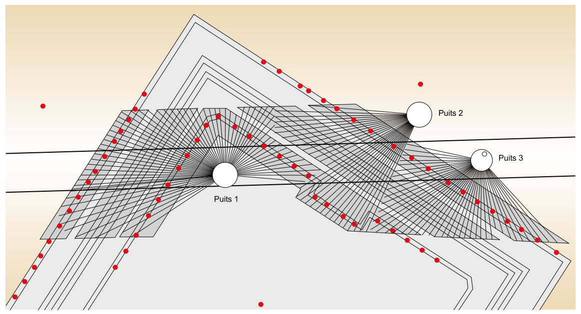
After tunneling is completed, a final grout phase is performed to deal with any post-tunneling settlement, particularly in clayey soils.

4. References

- London - Jubilee Line Extension: contracts 101 (Green Park), 103 (Southwark Station) et 105 (Bermondsey Station)
- Madrid - Vallecas Line 1
- Porto Rico - Station Rio Piedras
- Madrid Metrosur - Tramos V-VI Fuenlabrada-Getafe
- Moscow - Lefortovo Tunnel
- Edmonton - LRT Extension
- Richmond - Virginia Capitol
- London - King's Cross station

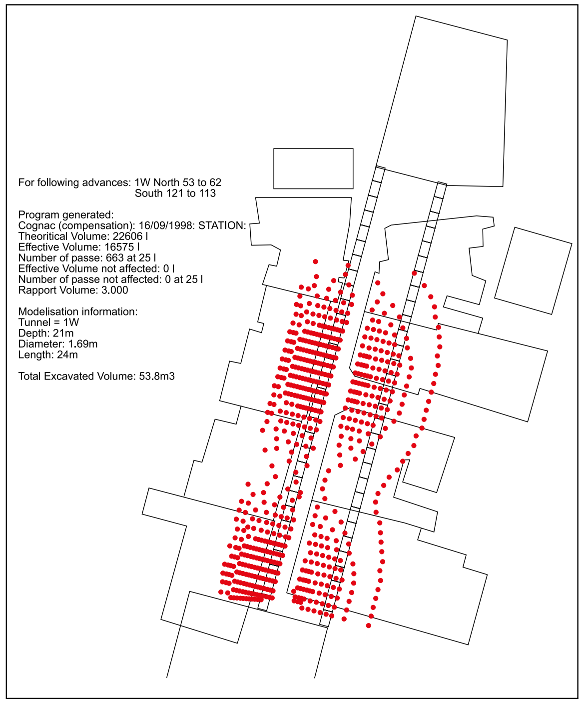
RUSSIA - Moscow - Lefortovo
Venetian shaft and drilling for compensation grouting

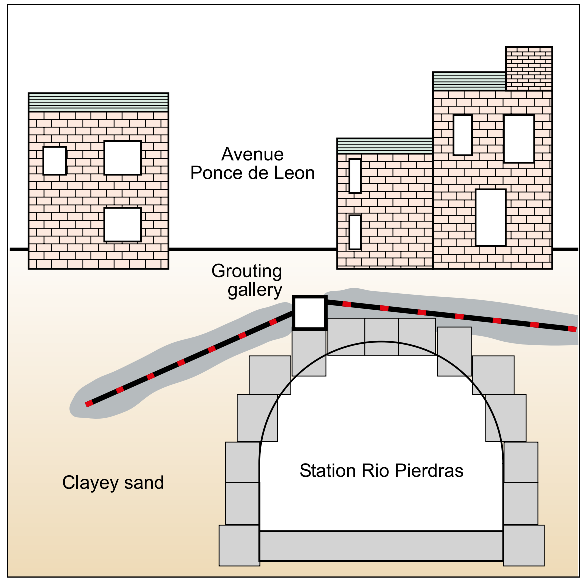
Monitoring the structure using the CYCLOPS® system



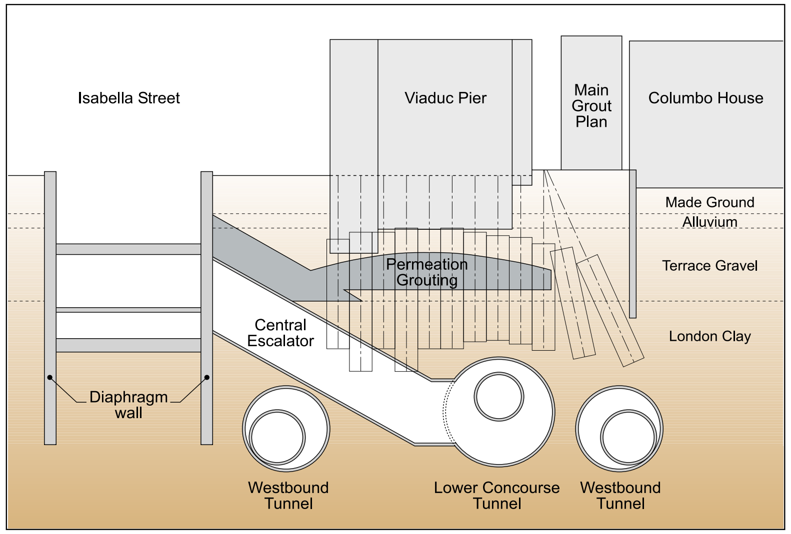
Compensation grouting from Venetian shafts

The military school





San Juan de Porto Rico, Station Rio Piedras: the compensation grouting program based on tunnel advancement.



Control room for compensation grouting

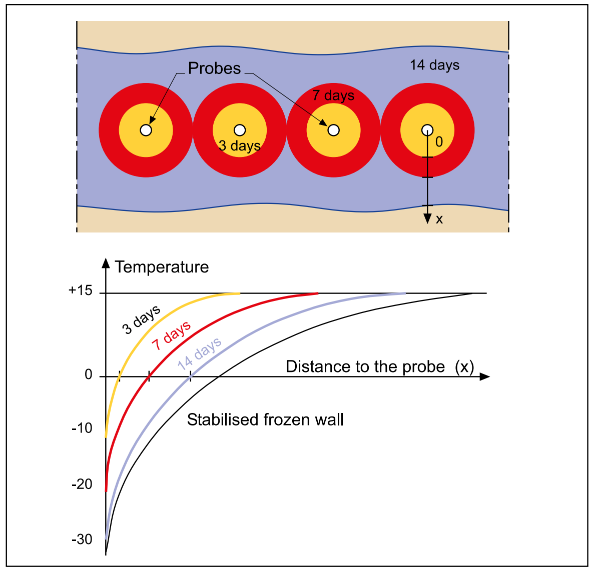
LONDON - Jubilee Line Extension ,
Southwark Station (Contract 103)

GROUND FREEZING

1. Principle

Ground freezing works on the principle of freezing the water in the soil pores, thus rendering the soil impermeable and of greater strength.

The soil is frozen by the transfer of calories to a lowtemperature fluid from the ground through a probe. When in contact with the probe, the water freezes and forms a sheath of frozen soil around the probe. The sheath gradually expands and can be used to build strong, impermeable barriers.



Development of the freeze wall over time : 3 days (yellow), 7 days (red), 14 days (blue). >

2. Applications

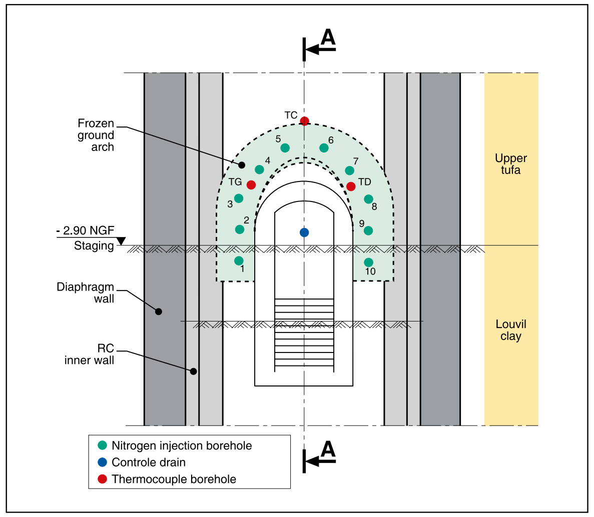
The main distinguishing features of the freezing process as compared to other ground support or treatment techniques are as follows:

- Freezing is a temporary process and there is no long term permanent change either in the subsoil or in the natural hydrology.
- The technique can be applied to ground below the water table or with sufficient water content. For work above the water table, additional water can be added for certain applications.
- The process renders the soil completely watertight and therefore there are no issues of pumping or treating the water or of external drawdown.

Pre-grouting may be needed where the ground to be treated includes areas subject to groundwater movement or is of a very open nature.

The most common uses of ground freezing are for mine shafts, cross passages between tunnels, safety niches and excavations beneath sensitive structures.

FRANCE - Lille Metro - Line 2
Tunnel excavation protected by the frozen ground arch installed as tunnelling proceeds

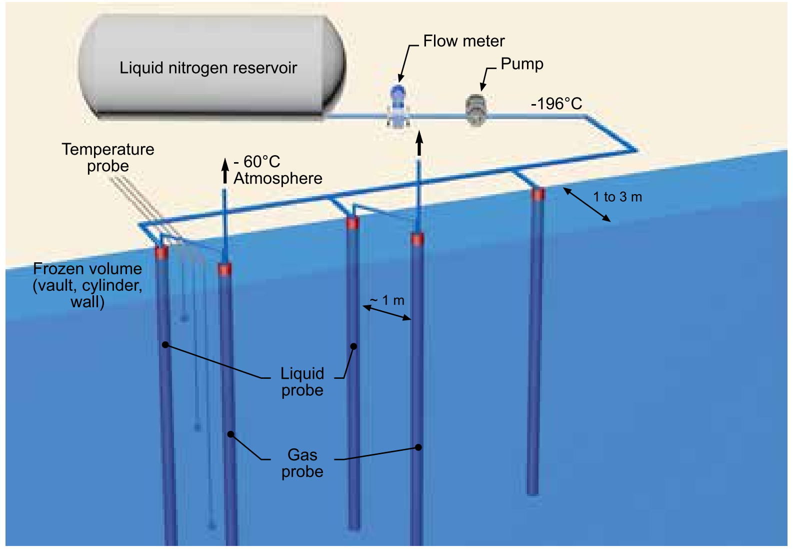


Borehole geometry > View from the fireman's access shaft

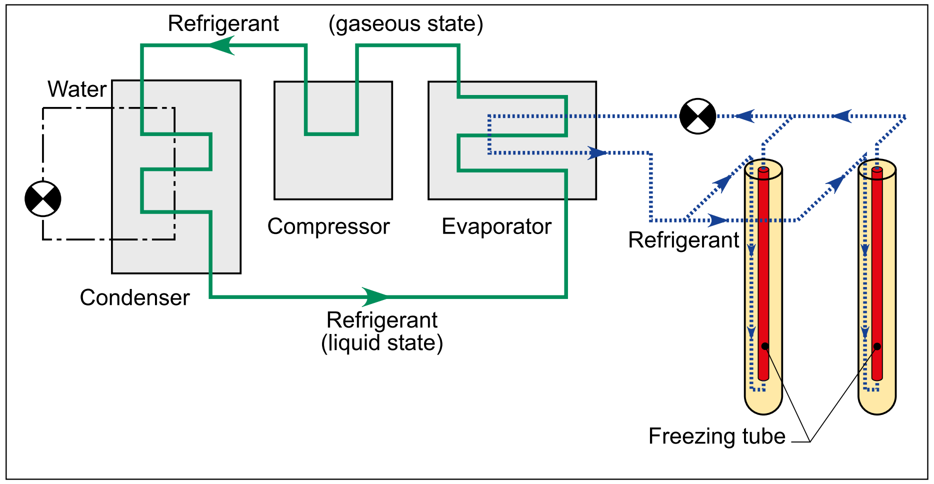
3. Techniques employed

There are two main methods used in ground freezing:

The direct expansion technique, in an open circuit
Liquid nitrogen is used as the refrigerating medium. The refrigerating effect comes partly from the latent heat absorbed when the liquid nitrogen boils and partly from the gas heating (from -196°C to about -80°C for nitrogen).
The gaseous nitrogen is discharged into the atmosphere.



The double exchange method, in a closed-circuit
(with a refrigeration plant)
The soil is frozen using a low temperature fluid (brine at a temperature of -25°C, -35°C). The brine itself is chilled by an evaporator through a vapor compression cycle refrigeration system using a refrigerant (ammonia, hydrofluorocarbons).
The brine flows through a manifold system, taking heat from the soil through the freeze pipes before returning to the refrigeration plant for re-chilling.



Closed circuit

Initial freezing of the ground requires several days when liquid nitrogen is used, or for several weeks in the case of brine. Where the soil needs to be kept frozen for a long period, it is generally more economical to use brine, where electricity is the main source of cost, rather than liquid nitrogen supply.

Nitrogen is an inert gas (it makes up 80% of the atmosphere), but is heavier than air. Special precautions need to be taken to prevent nitrogen buildup in confined spaces, as the lack of oxygen can cause suffocation.

Soil strength increases as the temperature goes down, and varies from 2MPa for silt, to 10MPa for sand, at a temperature of -10°C.

Frozen ground is however, like ice, liable to creep under load.

The freezing process can be monitored by measuring the temperatures of the ground mass in question through boreholes fitted with temperature probes. In the case of a treatment forming a closed box, there is an increase in pore pressure within the box caused by the increase in volume when the water changes into ice; an internal piezometer provides an excellent check on closure of the box.

In soils with low permeability, the increase in volume resulting from water changing into ice and the effect of cryogenic suction (water moving towards the frozen area) can cause deformations which must be taken into account at the design stage. Another factor to be considered is the possible loss of strength as the soil thaws out (one of the reasons for heavy vehicle restrictions during thaws).

4. Setting up a ground-freezing operation

A ground-freezing project is designed by reference to two distinct functions:
- A thermal design to determine the evolution of temperature over time, factoring in the borehole spacing (in general 1 to 2m), and boundary conditions (the possible presence of any heat sources, etc). Dedicated software is used to provide a detailed prediction, which allows any unforeseen anomalies to be detected during the works phase.
- A structural design which factors in the specific conditions of the operation: ground swelling, creep, final thaw.

For both phases, laboratory tests are carried out on intact samples to determine the thermal characteristics of the soil at different temperatures, the swelling pressures, the instantaneous and time-dependent strengths.
It is also necessary to assess any potential movement of the water-table and the quality and salinity of the ground water.
The project must be designed by an engineer specialized in the subject.

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