BUILDING A COMPLEX SHORING AND A DOUBLE TRENCHLESS CHANNEL WITH THE HELP OF JET-GROUTING

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ABSTRACT

In order to comply with new, more stringent regulations regarding cooling and ventilation of its world headquarters in Vevey, Switzerland, Nestle Inc. proceeded with construction of a 6m deep, 20mx20m chamber designed to house heat exchange units and pumps. The site selected for construction is adjacent to the edge of the main headquarters building. The proposed structure was to be built below lake water level, in an area formerly occupied by a marina. This site was filled in, around the year 1930, with random fills, including demolition wastes and other residues. The natural soils, found at depths ranging from 3m to 4.5m, are silty sands and silts with gravelly inclusions.

Construction constraints included a high water table, the need to limit noise and vibrations, and the possible presence of obstacles to drilling in the random fill. A construction technique based on the installation of jetted (Jet-Grouting) inclusions was selected for shoring. Uplift forces on the chamber required the installation of vertical tendons to mitigate buoyancy; steel rod reinforced jetted elements were also used for this purpose.

The chamber was to be connected to the lake bottom by two parallel, 500mm external diameter pipes. These pipes were to leave the chamber and slope at an angle of 12° from the horizontal towards the lake bottom, crossing very soft soil strata just beneath a rocky dike present at the lake shore. In order to allow for construction of these two pipes with minimum disruption of the site and the lake bottom environment, a trenchless excavation was performed: the soils were first reinforced by an array of parallel jetted elements sloping towards the lake, then reinforced materials were bored in three phases. The first phase was performed with a 200mm diameter tool, which was pushed towards the lake. Specially designed 450mm and 600mm tools were then pulled from the lake towards the chamber, in order to create two parallel cavities with the necessary constructional tolerance to allow for insertion of the 500mm diameter pipes, which were then grouted into place.

This paper describes the site, the project, and the relative merits of the application of the Jet-Grouting technique in building soil-grout structural elements for shorings and foundations undergoing both tensile and compressive forces. Results of loading tests of such elements are discussed. The use of jetted inclusions for soil reinforcements, as applied to trenchless excavations, is also described.

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A BRIEF HISTORY OF JET-GROUTING

Jet-Grouting was first introduced in Japan in the mid-sixties in order to build grout-soil mixed inclusions to reinforce weak soils. The technology was adopted and further developed in Europe and South America, where both equipment suppliers and special work contractors recognized applications for this innovative procedure. Indeed, the enhanced treatability of sandy and silty soils, the low use of steel reinforcement, and the lack of requirement for expensive aggregates quickly made Jet-Grouting a weapon of choice in the arsenal of specialty contractors. Jetting has been used extensively in situations requiring low noise-vibrations levels and small-sized machines.

Jet-Grouting Technological Review

Jet-Grouting technologies allow in-situ mixing of soil and grout by means of high pressure injection of one or more fluids. In Japan, Jet-Grouting was developed to overcome difficulties inherent in the application of classic grouting procedures to fine sands and silts. In 1970, Nakanishi et al. developed the "CCP-method," in which a grout jet is spouted from a nozzle located at the end of a drilling rod, which rotates and elevates, producing a pseudo-cylindrical solidified body in the ground (Miki, 1973). The drill bit also allows horizontal and vertical high speed water jets to excavate the soils during the drilling phase, which is performed immediately before jetting (ASCE, 1980).

The Jet-Grouting process is composed of three distinct phases:

- The soil destructuration under the hydrodynamic effect of the high kinetic energy (high velocity) jet of either grout, or other fluids.
- The extraction of a portion of the soils through the annular space around the drill rods.
- The in-situ mixing of a cement grout with the residual materials under the effect of the grout jet.

For purposes of consistency, the CCP-method shall be referred to hereafter as Jet-Grouting I, where the capital roman numeral indicates the number of fluids jetted into the ground.

In 1973, Jet-Grouting was recognized by an international panel of experts as a potential method for stabilization of the leaning tower of Pisa, in addition to two or three other proposals, including one based on Root Piles. In the mid-eighties, developments in jetting pressures, nozzles, and other technological components allowed construction of 500mm diameter jetted inclusions in sandy soils, and 300mm diameter jetted inclusions in clayly soils (Miki & Nakanishi, 1984). Papers concerning applications of Jet-Grouting I began to be published and presented at international conferences (Shibazaki & Otha, 1982; Aschieri, Jamiolkowski & Tornaghi, 1983), and by 1984, hundreds of projects incorporating Jet-Grouting had been completed in central Europe (Baumann & Dupeuble, 1984). Among these projects, numerous cases involved difficult underpinning or reinforcement of historically significant buildings. In these situations, Jet-Grouting was found to be a very competitive and sound solution: reinforcement depths of 25m-30m were attained in Germany, and depths of over 40m were claimed by Japanese contractors. Very early on, it was recognized that precision of drilling was the limiting factor in very deep jetting applications, especially if a low global permeability of the grouted volume was sought.

Types of Jet-Grouting techniques

All Jet-Grouting is performed while extracting the drilling rods from the bottom of the inclusion up towards the surface. The drilling rod is generally equipped with a destructive tool such as a tricone, allowing it to bore through hard strata, rocks, or other obstacles. The drilling machine, the high pressure fluid duct, the mixing plant, and the material silos constitute a typical Jet-Grouting installation.

▪**Jet-Grouting I**: A decade ago, standard Jet-Grouting practice demanded grout pressures of 25MPa to 40MPa, nozzle diameters of 3.5mm to 4.5mm, rotations of 6R.P.M to 10R.P.M, and uplift velocities of 600mm/min to 800mm/min (Baumann & Dupeuble, 1984) . Tests, observations, and construction site experience gained in the last eight years, on over one hundred applications of Jet-Grouting I in various geotechnical and typological settings, have brought the Author to today's current practice, which demands that Jet-Grouting I be applied at a pressure of 45-55MPa, while the rod is rotating at 15R.P.M. to 25R.P.M., and is lifted at a velocity of 100mm/min to 500mm/min. The diameter of the jetting nozzles (generally two in number, diametrically opposite to each other, and located on the rods just above the drilling tool), varies between 1.5mm and 2.5mm. Terrain characteristics and the type of application required dictate the final choice of the parameters for obtaining Jet-Grouting I inclusions with a 600mm standard nominal diameter (nominal diameter is defined as the smallest diameter encountered in any cross section of the inclusion); fine tuning is achieved by building test columns before mass production begins. The compressive resistance of the inclusion depends on the soil and the grout characteristics. Table 1 gives ranges of compressive strength of the soil / grout mix as a function of soil types and water/cement ratio of the grout.

▪**Jet-Grouting II and III**: Although the focus of this paper is a Jet-Grouting I application (monofluid high pressure grouting), a brief examination of two other jetting procedures, Jet-Grouting II (bi-fluid: air and grout) and Jet-Grouting III (tri-fluid: air, water and grout), is of interest for purposes of comparison. In order to enable the construction of significantly wider Jet-Grouted bodies, with diameters ranging from 1000mm up to several meters, Jet-

Grouting II and Jet-Grouting III were developed at the end of the seventies, also by Japanese applied researchers (Miki & Nakanishi, 1984). Jet-Grouting II uses air to enhance the cutting capabilities of jetting, and Jet-Grouting III adds to air the effect of water. In Jet-Grouting II, compressed air surrounds the jet of cement grout, thus minimizing the rate of rebound (return flow). In Jet-Grouting III, a triple rod system allows the use of a special two-stage jetting tool: at the top stage, one or two double nozzles allow water jets to emerge, which are protected by a crown of compressed air. A set of two nozzles, located 300 to 500mm below, delivers the grout. Generally, the triple rod (90mm diameter) is lowered into a predrilled borehole of 150mm diameter. Rotation is performed at 5R.P.M. to 10R.P.M., and lifting is conducted at velocities of 50mm/min to 300mm/min. One advantage of some of these alternate systems is that continuous monitoring of the diameter of the inclusion is possible. Another advantage is that by increasing the diameter of the inclusion, it is possible to reduce the number of inclusions and therefore speed-up construction. Because of the cost of equipment mobilization/de-mobilization, however, and the highly sophisticated equipment necessary to properly conduct and monitor a Jet-Grouting II or III construction site, these procedures are seldom used in the Author's practice.

Table 1 Compressive strength [MPa] of soil/grout mix produced by Jet-Grouting I in various soil types, with varying water/cement grout ratios.

Cement Grout	Clayly Silts	Silty Sands	Sandy Gravels
Water/Cement=0.7	6-10	10-14	12-18
Water/Cement=1.0	3-5	5-7	6-10

APPLICATIONS OF JET-GROUTING I

Columnar Elements (Compression)

Jet-Grouted inclusions can be used as columnar elements to solve foundation problems in weak terrains. The grouted core generally has compressive resistance in the ranges defined in Table 1. In clay-based silts, the Author's current practice leads to the application of a global strength reduction factor of at least 2, thus leading to a nominal structural allowable compressive capacity of 400kN for a Jet-Grouted I core of 600mm nominal diameter.

The structural capacity of the inclusion can be enhanced by introducing a reinforcement into the fresh column, such as a steel tube or a H beam (Figure 1). Generally, it is more effective to keep the diameter of the reinforcement small (less than 200mm) relative to the diameter of the jetted inclusion, in order to avoid excessive difficulties during insertion. In extreme cases, where particularly difficult obstacles, rock strata, or boulders are present, it may be prudent to plan for the redrilling of the columns 24 hours after initial jetting, in order to insert the reinforcement. In these cases, a gravity driven grouting of the cavity has to be performed to seal the reinforcement in the inclusion. The geotechnical capacity of the jetted elements, reinforced or not, is generally evaluated using standard deterministic pile analysis methods, or probabilistic evaluation of pile behaviour (Oboni, 1988, Oboni, 1989a; Oboni 1989b). Very often, a global treatment of the terrain by an array of relatively short inclusions offers an optimum solution. Economical foundations with creeping jetted columns have been built, on many occasions, on weak glaciolacustrine soils in western Switzerland (Oboni & Hlobil, 1991; Oboni, 1992).

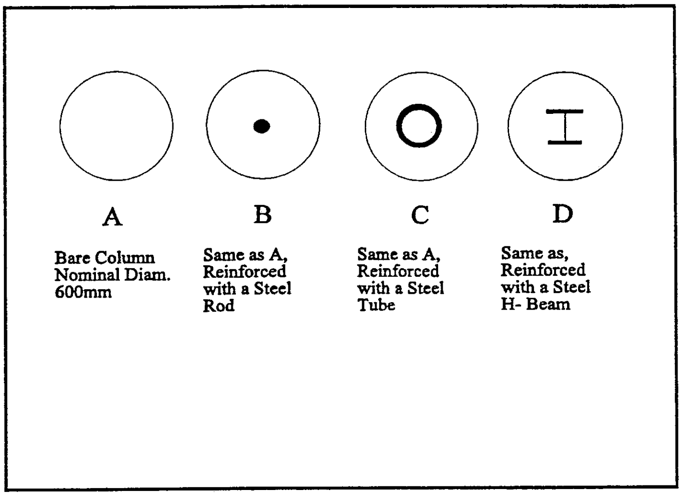

Figure 1 Typical jetted columns cross sections.

The determination of parameters to be considered in a jetted column geotechnical bearing capacity evaluation is a less documented process than the determination of the parameters for bored shafts, driven piles, and injected inclusions such as micropiles and/or injected tendons. The Author and co-workers are currently preparing a publication concerning this interesting and original subject.

Column/Beam (Compression and Bending)

▪**Underpinning**: The diameter of the drilling rods (D_{dr}= 63mm to 90mm) and the diameter of the drilling tool (D_{dt}=75mm to 140mm) can be varied to accommodate specific requirements. For example, when underpinning existing structures, it is necessary to limit pressure build-up in the column which could result in damaging heave of the existing foundation. This limitation is generally, and quite inexpensively, achieved by using a system where $D_{dt}>D_{dr}$, thus maximizing the surface of the annular space around the drill rods. Underpinning elements can be reinforced in the same manner as columnar elements. In some cases, the heads of the reinforcements can be designed so that jacking can be performed between the head and the existing structure to pre-load the elements and control deformations. Construction site experience and tests have shown that Jet-Grouting can be applied effectively to underpinning jobs performed in the open without space limits, but that it does not work well in limited space conditions, such as under existing slabs in underground levels. Indeed, in this last case, the stoppages required to disassemble the drilling rods lead to poor inclusion characteristics, and non-homogeneous results. It is the Author's opinion, therefore, that in limited space cases, other techniques are more suitable.

▪**Applications as shoring elements**: Jetted columns can be used as vertical or slightly inclined shoring elements. Jetted shorings can be created by discontinuous, tangent or secant jetted inclusions, with or without reinforcement. By reducing the spacing from 600mm (tangent column) to 500mm (standard spacing for secant column shoring design), and even down to 400mm (rarely used, only when working under high water pressure and stringent impermeability requirements), it is possible to cope with very difficult geotechnical and hydraulic conditions, and to build impervious jetted diaphragms.

Tests and numerous construction site observations have shown that the quality of the interpenetration between adjacent columns does not depend on the interval of time elapsed between their formation, which allows flexibility in the construction sequence. Reinforcement is inserted in the elements in the same manner as in foundation applications. Odd columns are reinforced, whereas even columns are generally left without reinforcement. Often, reinforced columns are built first, and then non-reinforced columns are jetted to fill the gaps.

The flexural resistance of the reinforcement is determined as a function of the horizontal soil pressure, and the amount and intensity of lateral support offered by stays, passive tendons, or prestressed anchors which are pre-determined during project planning.

Tendons (Tension)

▪**Passive Tendons**: The Author and coworkers have developed various designs incorporating the use of jetted inclusions as passive tendons (Hlobil & Oboni, 1991). Passive jetted tendons must be reinforced with a steel element, generally a rod. Rods are mounted with a plate head, and loadings are transferred from the diaphragm shoring to the tendons by means of steel, or reinforced concrete horizontal beams. In the case of steel beams, the plate is used to keep in place a wedge that connects the tendon to the beam, whereas in the case of reinforced concrete beams, the plate head is embedded within the reinforced concrete.

▪**Prestressed Tendons**: A newly patented application of Jet-Grouting I was developed in Switzerland by a specialty contractor in 1995. The Author's company has been contracted to monitor, evaluate, and report on a number of tests performed using this new technique.

THE PROJECT

The cooling of Nestle world headquarters in Vevey, Switzerland, utilizes water from Lake Geneva, which is pumped via two pumping stations, built in 1959 and 1974 respectively. These facilities were to be replaced by a new station which complied with the latest environmental regulations. The new facility was (Figure 2) required to be underground, adjacent to the headquarters, and connected to the lake bottom by two parallel, 500mm external diameter pipes. These pipes were to leave the chamber and slope at an angle of 12° from the horizontal (Figure 3) towards the lake bottom, crossing very soft soil strata just beneath a rocky dike present at the lake shore.

The new 6m deep, 20mx20m chamber was designed to house heat exchange units and pumps. The proposed structure was to be built 4m below lake water level, in an area formerly occupied by a marina, which was filled in, around

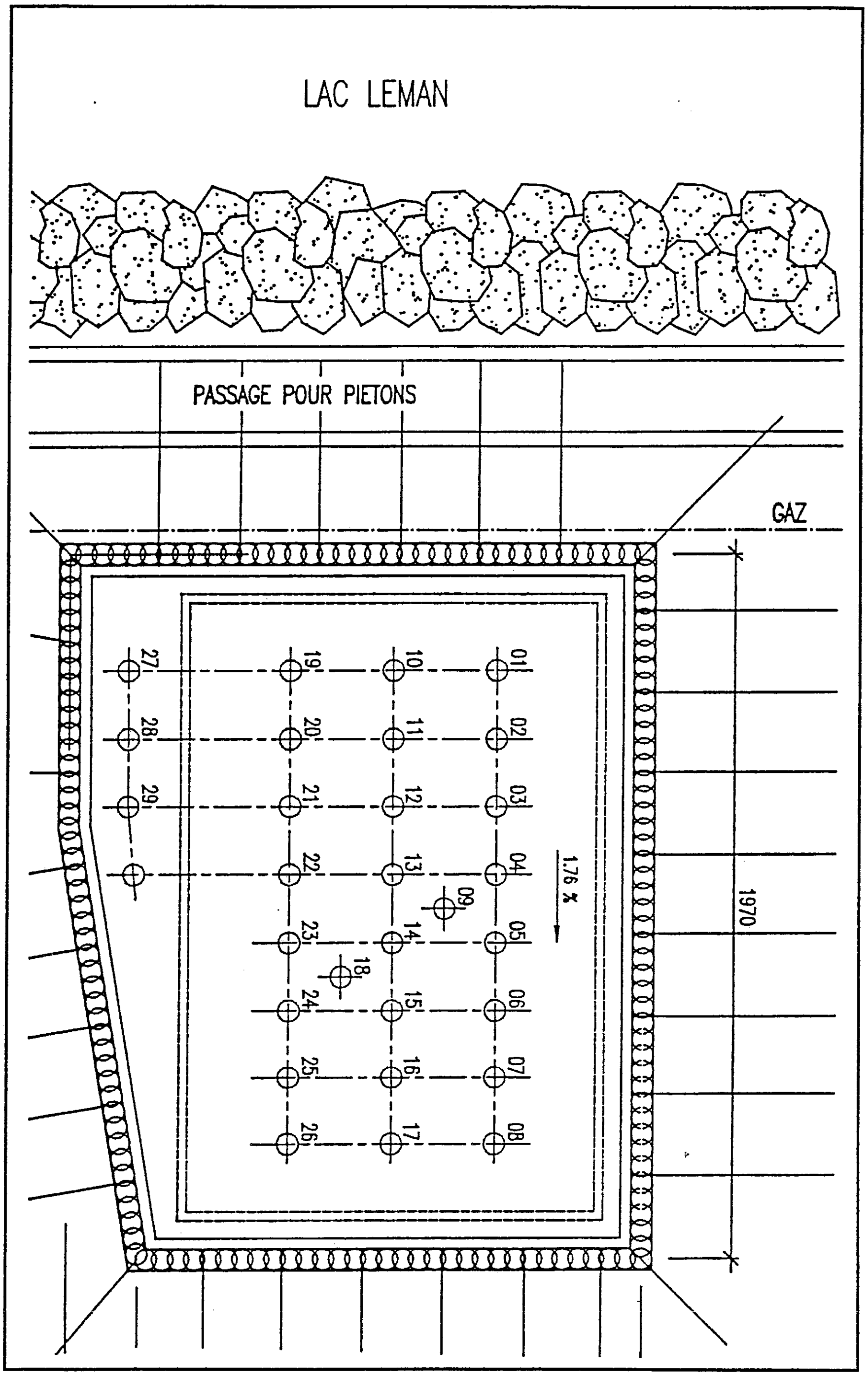

Figure 2 Plan view of the chamber. Note the lake (Lac Leman) and the pedestrian passage (Passage pour Pietons)

the year 1930, with random fills, including demolition wastes and other residues. The natural soils of the site, located at depths ranging from 3m to 4.5m, are silty sands and silts with gravelly inclusions.

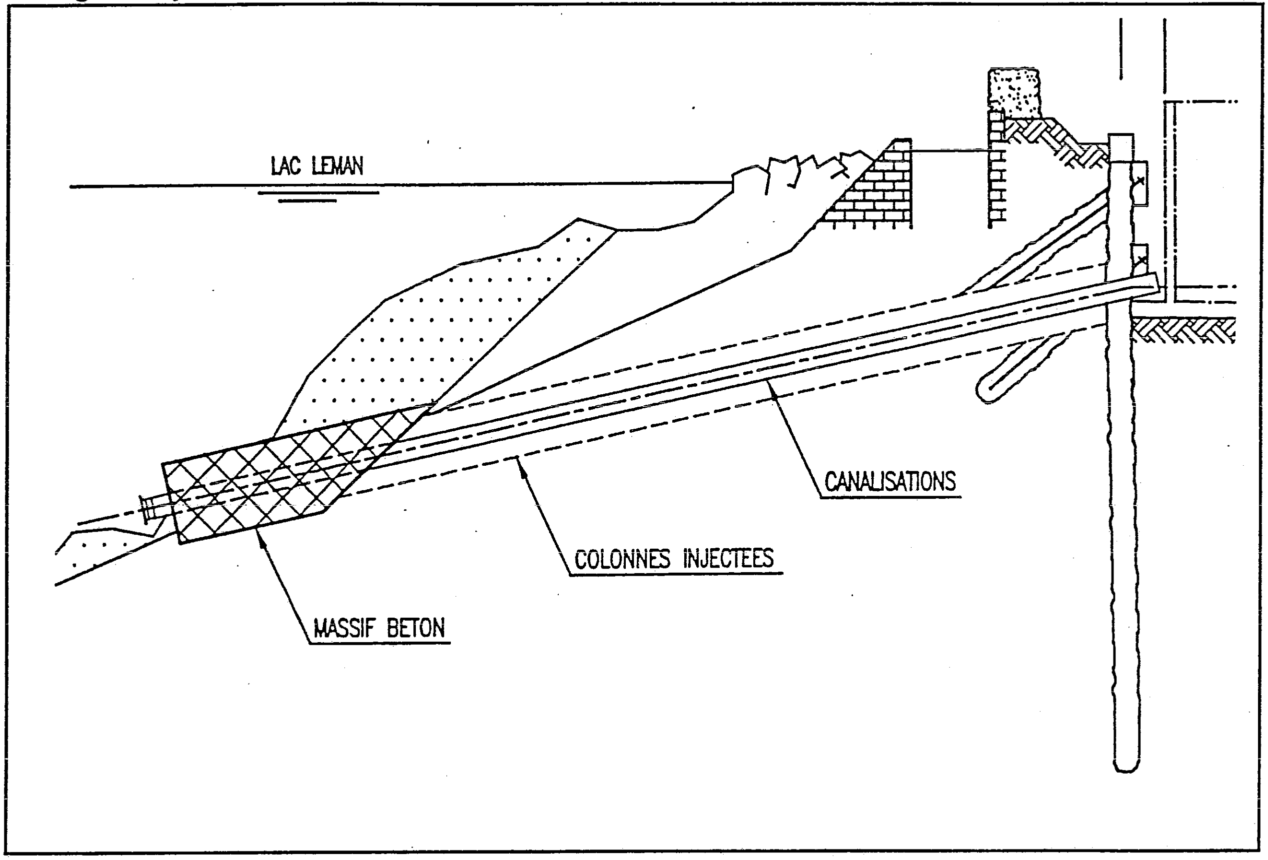

Figure 3 Profile of the 500mm diameter pipes connecting the chamber with the lake bottom.

Construction constraints included a high water table, the need to limit noise and vibrations, and the possible presence of obstacles to drilling in the random fill. In addition, strict noise and vibration control was enforced by the owner. A construction technique based on the installation of jetted (Jet-Grouting) inclusions was selected for shoring. Uplift forces on the chamber also required the installation of vertical tendons to mitigate buoyancy; steel rod reinforced jetted elements were used for this purpose.

Excavations and Shoring

As indicated, the site was filled, around the year 1930, with various material, including demolition debris from a major hotel. And on-site underground water is in direct link with Lake Geneva Lake water, approximately two meters below grade.

Other factors to be considered at the site included the high permeability of the sandy matrix, and the space constraints imposed by the client. In addition, the site was in a high visibility location, and a number of legal obligations to the general public regarding non-interference with walking routes had to be accommodated. It became evident at a very early stage of project design that shoring would be required in order to to comply with this complex set of constraints.

Given the high permeability of the soils, and the direct hydraulic connection with the lake, the shoring had to be impervious to water, and had to be present at sufficient depth so as to limit water inflow from the bottom, and avoid internal erosion or hydraulic failure of the bottom of the excavation.

Slurry trench walls and diaphragm wall alternatives were rejected on the grounds of their sensitivity to the presence of blocks and unknown materials in the fill, their initial foreseeable cost, and the risk of litigation with the contractor.

Sheet pile alternatives were also rejected because of the obstacles present in the ground, and because of noise and vibration concerns.

Ultimately, Jet Grouting I was considered to be the only safe, reliable, and viable method for building the necessary shoring. The procedures selected involved the application of Jet Grouting I (Figure 4) for construction of three structural elements:

- A shoring face, made of 600mm columns installed in a plane at intervals of 450mm, reinforced with HEB 100 steel profiles inserted in 24hour old columns by means of redrilling and grouting. The heads of

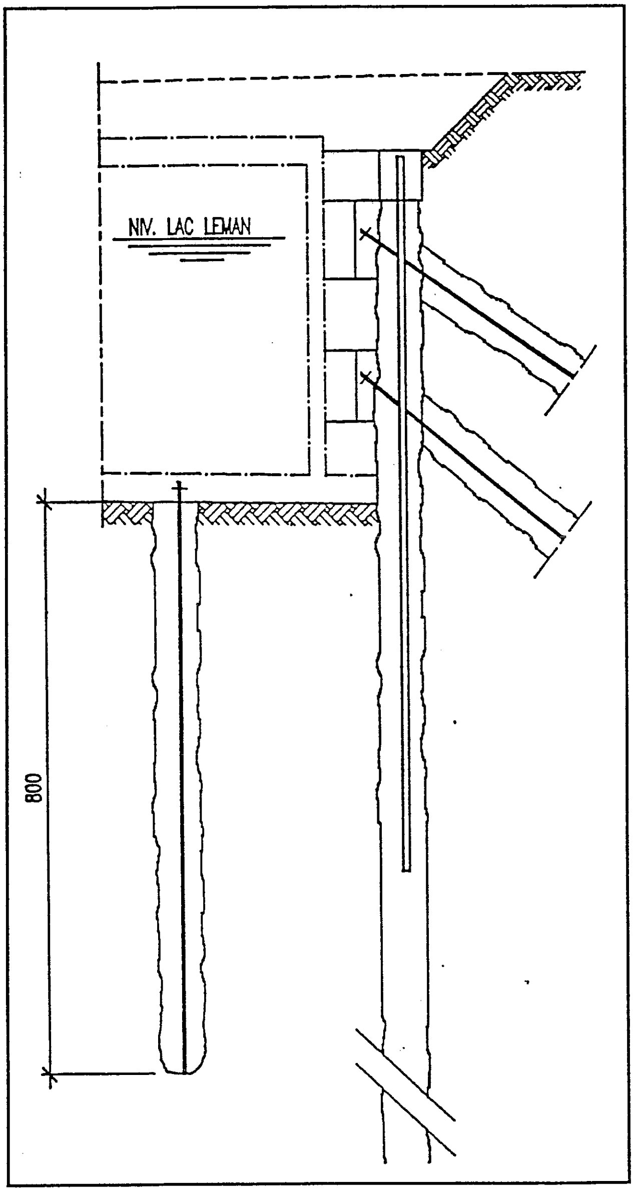

Figure 4 Typical cross section of the shoring with two rows of passive oblique tendons. Vertical passive tendons at the bottom of the excavation. to counteract hydraulic pressure.

the columns were linked to each other with a reinforced concrete cap beam, describing the perimeter of the excavation.

- Passive tendons, made of Jet Grouting I columns, reinforced with 26mm diameter steel rods, equipped with a steel plate heads, and linked to each other with reinforced concrete horizontal beams. Due to the depth of the excavation and the hydraulic pressures, two layers of tendons were built.

- Vertical passive Jet Grouting I tendons, installed at the bottom of the excavation to counteract hydraulic pressure acting under the bottom of the structure. The weight of the finished structure was not sufficient to safely equilibrate this hydraulic pressure, so an adequate safety margin was introduced in the form of passive tendons, which were reinforced with 40mm diameter steel rods.

Dewatering of the excavation during construction was performed by means of ten wells equipped with submerged pumps.

On-site construction was conducted in the following stages:

- Preliminary excavation and preparation of a working platform, approximately one meter below grade.
- Construction of the shoring face.
- Construction of the reinforced concrete cap beam and simultaneous construction of the dewatering wells.
- First excavation phase: excavation, construction of the first layer of passive tendons, construction of the first peripheral horizontal concrete anchor beam.
- Second excavation phase: same as the first excavation phase, plus implementation of the vertical passive tendons for hydraulic pressure control.
- Third excavation phase: excavation completed and beginning of structure construction.
- Preparation of soil reinforcement for trenchless installation of the parallel pipes to the lake.

Once the reinforced concrete structure was finished, dewatering was stopped, and trenchless work for the installation of the parallel pipes within the soil reinforcement was begun.

Trenchless Installation of the Parallel Pipes

In order to allow for construction of the two pipes with a minimum disruption of the site and the lake bottom environment, a trenchless excavation was done: the soils were first reinforced by an array of parallel jetted elements sloping towards the lake, and then the reinforced materials were bored in three phases.

During all the phases of this project, technical monitoring, and direct observation of the effects of the work on the natural habitat was performed by underwater divers. In order to maintain hydraulic pressure equilibrium, the boring of the reinforced soils was conducted after stoppage of the dewatering pumps.

The first phase of boring was done with a 200mm diameter tool, which was pushed towards the lake. When this first insertion was completed, divers replaced the 200mm tool with a 450mm one, which was then pulled from the lake to back to the chamber. Phase three involved replacement of the 450mm tool with a 600mm tool, which was then also pulled from the lake towards the chamber. These procedures created two parallel cavities with the necessary constructional tolerances to allow for insertion of the 500mm diameter pipes. The pipes were then inserted, and grouted into place.

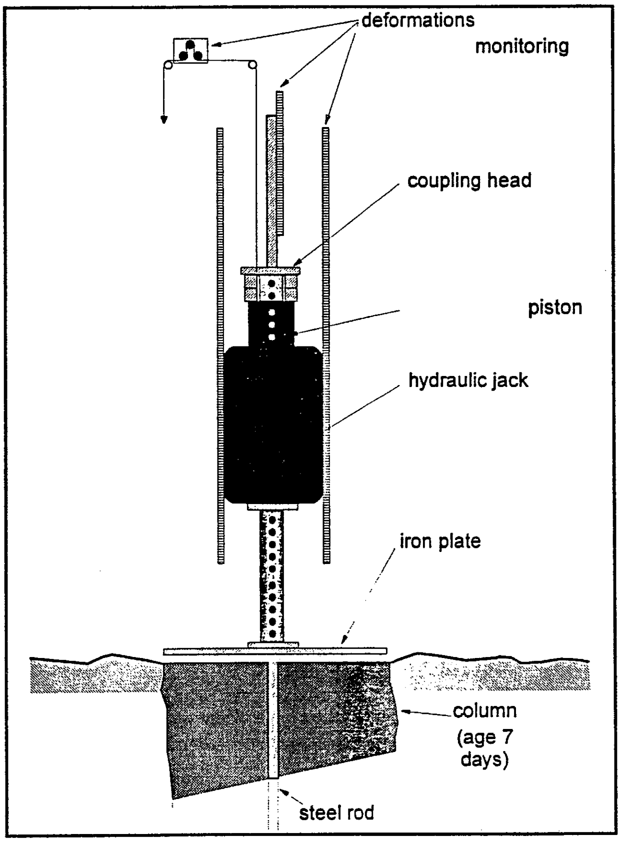

Figure 5 Experimental device for the 40mm diam., 4.95m long socketed bar in a vertical jetted inclusion

EXAMPLES OF LOAD TESTS ON JET-GROUTING I ELEMENTS

Grout-Steel Tendon Slippage Test

Pull-out tests of steel reinforcements have been performed on several occasions to study the efficiency of the grout-steel bond. A steel bar ($\O_b$=40mm, A_b=1257mm^2, P_{bu}=628kN) was inserted a distance of 4.95m into a freshly jetted core, and left for 7 days before testing (Figure 5). This test featured standard construction site installation conditions for both the jetting, and the reinforcement insertion.

At P_b=500kN, which corresponds approximately to the limit of elasticity of the bar

(σ_b=397N/mm^2), the deformation of the system was δ_b=5mm. By assuming a Young modulus of the steel equal to E_b=210000N/mm^2 and applying Young stress deformation formula (assuming that there was slippage of the bar relative to the grout in the top part of the jetting), a theoretical slippage length of l^*_b= 2.65m is obtained.

Since the bar was inserted 4.95m in the jetted inclusion, and 0.9m was free in the air between the head of the column and the testing device (Figure 5), it may be assumed that the transmission length necessary to transfer the loading in the tendon to the grout was l_r=3.20m.

In this case $l_r/Ø_b$=320/4=80>65, which is the minimum transmission length ratio for a bar under tensile loading anchored in concrete, in accordance with Swiss codes.

Grout-Ground Slippage

Figure 6 shows diagrammatically the results of one of these tests, conducted in a silty-clay terrain presenting the geotechnical characteristics listed in Table 2.

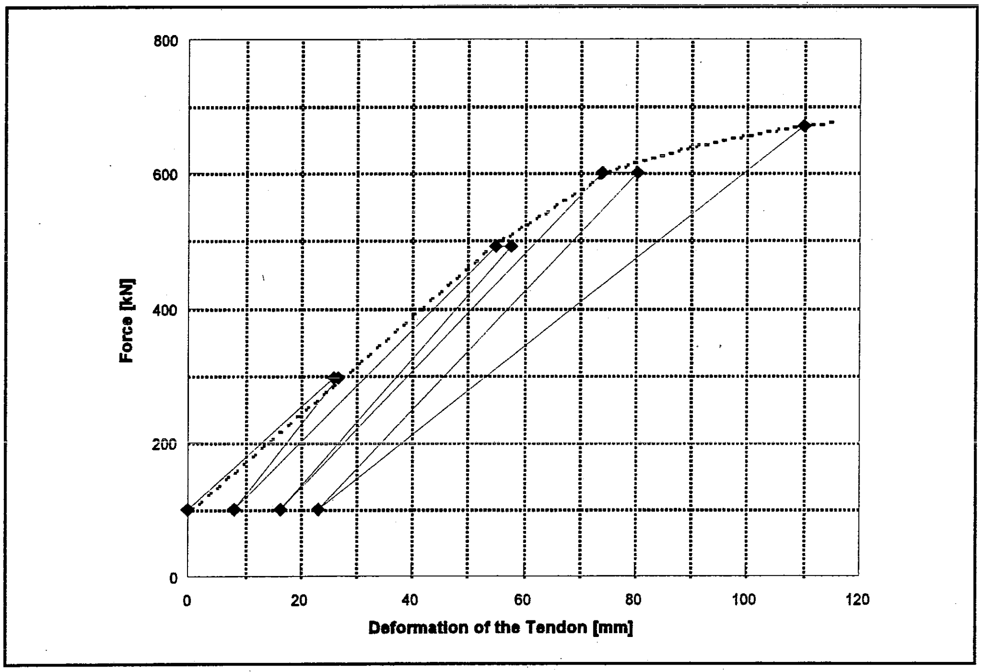

Figure 6 Force-Deformation Relationship Measured During a Pull-Out Test of an Anchor in Silty-Clay.

The anchor had a free length of 5.5m, and an angle of 20° with the horizontal. The jetted inclusion, which was 3.6m long, was located at a depth between 2.5m and 3.5m.
The anchor failed under a pulling force of approximately 500kN.

Table 2 Geotechnical characteristics of the silty-clay terrain for the anchor test of Figure 6.

γ [kN·m^{-3}]	ϕ' [°]	c' [kPa]	S_u [kPa]
19-20	26-28	0-5	40-50

CONCLUSIONS

Jet-Grouting is a versatile special work technology that allows, even in its simplest monofluid form (Jet Grouting I), construction of isolated or grouped columnar, underpinning and shoring elements made of native soils and cement grout.

Low permeability diaphragms can also be built, but construction tolerances become the limiting factor as depth increases, so that non-treated surfaces can finally significantly decrease the efficacity of the diaphragms. Jet-Grouting I can also be used to create passive tendons, and to reinforce soils for trenchless works.

To date, only a limited number of academic studies of the Jet-Grouting process have been conducted. Much, however, has been learned through the practical application of Jet-Grouting techniques.

Challenging structures may now be built with confidence, based on a vast database of construction projects, which have been monitored for performance in a wide array of difficult geotechnical and geological situations.

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