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ARTIFICIAL GROUND FREEZING IN BELGIUM

1. DEVELOPMENT OF THE GROUND FREEZING TECHNIQUE

1.1. Introduction

The ground freezing technique came up more than a hundred years ago to allow mine-shaft sinking through soft water bearing formations in Germany, France and Belgium. Belgium, more particularly, has been very active in this field since the beginning of the century when the mine-shafts in the eastern part of the country were sunk.

In view of coal mining at great depth, 14 shafts had to be sunk through loose water-bearing formations up to depths of 620 m. One remains admiring before the audacity and the imagination of the mining engineers who, more than sixty years ago, succeeded in realizing vertical freezing at such depths, considering the limited mechanical means at the disposal: slow single-stage compressors which during summer hardly could supply temperatures of $-16\,°C$ at the brine, deep drilling methods (600 m), without any correction possibility for deflections being inevitable at this depth (presently directional turbine drilling). Until some twenty years ago, this method was only occasionally used in civil engineering. It isn't until recently that the application of this technique, which has become current for mining works, was extended to civil engineering and became practically of common use.

Introduction of this technique into civil engineering has been difficult because of the multitude of new problems to be solved. Generally, the civil engineers only had relatively hazy notions about the use of ground freezing. The prevailing opinion was that this technique was too expensive and too slow. Its application was reserved to desperate cases.

Realization of ground freezing in an urban environment set serious technical problems. Knowledge of the mechanical characteristics of the frozen soils had remained too intuitive to be able to justify, by calculations as for the other civil engineering works, stability of excavations in frozen soils. Sollicitations on the frozen soil masses in very varying projects were notably more complex than sollicitations applying to the ice walls protecting the mine shafts for the execution of which empiricism was almost sufficient.

In this respect, it might be interesting to recall to some of the problems which had to be solved.

1.2. Technological developments

In view of meeting the civil engineering requirements, one had to develop refrigeration plants being 3 to 4 times more powerful than those currently used for shaft sinking. The capacity per meter of freeze tube had to be raised from 50 kfrigories/hour to more than 200 kfrigories/hour. Owing to the very limited surfaces available in the middle of urban centres, compact powerful refrigeration plants, with a double stage, had to be constructed to reach the lowest temperatures and had to be assembled into one unit: compressors, condenser, evaporation and cooling tower. These freeze units had to operate with an advanced automated system to reduce as much as possible manual labour costs.

Since boring and installation of the freeze tubes had to be realized on limited surfaces—rooms measuring hardly $2 \times 3 \times 3$ m—powerful compact drilling-machines had to be adapted and freeze tubes, sometimes in pieces with a length of less than 1 m, had to be installed. In view of reducing delays, current boring by rotation with injection of bentonite was replaced by percussive boring with injection of compressed air.

Where generally freezing was carried out in water-saturated soil below immovable water-table for shaft-sinking, freezing had now to be carried out in just wet, almost dry soil outside the hydrostatic level. Sometimes freezing had to be carried out in formations affected by active water circulations, which were not only natural in the valleys but also artificial because of the passage of watertight sewers or groundwater lowering.

Installation of freeze pipes through obstacles of all kinds such as foundation debris, sheet piles, anchors, cavities, etc., was more difficult than for shaftsinking.

1.3. Design of excavations in frozen soil massives

For shaft sinking realized inside a cylindrical ring of frozen soil, one was satisfied with a simple compression calculation and the used formulae were limited to the formula of Lamé adapted by Domke. This formula was based on uniaxial compression strength obtained by laboratory tests. This was unanimously adopted for characteristic frequently wet soils, namely sands, clays and water saturated silt. For what civil engineering is concerned, the situation is much more complex.

Following structures must be studied:

- vertical frozen soil walls submitted to
  - horizontal pressure, inducing bending, traction, shear;
  - taking over of vertical loads, causing bending, compression, buckling;
- horizontal slabs of frozen soil submitted to
  - vertical pressure, and consequently to bending, traction, shear;
- combinations of vertical walls and horizontal slabs.

On the other hand, while for mining works the definitive lining is installed as soon as possible after excavation, thus limiting the sollicitation duration of the frozen soil wall, it frequently occurs in civil engineering works to leave the frozen soil walls in activity during a longer time, even several months, from which results a phenomenon which was badly known up till then, the phenomenon of creep of the frozen soil.

In view of solving these new problems, one had to carry out rapidly a thorough study of the mechanical characteristics of the frozen soil and of its behaviour in situ under the imposed sollicitations and chiefly to call out the factor "time".

A dynamic and intensive co-operation between the various consulting engineers, universities and the company specialized in freezing, has allowed to give practical answers to these questions.

We believe that Belgium, thanks to the dynamism of our engineers and more particularly thanks to the multitude of works giving the research workers the occasion to a permanent confrontation with the working sites, had made a remarkable progress in the practical knowledge of frozen soils.

Just like for the introduction of the ground freezing technique for shaft sinking, Belgian engineers are co-leaders in the progress for the application of this same method in civil engineering works.

In this respect we refer to some realizations carried out in recent years. Since 1960, more than fourty different ground freezing jobs have been realized in Belgium (Dom and Gonze, 1983). To illustrate this activity, we give hereafter some recent realizations carried out in this country.

2. EXAMPLES OF RECENT APPLICATIONS

2.1. Antwerp subway

Since 1975 different methods for the realization of the underground subway have been envisaged. One of these methods consisted in ground freezing. An in situ test allowed to conclude that this technique could be applied successfully for the passage under blocks of buildings over lengths up to 210 m (Hemerijckx, 1984).

In situ freezing test

The soil is loose. It consists firstly of 2 m of remolded and heterogeneous sand with pebbles and foundation debris. Then one finds 3 to 4 m quaternary soil composed of silt and clayey sand with shells. From 6 m depth, one finds a layer of about 3 m of fine glauconifer sand with shells.

Ground freezing must be realized in these first three layers. It is to be noted that the groundwater level, which is normally found at a depth of 5 m, has been lowered by successive pumpings and that freezing is consequently carried out above this level.

In August and September 1975, a small-scale freezing test has been realized under buildings, in view of determining the strength of the frozen soil, as well as swelling of the soil and settling after freezing (fig. 1).

Therefore a strutted excavation of 2.50 × 3.50 m has been dug at a depth of 3 m.

Starting from this excavation, horizontal freezing has been realized under a bearing wall of the building. Next, the strutted excavation has been deepened so as to be able to excavate a depth of 30 cm underneath the frozen slab. Different reference points on the building walls have allowed measurement of the settlings or risings.

A maximum rising of 2.6 mm of the bearing walls and a difference between rising and settling of the raft of the cellar of 4.5 to 6 mm has been found.

It could be concluded that the movements in the bearing walls (under load) are minimal and smaller than those recorded in the raft (free). These satisfactory results have led to a positive conclusion for what the use of the method is concerned.

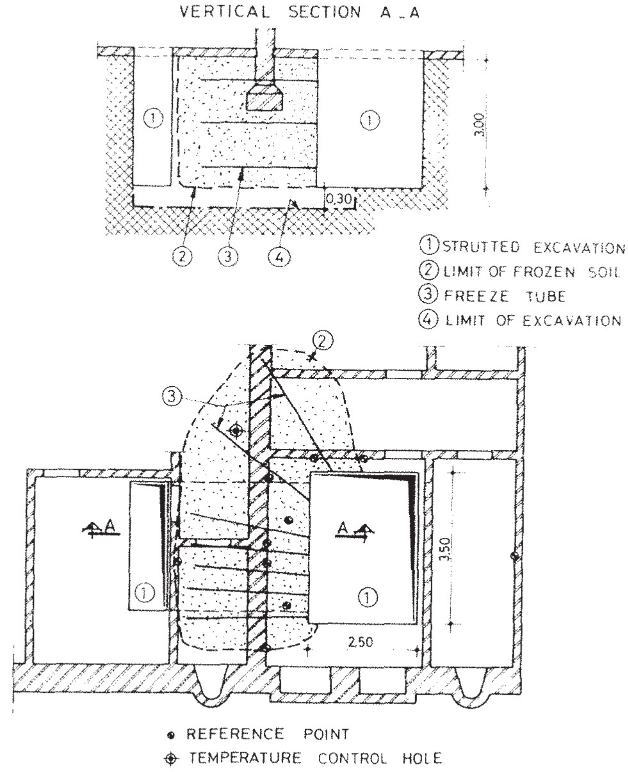


Fig. 1. — In situ freezing test in Antwerp.

Van Wesenbeke tunnel in Antwerp

This tunnel with a total length of 450 m is part of the northern extension of the first axis between Astrid Station and Elisabeth Station (fig. 2).

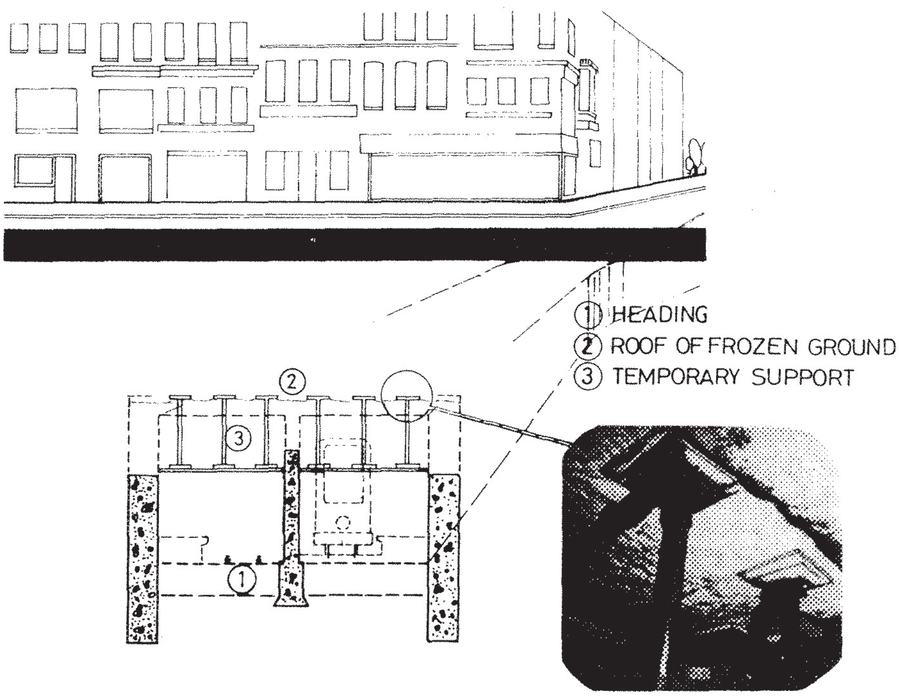


Fig. 2. — Principle of the execution of the Van Wesenbeke tunnel in Antwerp.

A part of approximately 210 m of this tunnel has been realized under the protection of a flat slab of frozen soil passing hardly 1 m underneath the foundations. The works were started early 1979 and the execution time was 36 months. The principle of the method consists in creating a slab of frozen soil with a theoretical thickness of 1 m under which excavation was carried out, including installation of 400 kN props on a square of 2 × 2 m.

To limit the total freezing duration, thus reducing the effects of rising, freezing has been realized in successive phases.

In the beginning, sometimes relatively important risings and settlings were recorded, which were mainly due to the fact that it is difficult to pour a concrete slab which sticks perfectly to the excavated roof; on the other hand, multiple obstacles were met such as wall debris and old wells.

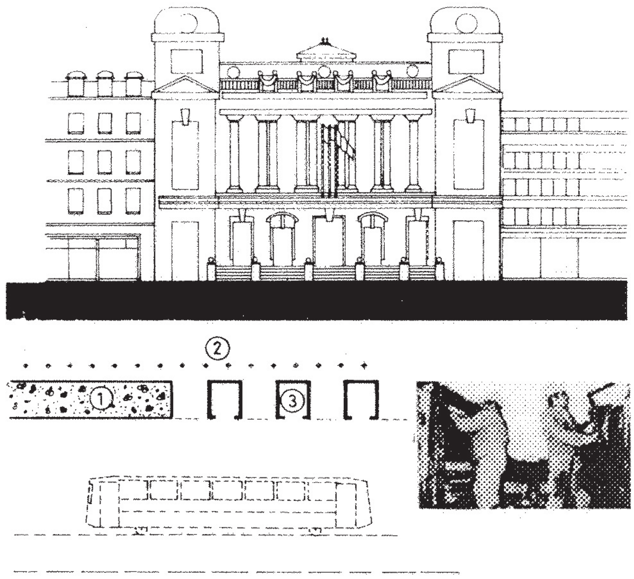
One of the problems consists in the contact between props and the frozen slab; an isolating plank in azobe wood measuring 80 × 80 cm has been placed between the head of the prop and the frozen soil, instead of the initially foreseen planks of 60 × 60 cm. Unfortunately, this support is located in the least resistant area of the ice roof in contact with ambient air, because at this place the highest temperatures and consequently the lowest resistances are recorded. To solve the problem arised by the contact of concrete with frozen slab, injections with cement were carried out so as to fill up the inevitable cavities left after concreting.

Thanks to the freezing method it has been possible to realize the subway tunnel over a length of 210 m, completely underground underneath a garage, a studio complex and eight houses. However, many unforeseeable obstacles have been met. The freezing method presented the great advantage of adapting easily to these situations, even there where one had to pass at 1 m underneath the foundations.

For what risings and settlings are concerned, it was established that they were practically negligible under loaded foundations but that they were more pronounced in less loaded fine sand areas, or in mud pockets. If this is possible, it would be better to avoid excavation underneath a frozen soil slab with the use of props, considering the important punctual sollicitations resulting from the contact of the head of the pillars with the frozen slab. The realized system approaches the flat slab system which is well known for its deformability. Pouring of a horizontal concrete slab against a horizontal frozen roof appears to be difficult.

Opera-Astrid tunnel in Antwerp

This tunnel, with a length of 400 m, connects Opera Station to Astrid Station and goes underneath the buildings of the insurance company "De Vaderlandsche" (fig. 3).



1 CONCRETE ROOF IN CONSTRUCTION
2 FROZEN GROUND
3 TRANSVERSE GALLERY

Fig. 3. — Principle of the execution of the Opera-Astrid tunnel in Antwerp.

The Opera constructed in the beginning of the century on backfilled trenches from the 16th century, rests on remolded soils comprising a thin layer of mud, having a thickness of 0.8 m to 1 m, at a depth of 6 m to 7 m.

This layer keeps its water in spite of draining and cannot be grounted. Before starting the works, the whole of the foundations were in bad condition. To consolidate temporarily this silty layer, freezing of this layer was carried out over a thickness of 1.50 m at an average temperature of − 10 °C, starting from a longitudinal gallery (fig. 4).

Underneath this frozen zone, small transversal galleries with a width of 1.65 m and a height of 1.90 m were excavated in two phases, first the primary galleries B, and then the secondary galleries J (fig. 5).

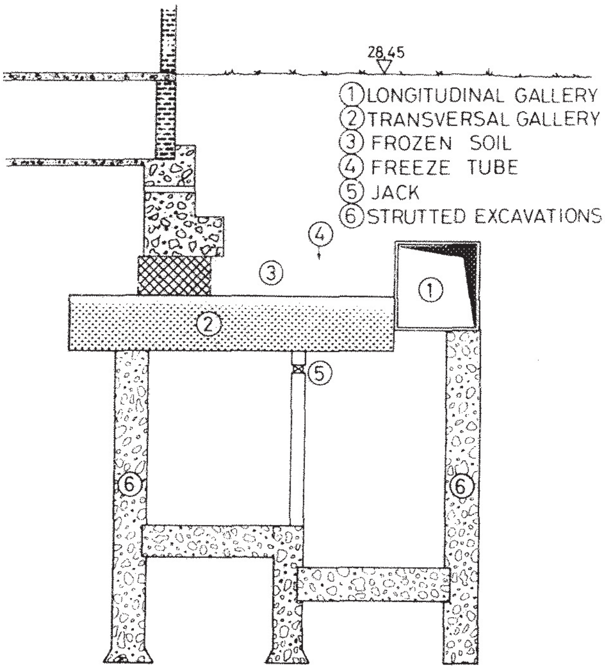
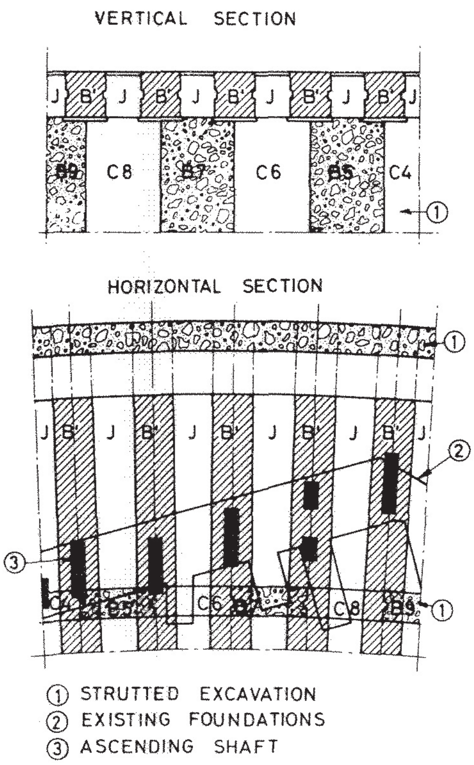


Fig. 4. — Detail of execution at the Opera-Astrid tunnel in Antwerp.



ig. 5. — Detail of underpinning at the Opera-Astrid tunnel in Antwerp.

To allow underpinning of the building foundations mmersed in the frozen slab, ascending shafts, having section of approximately 2.5 m, were dug out in the oof of the primary galleries, over a height of about 1.5 m in the frozen slab. These small shafts and the galleries were subsequently filled with concrete carefully grouted to underpin the heavy foundations through these concrete blocks on the supported excavations. After this operation, the secondary galleries were dug out and one proceeds the same way to underpin the loads of the foundations to the supported excavations.

The settlements have been carefully measured and may be summarized as follows:

- dewatering and digging working gallery: 8 mm
- formation of the ice: 0 to 2 mm
- digging primary galleries: 9 to 17 mm
- digging secondary galleries: 2 to 3 mm
- thaw and final works: 2 to 7 mm

Total settlement: 21 to 33 mm

It may be concluded that freezing has allowed to realize successfully this operation and has prevented catastrophic settlements. During the freezing period, no notable heavings have been recorded, and the settlements recorded afterwards are mainly due to the very complex digging works and to concreting.

2.2. Brussels subway

The design department of the Brussels subway (S.T.I.B.) has chosen the ground freezing technique for the realization of particularly difficult passages:

- passage under the boulevard Anspach;
- passage under a cable network and pipings in the rue de l'Ecuyer;
- passage under a wing of the Palace Hotel;
- connection of the underground canalized river Senne;
- passage under a road tunnel at the Place Rogier;
- passage under the North-South railway connection at the Midi Station;
- passage under the underground Senne Canal, near the South Station;
- passage under the Côte d'Or factory;
- passage under the buildings at the Porte Louise.

To illustrate the fast progress made in the application of the ground freezing technique, we hereafter give some realizations which are spectacular because of the originality of their conception and the complexity of their execution (Hulet, 1984), (Woitchik, 1984).

"South Station" subway station

The South Station or Gare du Midi-Zuidstatie is the most important railway station in Belgium. The Midi subway station, which has a length of 200 m, is a work comprising several levels, the raft of which is situated 22 m under the road level. It passes underneath a railroad bridge, the piles of which rest on the bottom of the valley of the Senne river, composed of alluvial layers which are based on Ypresian, composed of sandy clay becoming more watertight with depth.

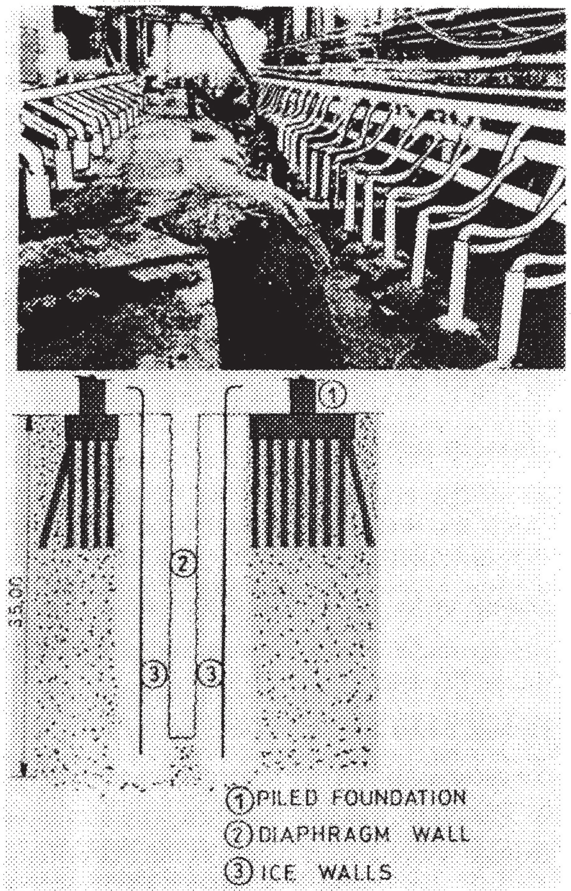


Fig. 6. — Principle of the execution of the "South Station" subway station in Brussels.

The station is surrounded by diaphragm walls dug from the surface under bentonite mud (fig. 6). These walls have following functions:

- since they are embedded at the bottom in the watertight Ypresian, they constitute a closed watertight enclosure, allowing inner dewatering;
- they have to support part of the edge of the works;
- they serve as retaining walls.

Due to the heavy loadings of the railway bridge the bentonite mud does not assure efficient protection against lateral collapses. In addition, the soil around the excavation decompresses. It was unthinkable to take such risks with the existing works, namely the piled footings of the railway bridge giving passage to one of the most important European railroad connections. This problem has been solved by freezing.

Ice walls have been formed previously along the future diaphragm. These ice walls are realized continuously from the foundation level of the adjacent works until the base of the diaphragm walls (fig. 6).

These ice walls has been executed for four reasons:

- supporting the neighbouring piled footings during excavation of the diaphragm walls;
- reducing soil pressure on the lateral faces of the bentonite wall;
- assuring conservation of the inner soil structure by avoiding its decompression;
- limiting propagation of possible collapses.

This juxtaposition of diaphragm walls in the soil and continuous freezing is, to our knowledge, a first realization, in spite of the exceptionally difficult conditions of their realization: 1.50 m thick diaphragm walls up to depths of 34 m, refrigeration pipes at the same depth, all this with only a reduced working height of 5.5 m.

The operation was extremely successful and the observation of benchmarks has demonstrated that freezing accomplished very well its stabilizing function. The safety of the railway bridge has been assured at any moment and traffic on its 24 tracks never had to be interrupted.

Passage under the "Côte d'Or" factory

Beyond the South Station, the subway has to pass under the most important Belgian chocolate factory COTE D'OR. It was indispensable to carry out the works without disturbing the economic activity of this factory (fig. 7).

The soil underneath the factory is composed of about 15 m alluvium deposited on Ypresian. The hydrostatic water table is situated at about 4 m under the surface level. This aquifer is the seat of currents running perpendicularly to the subway tunnel.

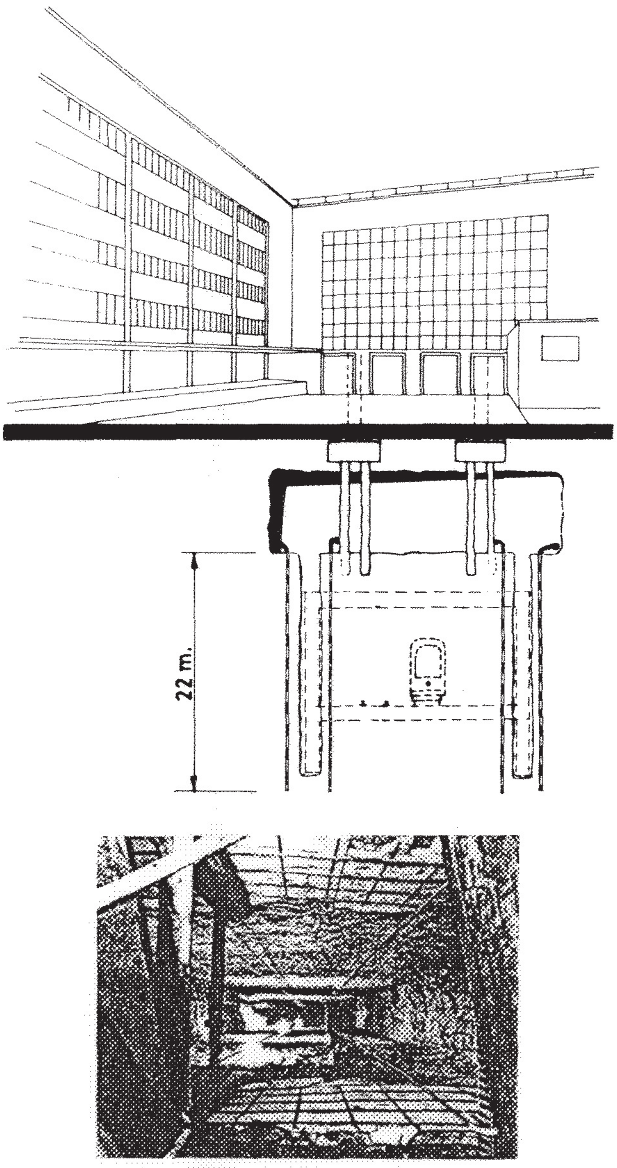


Fig. 7. — Principle and view of the excavation under the building "Côte d'Or" in Brussels.

The method adopted under the factory was the execution of strutted excavations in a massif of frozen soil starting from the excavations dug out under the factory foundations which, due to the presence of the water table at a depth of 3.75 m, only allowed a working height of 3.5 m.

The panels have been realized in elements with a length of 3 to 4 m at depths up to 22 m.

Elaborated tests have been carried out in view of studying the behaviour of the concrete cast in 1 m thick layers between the walls of the soil frozen at −20 °C.

During concreting, it is observed that the setting of the concrete develops temperatures close to +30 °C. After 7 to 9 days, the concrete temperature becomes negative and its hardening is restrained. During thaw of the frozen massive, the temperatures gradually increase and hardening becomes normal. Core samples taken in the concrete after complete hardening showed that the final quality of this concrete was rather better than that of concreting in usual diaphragm walls.

Louise subway station

In the complex of works, the subway tunnel leading to the Place Stéphanie, juxtaposed with the traffic tunnel, has been installed under the buildings at the Saint-Gilles side (fig. 8). To realize this part underneath the buildings, the consulting engineer designed the creation of a soil slab frozen horizontally under the building foundations. This protection slab allows digging of the tunnel roof, as also of the vertical walls realized by strutted excavations, starting from two galleries dug out under the protection of the frozen soil.

The nature of the soil arises a big problem. The soil is a very loose decalcified sand, with a porosity of

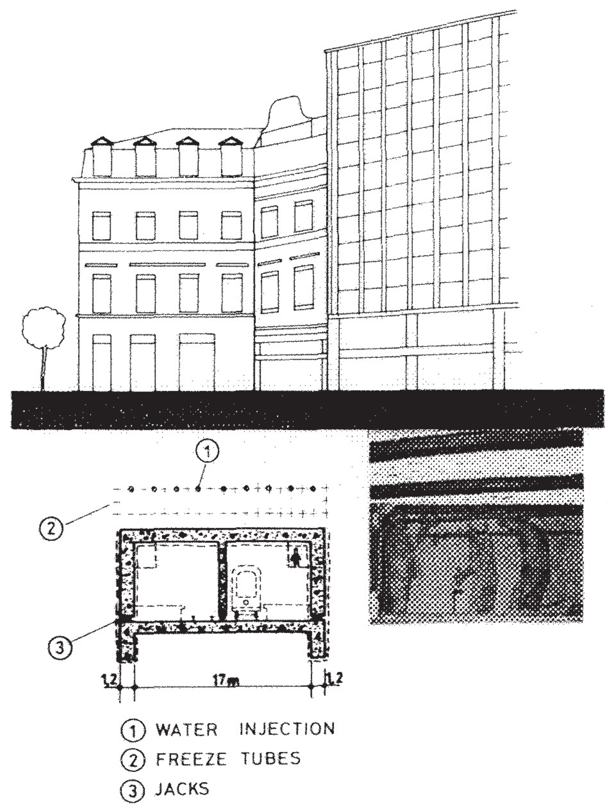


Fig. 8. — Principle and view of the excavation for the Louise subway station in Brussels.

nearly 50%. The ground water is located at a depth of ±20 m. In a preceding section of the subway, the tunnel roof had been realized by means of pipe jacking. However instability of the sand structure was such to produce large settlements at the surface. Fortunately, these movements occurred underneath the railroad and not underneath buildings. An other technique has been chosen for the realization of the next part of the subway which is completely situated underneath commercial buildings.

For this more delicate situation the consulting engineer has chosen to let the building foundations rest on a frozen soil slab during realization of the works. This method created an unusual problem: at certain places, the soil is practically dry—up to less than 2% water content—and it is thus necessary to realize its humidification to be able to realize its freezing.

Before starting the works, a preliminary in situ test had to be carried out to determine the values of the geomechanical, thermal and hydraulic parameters of the soils. The values of these parameters are then used for establishing the final project allowing realization of a good compromise between:

- as small settlements as possible by limiting drilling operation and thus the number of refrigeration pipes, freezing time and temperature, quantities and pressures of injected water;
- a sufficiently strong but not excessive frozen soil. A better knowledge of the behaviour of the frozen soil is acquired by realizing in situ creep tests.

A small vertical test shaft is firstly dug screened from freezing, in view of determining the thermal and rheological properties of the soils. Sixteen vertical freeze tubes and three borings T1 to T3, equipped with thermistors are realized (fig. 9). The water content of the soil is determined on the basis of samples taken in the borings.

The deviations of the freeze tubes and of the three control holes are carefully measured. The freeze pipes are set at cold and progress of the cold is monitored by means of thermistors located in the control holes.

This in situ test has allowed to determine:

- progress of freezing (thermal conductivity, water content ...);
- mechanical strength of the frozen sand and creep law by means of jacks installed in the shaft (Gonze, Lejeune and al, 1985);

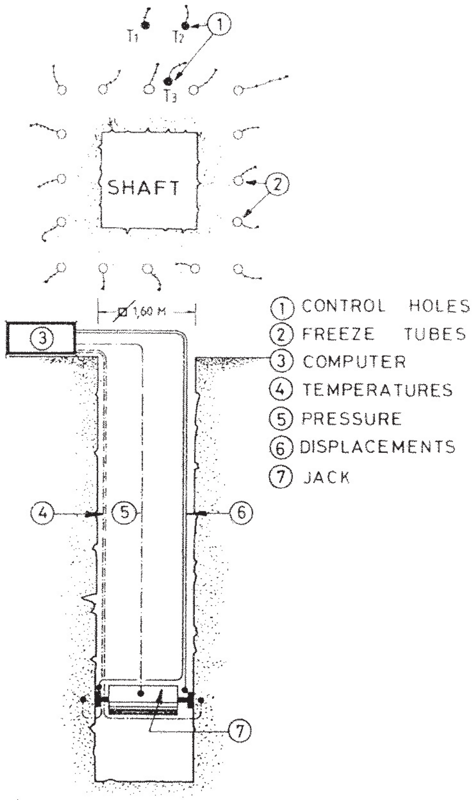


Fig. 9. — Test shaft. Louise subway station in Brussels.

- the exact composition of the frozen soil by on site sampling;
- permeability of the soil.

The conclusion of these tests was that freezing could only be applied after prior humidification of the soil, considering the absolutely too low water content. A water injection technique before and during creation of the frozen slab has been developed and gave satisfying results.

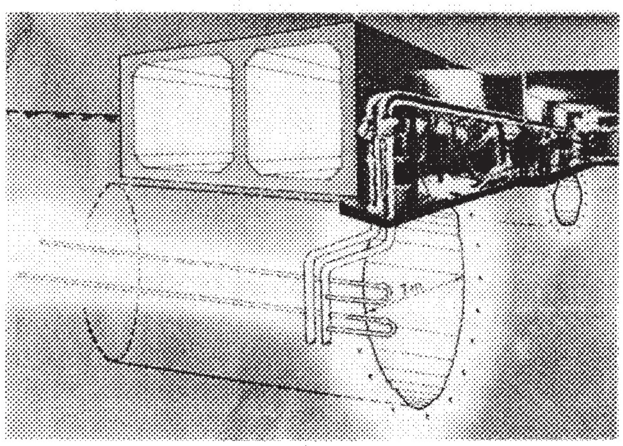


Fig. 10. — Principle of the execution of the subway underneath the river Senne in Brussels.

The works have been realized with success and no damage at all has been recorded, the soil movements due to the whole of the freezing, digging and concreting operations, generally not having exceeded 15 mm.

This result could only be obtained thanks to a detailed study including in situ tests.

Passage of the subway underneath the river Senne

The passage underneath the underground canalized river Senne, in a soil made up of waterlogged sand and gravel, is realized through two galleries with cylindrical sections under the protection of rings of frozen soil (fig. 10).

Both sections, having an 7 m digging diameter, have been lined with reinforced gunite. The concrete was projected directly on the frozen soil. Digging was carried out in full face.

Figure 11 shows the face on one of the tunnels. Figure 12 presents the installation of the reinforcing bars before projecting of the concrete.

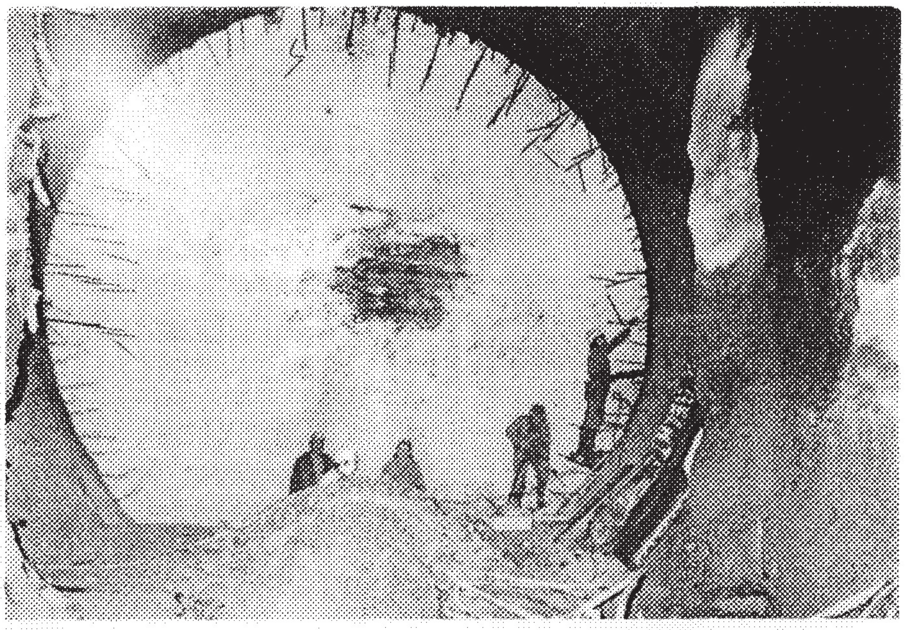


Fig. 11. — Subway underneath the river Senne in Brussels. View of the face.

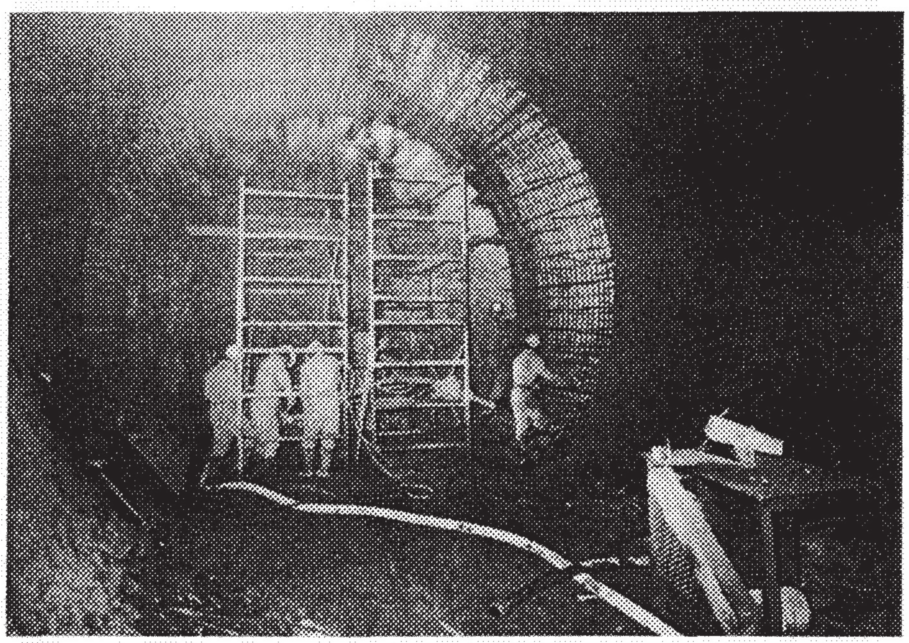


Fig. 12. — Subway underneath the river Senne in Brussels. Installation of the reinforcing steel.

To be noted:

- the importance of the section: 7 m excavation diameter;
- the application of the gunite directly on the frozen ground;
- the 1 m thick frozen soil situated directly underneath the raft of the river Senne canal which is filled with running water.

Execution of this work did not cause particular problems.

2.3. C.G.E.R. (Caisse Générale d'Epargne et de Retraite) building, Brussels

The building to be constructed comprises two underground levels and must go through the cellars and foundations of an old demolished building (fig. 13) (Scarceriaux, 1984).

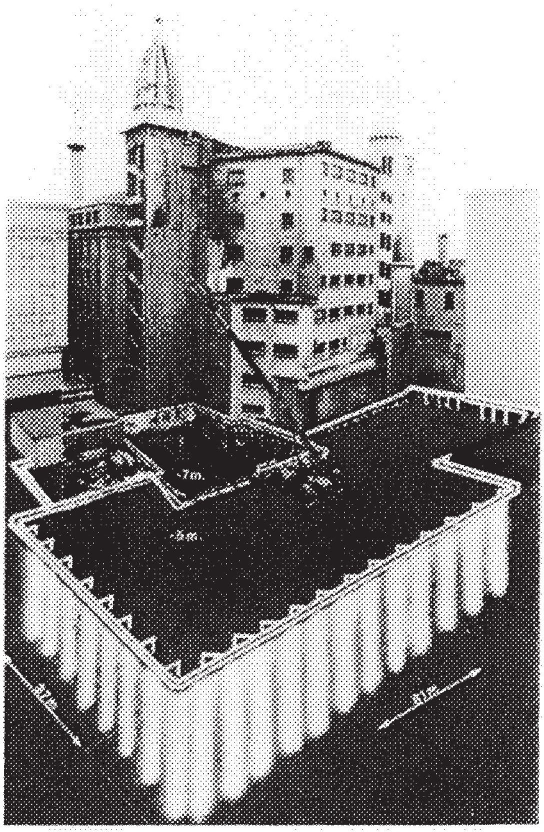


Fig. 13. — CGER building in Brussels. Principle of execution.

The soil consists of deposits constituted by alluvial clays, with the presence of a peat layer with a thickness of 3 to 4 m and, at the base, alluvial sands and gravels resting on a thick layer of watertight Ypresian clay. The hydrostatic level is situated at about 2.60 m. The neighbouring constructions, many of which erected in the 19th century, rest on shallow foundations, others on short piles not always reaching the gravel layer and consequently very sensitive to the effect of a differential dewatering. It is thus necessary to realize previously a watertight peripheral enclosure allowing dewatering inside this enclosure without influencing the ground water level outside the thus created watertight tank.

To assure this tightness, the wall of the enclosure must be continuous all over the periphery and reach the watertight layer, i.e. an average depth of 15 m, of which 12.5 m under water. The development of this wall is 275 m. Classical realization by means of diaphragm walls has not been chosen for various reasons, mainly the presence of obstacles, old foundations, old piles, pipings, as also an important loss of useful surface.

The selected solution consisted in the creation of a watertight enclosure by freezing of a vertical wall allowing to solve following problems:

- the frozen wall could be realized outside the new building, that is either in the street or underneath the foundations of the maintained buildings, causing no surface loss at all;
- at the end of the works, the ice wall disappears; there is consequently no permanent barrier and the water-table restores as before the works;
- by adopting an adequate thickness of the frozen wall, the stability of the excavation, besides watertightness of the enclosure, is assured.

Freezing was continued from April 1982 until July 1983, including preliminary tests. The projected planning has been respected and nuisances with respect to the neighbourhood were limited to:

- the flow of boring mud when drilling the freeze pipes;
- the noise of air compressors during boring with compressed air;
- a few fissures in the surrounding buildings mainly localized between buildings.

Dewatering inside the working site did not have any effect on the levels of the outer water level.

The heaving and settling movements measured by surveying the surrounding constructions, were of the order of 1 cm. The presence of the peat layer with more than 150% water required a preliminary study in view of fixing the geomechanical characteristics of the frozen peat and its heaving and settling beha-

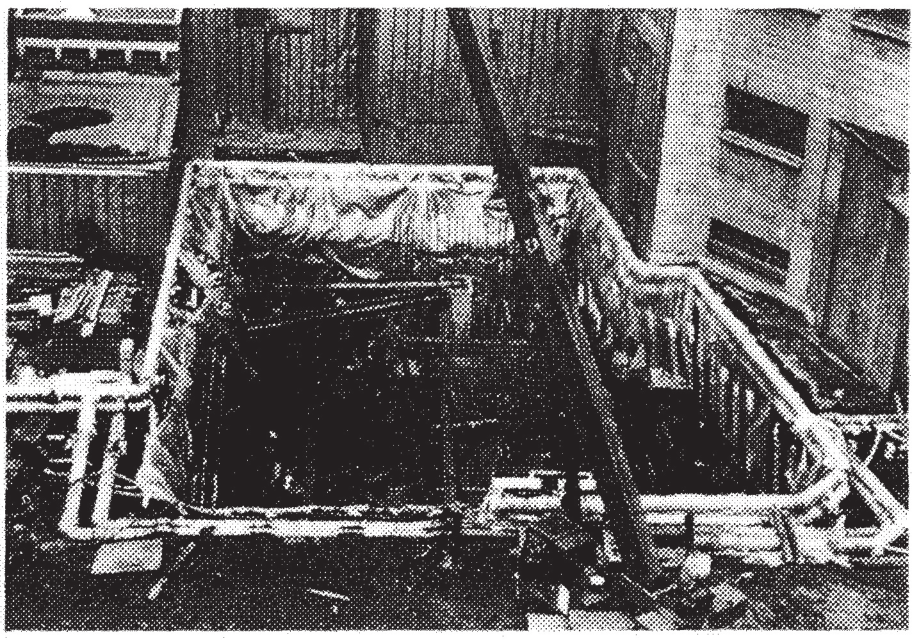


Fig. 14. — CGER building in Brussels. Detail of excavation.

viour. Therefore, an in situ freezing test has been previously carried out. During this test, we have been able to establish the influence of the water content on the formation of the frozen wall. Figure 14 shows a view of the first excavations.

2.4. Underground laboratory in the clay for the Centre d'Etude de l'Energie Nucléaire at Mol

The initial concept of the project is represented on figure 15 (Funcken and al, 1983).

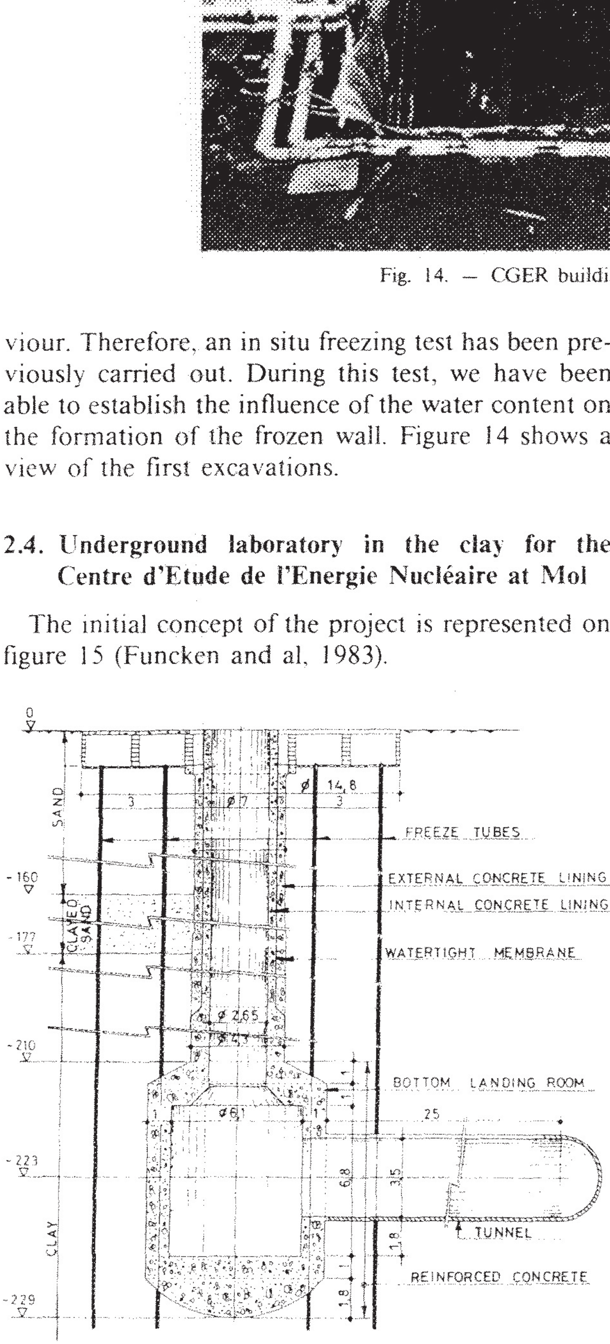


Fig. 15. — In situ laboratory in clay at Mol.

It comprises successively realization of:

- a vertical shaft with an inner diameter of 2.65 m until the depth of 214.75 m;
- an enlarged construction at the bottom of the shaft (inner diameter 6 m) allowing realization of borings for horizontal refrigeration pipes required for execution of the gallery (two rings of concentric and parallel borings around the future gallery);
- a horizontal gallery with a diameter of 3.5 m, lined with nodular cast iron segments, and having a length of 25 m; the axis of the gallery being at a depth of 222.9 m.

The freezing method has been chosen not only to insure on watertightness and stability of the excavations during shaft sinking through the aquifer sands but also to improve the digging conditions in clay.

Indeed, as it is submitted to the pressure of upper soils, this plastic clay risks to flow rapidly to the excavations. A double concrete lining is selected for the shaft. The first poured downwards, follows the digging of the shaft. It is poured directly against the frozen soil and insures temporary support of the excavations. Since that concrete lining is permeable at the construction joints, it holds back only the pressure of the buoyant soils in the final stage.

After having achieved shaft sinking, the inner concrete lining is poured in ascending way. Simultaneously, a polyethylene sheet, the tightness of which is carefully controlled, is installed between both linings. The inner lining only serves to take up water pressures.

The watertightness sheet is connected to a stainless steel ring embedded in the concrete lining of the shaft at a depth of 200 m. Under this same level, the outer watertightness of the shaft is insured by the watertightness of the clay itself, the lining of the shaft being realized in one single concrete thickness from this depth onwards until the raft of bottom landing.

Shaft sinking started on October 1, 1980. It is continued without any difficulty through the sands and the sandy clay transitional zone. But in the clay, four days after deshuttering of the concrete at a depth of 199.9 m, the lower metre of the concrete starts to crack under compression.

The origin of these cracks may be found in the fact that the part excavated under the last concreting remained too long without lining, allowing the development of the creep of the clay, in spite of its frozen state. After these incidents, the pressures acting on the lining are measured by way of total pressure cells already installed at the upper level, and convergence measurements of the clay body are performed.

Convergence measurements yields radial closing rates of the excavations of the order of one centimetre per day. At the end of the shaft sinking, the bottom of the excavations goes up at a rate of several centimetres per day. After a few days, the pressures on the lining reach values equal to or even higher than the weight of the soils, approximately 4.5 N/mm^2. Parallel to these tests, the works are continued but it appears quickly that enlargement of the digging section of the shaft to that required for the realization of the underground room could not be done without difficulties.

Indeed, an enlargement test at the depth of 207.5 m, by a conical shaped section (fig. 17) at the extrados, lead to a failure: through its vertical component, the too fast creeping of the frozen clay has induced an important horizontal cracking at the connection with the upper cylindrical section.

Consequently the bottom room could no longer be realized according to the initial conception: it was decided to go down as quickly as possible, with the shaft up to the raft by way of a section with an outer diameter of 5.8 m and an inner diameter of 4 m. The raft of the shaft was poured on December 21, 1981, under the depth of 227.5 m.

Simultaneous installation of the watertighness sheet and of the internal lining, going up, has been carried out during the first three months of 1982.

During the period of inner concreting of the shaft, many actions are undertaken in view of developing a new concept for the experimental gallery, the technical realization possibility of which was questioned again as a consequence of the incidents occurred at the shaft. A laboratory test campaign is set up in order to study the rheological properties of the frozen clay at different temperatures.

Various studies have been carried out and different experts have been consulted. The contractor realizes simulation of the observed phenomenon by way of mathematical models to reproduce, by calculations, the deformations and pressures observed in situ. This allows to attempt to give a qualitative and quantitative explanation of the frozen clay behaviour.

Confinement induced by freezing may be at the origin of the development of high pressures, this phenomenon being accompanied by a loosening and a creeping of the soils when opening the excavations. The results of these analyses bring about the conclusion that execution of the gallery is possible.

The contractor then decided to start boring, from the bottom landing, a series of freeze pipes each having a length of 20 m, diverging around the section of the future gallery as shown by figure 16, using refrigeration plant with freon (brine temperature −32 °C), such as configuration has allowed to start digging after one month of freezing.

The whole of the gallery digging works has been scheduled in two phases (gallery at depth 222.9 m).

A first phase included realization of:

- a first rectangular section, having a length of 5 m and an inner section of 1.5 × 2 m. Final support is assured by IPN 450 joining rectangular steel frames;
- a reinforced concrete section, with a thickness of 1.4 m and an outer diameter of 4 m, allowing to insure transition between the first rectangular section and the next circular section with an outer diameter of 3.5 m and lined with cast iron segments, as foreseen in the initial project;
- this circular section lined with segments has been realized over a length of 6 m;
- finally, a last reinforced concrete section, with an outer diameter of 4 m, has been poured against the digging face, thus ending the first phase.

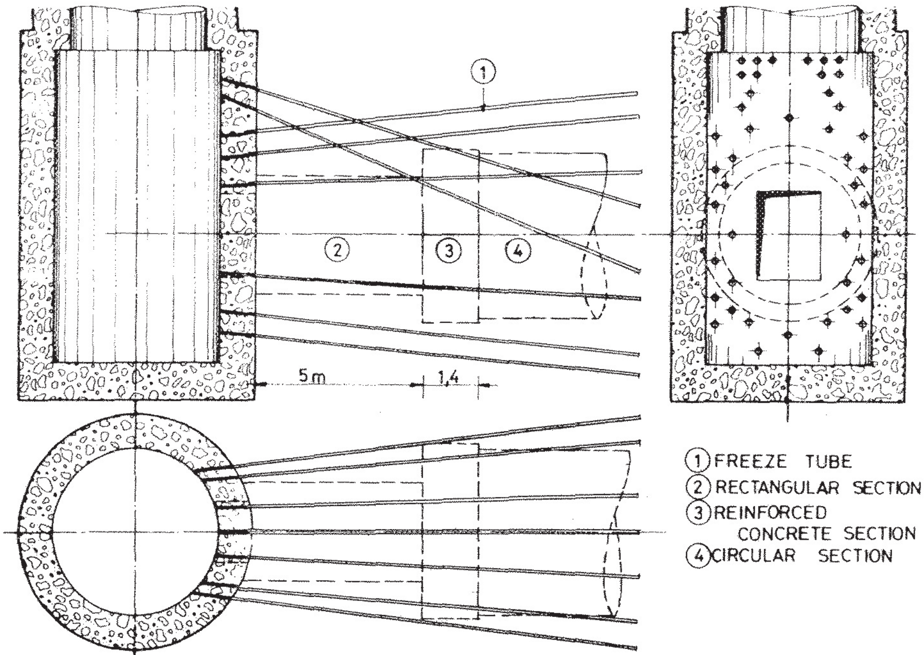


Fig. 16. — Layout of the freeze tubes for the main gallery of the in situ laboratory at Mol.

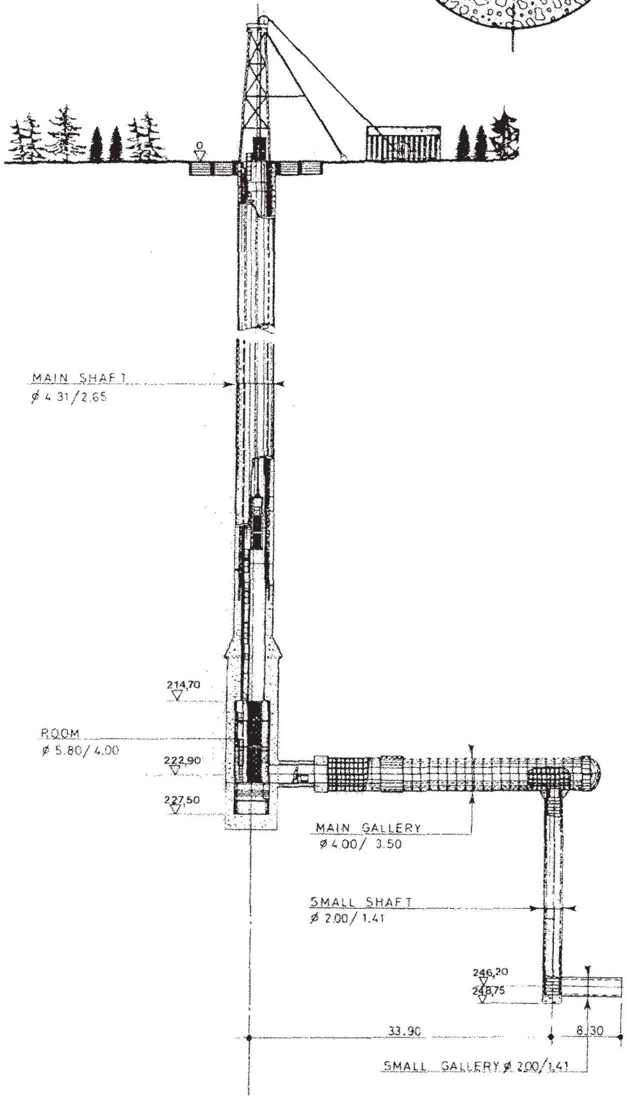


Fig. 17. — Laboratory in clay at Mol as built.

The second phase comprises (fig. 17):

- placement of freeze tubes having a length of 30 m through bore-holes drilled to this end in the last concrete section;
- after a freezing time of five weeks, digging and lining of the last twenty metres of the gallery in circular section with an inner diameter of 3.5 m and lined with cast-iron segments;
- a concrete end plug completes this second and final phase.

Taking into account the good agreement between the calculated and the observed convergences during execution of the first phase, only one single freezing crown has been adopted for the second phase, according to calculations.

Additional excavations, —a small shaft and a small gallery—, have been made from the end of the main horizontal gallery. These additional excavations, performed in unfrozen clay, have given information about the behaviour of the unfrozen clay at great depth.

2.5. Freezing of back filling material in old mine-shafts

Old mine-shafts, some of which are more than 100 years old, have been back filled with all kinds of products: gravel, pebbles, ashes, etc. generally leaving metallic shaft equipment in the shaft (fig. 18).

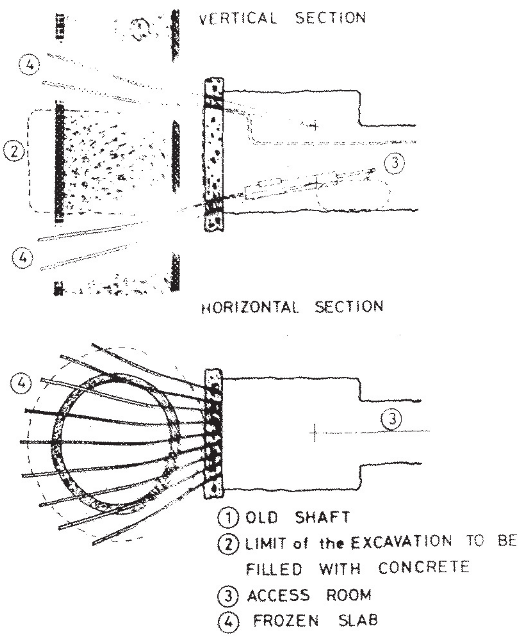


Fig. 18. — Shaft plug.

This not careful backfilling has caused several accidents due to a sudden settlement of these backfilling materials, provoking sometimes releases of fire-damp accumulated in the old exploitation sites.

In view of stabilizing this backfilling material, at least over the first 50 to 100 m, it is necessary to install a concrete plug in the shaft section at depths of the order of 50 to 100 m. Realization of such a work requires emptying of the shaft starting from the surface until the depth where the concrete plug has to be installed. This operation is extremely dangerous since the personnel working at the bottom is always exposed to sudden collapse of the filling material.

Laterally to the shaft to be treated and at a distance of about ten metres from this shaft, a small access shaft is sunk, either by boring or by digging. This shaft has an inner diameter of 1.2 m. At the depth where the plug has to be realized, a small working room, with a width of approximately 2.5 m, a height of 2.5 to 3 m and a length of 4 to 5 m is dug, starting from this small auxiliary shaft until as close as possible to the shaft to be treated.

From this room onwards, freezing of two slabs going through the shaft is realized. The first slab is situated above the plug, the second one underneath it. Both slabs of frozen soil allow digging through the shaft and completely safe installation of reinforced concrete.

Indeed, above the excavation to be realized, the personnel is protected from any fall of filling material which is present in the upper part. Underneath the excavation, the lower slab protects the personnel from any sudden collapse of the filling material which is present in the lower part of the shaft. This method has been successfully applied to more than twenty shafts during the last ten years for the realization of underground gas-storage in the old works and/or for stabilizing shafts in the immediate vicinity of underground civil engineering works: tunnels, subway...

3. DESIGN OF GROUND FREEZING JOBS

Since the end of the last century, contractors in mining engineering have used artificial ground freezing as an aid to safe execution of their jobs: shaft sinking and tunnelling in waterbearing grounds. The complexity of phenomena of frost propagation and mechanical behaviour of frozen earth masses made the scientific interpretation of the statements made during job execution difficult, without the presently available techniques and means of computation. Projects were based upon a few empirical formulae and experience acquired by the contractor during the execution of his work.

The use of artificial ground freezing in civil engineering works has increased in recent years. Excavations are performed in towns in the vicinity of or below buildings, bridges and other constructions. The contractor using ground freezing is not the only one who is concerned with an incident to it. The people that use the facilities near which excavations are made, the consulting engineers and the various insurers are also interested in a good execution of the operations. A consequence of this is that it is at the present time more necessary than before to be able to justify the stability of the excavations made in frozen soil, as for any other work in civil engineering.

Empirical formulae and experience, —although always essential—, are not sufficient because every job in civil engineering differs from the previous and because the empirical formulae have a validity of application only for shafts and circular tunnels.

The method used to check the stability of excavations made in frozen ground is composed of two steps:

– a time transient thermal analysis that gives as functions of time a forecasting of the temperatures in the ground and of the refrigerating power required;

- an evaluation, —also time dependent—, of deformations and stresses in the frozen earth mass.

3.1. Thermal analysis

For thermal analysis we chose to use the following equation for heat transfer:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial T}{\partial z}\right) = c\frac{\partial T}{\partial t} \qquad (3.1)$$

where:

x, y and z are the co-ordinates,

k_x, k_y and k_z the thermal conductivities of the soil in the different directions,

c the specific heat of the ground,

T the temperature,

t the time.

The thermal conductivities and specific heat are functions of the nature of the ground, its temperature and its water content.

Equation (3.1) can be transformed into equation (3.2) in order to be solved by the finite element method:

$$[C]\times[\dot{\theta}]+[K]\times[\theta] = [Q] \qquad (3.2)$$

where:

$[\theta]$ is the vector of the temperatures at every nodal point at time t,

$[\dot{\theta}]$ the derivative of $[\theta]$ with respect to time,

$[Q]$ the vector of heat flux at every nodal point at time t,

$[C]$ the specific heat matrix,

$[K]$ the matrix of thermal conductivities.

The numerical techniques for the evaluation of the matrices $[C]$ and $[K]$ and for the resolution of equation (3.2) do not cause any large problems. Nevertheless, it is necessary to be sure that the time intervals Δt used in order to solve equation (3.2) are small enough. The difference between the computed temperatures at time t and the computed temperatures at time $t+\Delta t$ must remain below a maximum value, otherwise artificial numerical oscillations appear in the computed temperatures and heat fluxes. This leads to a rapid divergence of the computational process.

The latent heat of fusion of water can be simulated by higher specific heats between for example -0.5 °C and $+0.5$ °C. Solving (3.2) may require slicing of the freezing time into 4000 or 5000 time intervals. If for a mesh containing some hundreds of nodes we consider, for example 15 seconds for the evaluation and assembly of matrices $[C]$ and $[K]$ and the solution of equation (3.2) at every time step, then the *CPU* time needed for a thermal analysis is evaluated by tens of hours. Cost of such a study by a service bureau may vary from 1 000 US $ to 10 000 US $.

But the sensitive aspect is the evaluation of the thermal parameters to be used. Of course they can be evaluated in a laboratory but:

- Equation (3.1) takes into account only part of the thermal phenomena involved. For example, during the freezing process, water migrates from warm zones to cold zones. The migration itself causes a variation of the conductivities and of the specific heats;
- Equation (3.1) is transformed into (3.2) and is solved using some given computational technique and some given mesh;
- The raw data coming from laboratory tests must be transformed into conductivity and specific heat values by using some adequate formulae. It is possible that these values do not correspond to the values that could have been obtained by interpreting data from the laboratory;
- Generally the soil is more heterogeneous at the scale of the samples taken for the laboratory tests than in its overall mass.

The method used is as follows:

- Choice of a general law as equation (3.1) that can reproduce thermal phenomena;
- Choice of a computational technique that allows the construction and solution of equation (3.2);
- Performing an in situ test. This test may consist of refrigerating the soil by a single freeze tube. Temperatures in the soil are monitored by thermistors or thermocouples installed in holes drilled in the vicinity of the freeze tube. This test also allows for the discovery of any water movement in the ground, shown by a non-axisymmetric temperature regime around the tube. This is shown in figure 19;

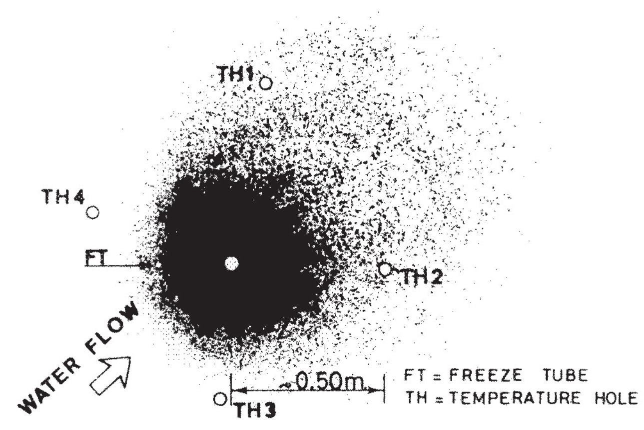


Fig. 19. — In situ frost propagation test.

— Building of a finite element model representing the single freeze tube and the surrounding soil;

— Simulation of frost propagation by introducing into this model various values of thermal parameters. The right values of these parameters, — and this is an important point —, are the ones that give, with the chosen law, equation (2.1) for example, and with the chosen computational method, the reproduction of the measurements made in the field;

— Building a finite element mesh representing the freeze tubes and the ground of the real future work;

— Simulation of frost propagation using this model and the values of the thermal parameters evaluated by the preliminary test above. Figure 20 shows an intermediary step of such a simulation made for the job described for the Louise subway station (see paragraph 2.2).

Temperatures are monitored during the progress of the job in order to check that ground freezing is taking place correctly.

Ground freezing contractors own very accurate data about the closure time of the frozen enclosures that must be watertight. This has permitted us to check that the error of the computed time at which a given temperature distribution is actually obtained in the site can be less than 10%. It could be thought that this is very small in comparison with, for example, the error of 50% which is normally made in the evaluation of the forces in a classical civil engineering structure. Nevertheless, this accuracy is necessary because the cost of a ground freezing job is much more sensitive to estimated freezing time than the cost of a

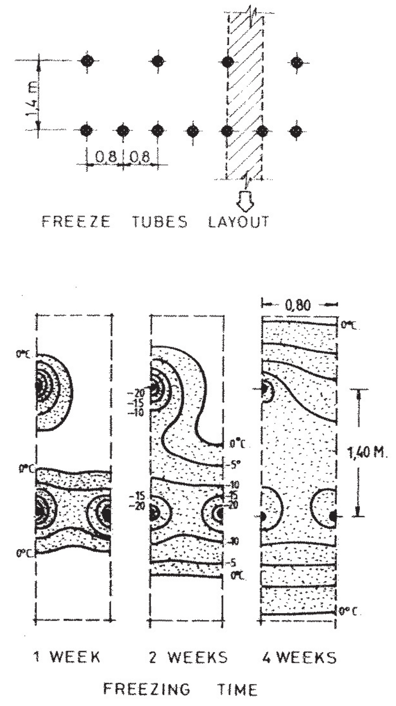


Fig. 20. — Frost propagation at the Louise subway station in Brussels.

civil engineering structure is sensitive to the evaluation of the structural forces.

3.2. Stability analysis

The prediction of the evolution with time of the stresses and deformations is more delicate. Frozen ground, — like any other materials —, deforms with time under constant loading.

Usually time is neglected in civil engineering applications. This can not be done for the study of the behaviour of frozen earth masses. The time influence cannot be neglected when selecting the characteristics of the tunnel lining. Indeed, the evolution of the forces acting on the lining depends, amongst other things, very closely on the rheological properties of the rock, the time given to the ground in order to expand before the lining is set and the lining flexibility. A stability analysis that does not take into account time and method of execution leads to erroneous results. The computational techniques used to check the stability of the frozen earth structures can be directly used for the study of the excavation and lining in rock.

Rheological law

The first step is to choose a rheological law for the material to be investigated, for example (Fish, 1980):

$$\dot{\varepsilon}(t) = \frac{C}{t_f} \times \left(\frac{\sigma}{\sigma_c}\right)^n \times \left(\frac{t_f}{t}\right)^\delta \times e^{\frac{\delta t}{t_f}} \tag{3.3}$$

$$\text{with: } t_f = t_0 \times \left(\frac{\sigma}{\sigma_c}\right)^{-m} \tag{3.4}$$

$$t_0 = \frac{h}{kT} \times e^{\frac{E}{RT}} \tag{3.5}$$

where:

$\dot{\varepsilon}$ is the strain rate,

t the time,

σ the stress,

σ_c the instantaneous strength when the ground is loaded at a strain rate constant with time, 1 % by minute for example,

T the temperature in degrees Kelvin,

t_0 Frenkel's relaxation time,

h Planck's constant,

k Boltzmann's constant,

E the activation energy of the ground,

R the gas constant,

C, m, n and δ constants varying with temperature and nature of the ground.

Determination of the values of the rheological parameters

The cost of an in situ test before the work is so high that such a test is reserved for specially important jobs.

Moreover, in the case of frozen soils, it is very difficult to maintain the temperature at a given value and thus to discern the influence of the temperature on the experimental values. It is so generally necessary to make up one's mind to perform laboratory tests. The job itself will serve as in situ test that can be used in the future.

Laboratory tests are performed at various temperatures in a cold space. Two kinds of tests may be performed: the determination of the instantaneous compressive strength and the determination of the creep parameters, both in uniaxial or triaxial conditions.

Two laboratories are equipped in Belgium for the working out of these tests.

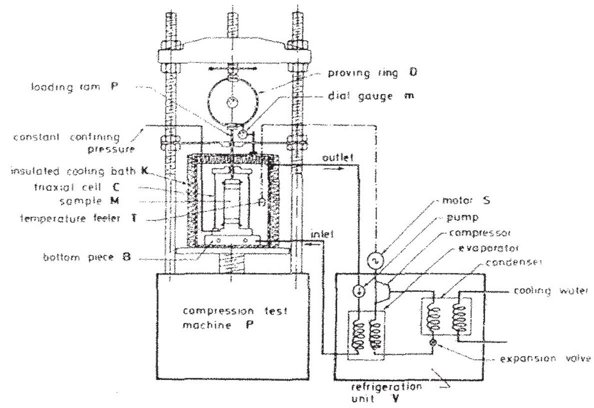


Fig. 21. — Equipment at the Belgian Geotechnical Institute.

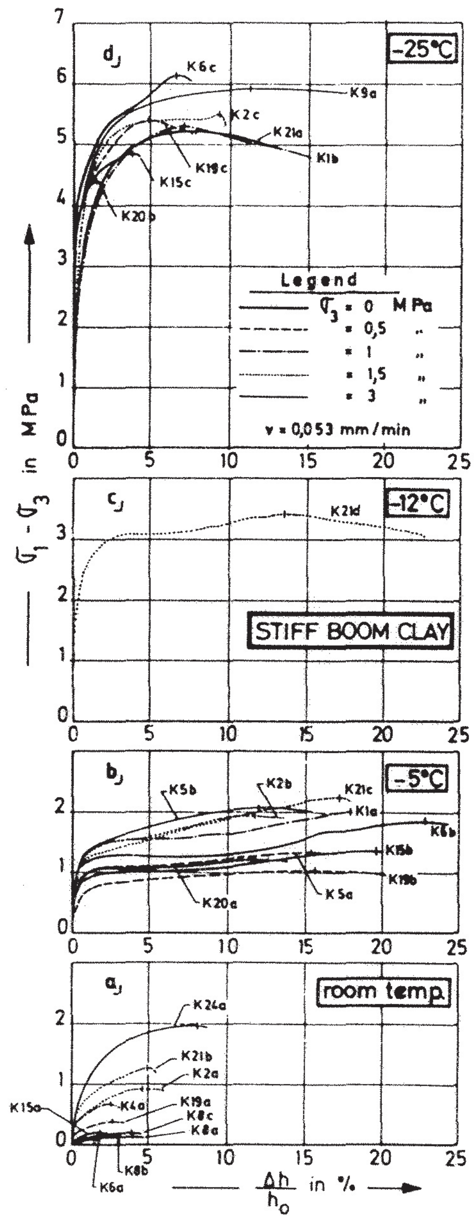


Fig. 22. — Triaxial compression tests on frozen and unfrozen stiff clay.

The Belgian Geotechnical Institute disposes of a triaxial cell for the determination of the compression strength of frozen samples.

The triaxial cell is immersed in a cooling bath. As a coolant an equal part mixture of ethylene glycol and water is used. A scheme of the test equipment is given in figure 21. The coolant is cooled by a smal freezing group *V* and is circulated in the cooling vessel *K* through a special bottom piece *B* serving as a pedestal for the triaxial cell. The temperature of the coolant is controlled by a temperature feeler *T* suspended in the cooling vessel and steering the motor *S* of the freezing group.

Cylindrical soil samples with a diameter of 38 mm or 100 mm can be tested under cell pressures up to 7 MPa and temperatures down to −25 °C.

As an example the results of triaxial tests on frozen and unfrozen stiff Boom clay samples are represented in figures 22 and 23 (Carpentier, 1982).

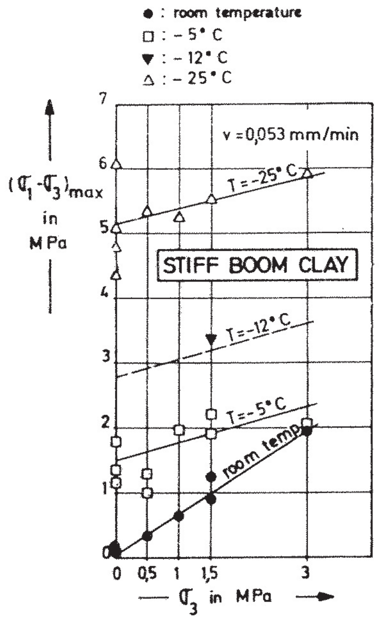


Fig. 23. — Results of triaxial compression tests on frozen and unfrozen stiff clay.

The Laboratoire du Génie Civil of the Université Catholique de Louvain disposes, with Foraky S.A., of a cold room (3 m × 2.8 m) in which the temperature can be regulated until −20 °C. In this room one finds a hydraulic jack and 13 creep units. In table 1, one finds a description of the tests that can be performed.

TABLE 1. — Cold room of L.G.C.-U.C.L./Foraky s.a.

APPARATUS		HYDRAULIC JACK	CREEP UNIT
NUMBERS		1	13
TEMPERATURE		-5°C à -40°C	-5°C à -20°C
TESTS	Unconfined compression	Strength and creep	Creep
	Triaxial compression	Strength and creep	Creep
	Flexion	Strength and creep	-
FORCES		P_1 = 100 kN	P_1 = 14 kN
STRESSES		(σ_1 = 90 MPa) * σ_3 = 10 MPa	(σ_1 = 13 MPa)* σ_3 = 10 MPa
		* with a cylindric sample of 38 mm diameter	

Figure 24 gives a scheme of the hydraulic jack which is an adapted triaxial equipment. Temperature of −40 °C can be reached. Figure 25 gives a scheme of the creep unit which is an adapted oedometer. The tests are monitored by a computer and the parameters such as temperatures, stresses, displacements are automatically regulated and recorded (Lousberg and al., 1984), (Gonze, Lousberg and al, 1985).

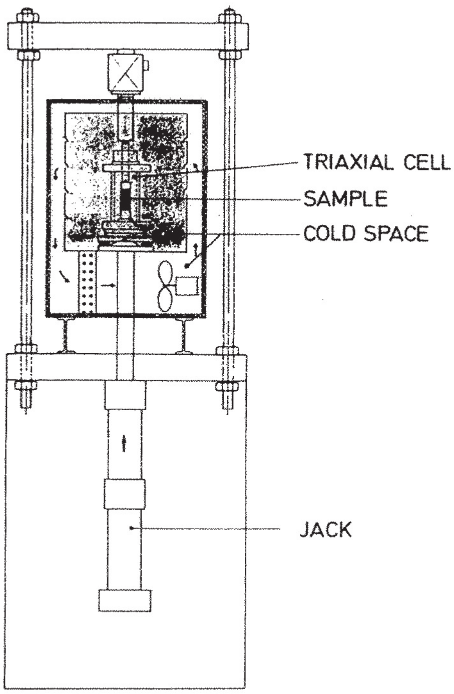


Fig. 24. — Hydraulic jack at the Laboratoire du Génie Civil (UCL/Foraky).

Figures 26 to 28 give some results of tests performed on frozen ground.

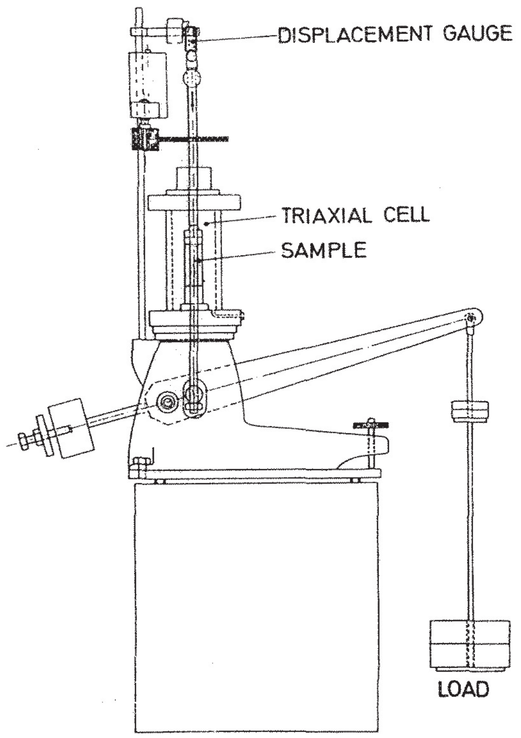


Fig. 25. — Creep unit at the Laboratoire du Génie Civil (UCL/Foraky).

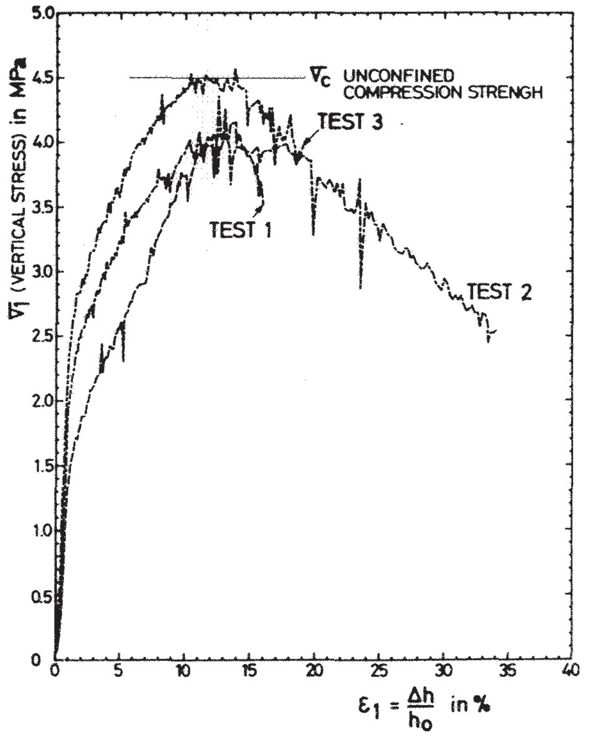


Fig. 26. — Unconfined compression tests at −10 °C on a clayey sand.

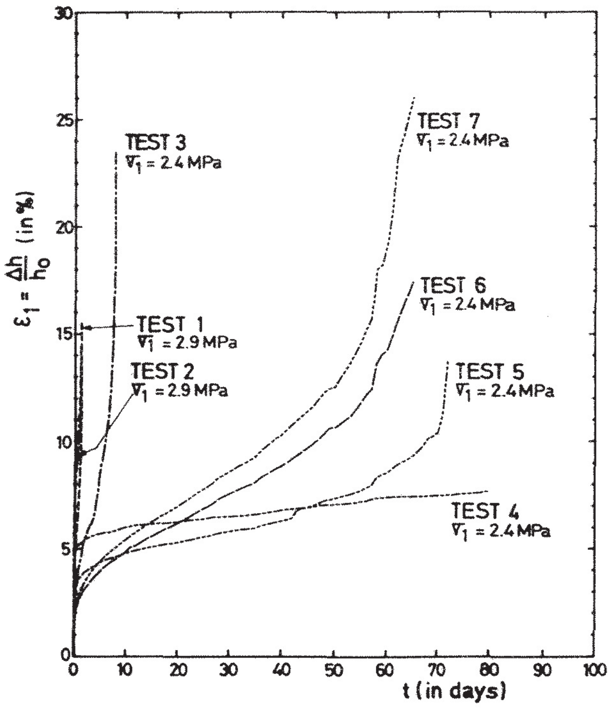


Fig. 27. — Uniaxial creep tests at −10 °C on a clayey sand.

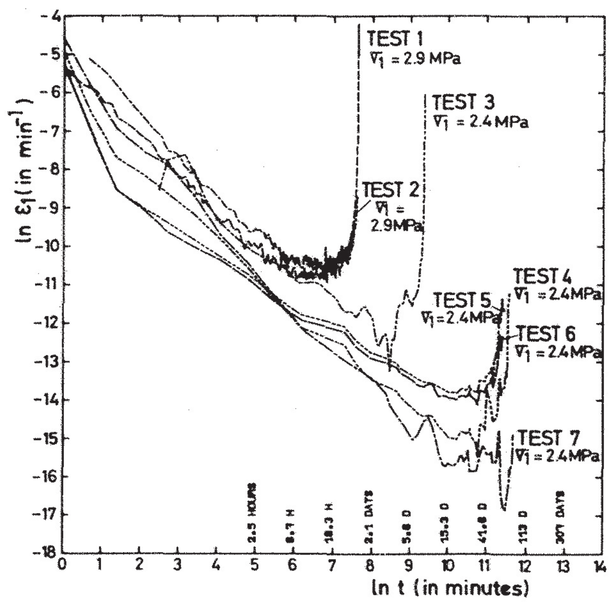


Fig. 28. — Uniaxial creep tests at −10 °C on a clayey sand.

For each temperature a series of tests is performed at fixed strain rate $\dot{\varepsilon}$. Figure 26 shows curves for different tests related to instantaneous compression strength σ_c for a clayey sand frozen at −10 °C. This strength is of the order of 4 MPa. After that and also for every temperature, creep tests are conducted at stress levels, — minimum 2 —, below σ_c. This is shown on figure 27 for $-10\,^\circ C$. The creep curves (ε, t) of this figure can be transformed into $(\dot{\varepsilon}, t)$ curves as shown on figure 28 where logarithmic scales are used.

This makes it possible to evaluate the values of the rheological parameters to be used in equation (3.3). By convention, the time t_f of equation (3.3) that corresponds to the beginning of increasing strain rate, is the failure time of the material for the stress level σ.

The security against rupture, for every temperature and for every stress level, can be defined as the time t_f divided by the time of use of the structure of frozen soil.

Figure 29 shows the relation between stress and time of failure for a given sand at various temperatures.

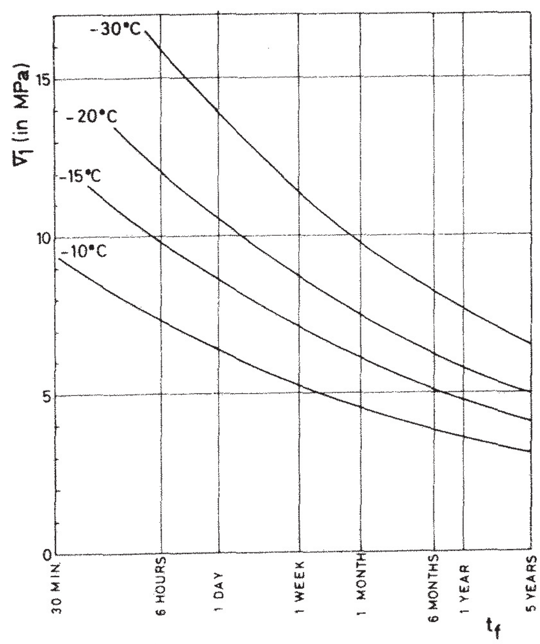


Fig. 29. — Strength versus time for a frozen sand.

Numerical analysis

The rheological parameters thus evaluated, can be used in a finite element model (Gonze and Rossion), (Klein, 1979). Figure 30 shows such a model relating to the tunnel in frozen clay at a depth of 220 m and described in paragraph 2.4.

Figure 31 gives the computed convergence of the face. During execution of the work, deformations of the earth mass are monitored. Then one tries to

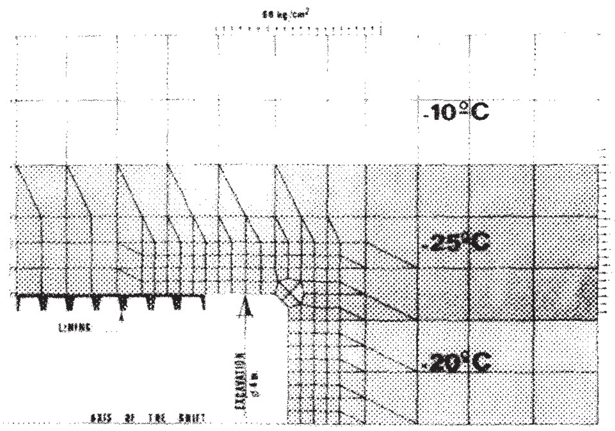


Fig. 30. — Model for creep analysis around the main gallery in the in situ laboratory in clay at Mol.

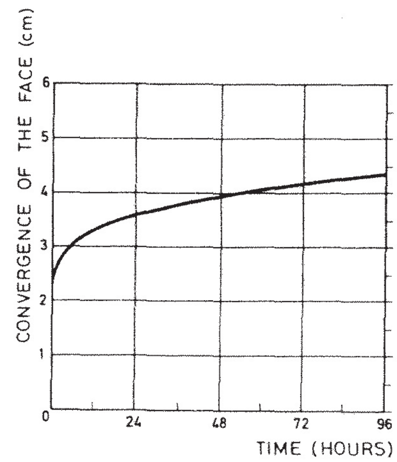


Fig. 31. — Computed convergence of the face of the main gallery in the laboratory at Mol.

reproduce by computation these deformations by assuming various values of the rheological parameters in law (3.3). This allows for a better evaluation of the parameters than with the laboratory tests. These values will then be available for other projects in similar conditions and will also give an evaluation of the modifications that are to be made to the values given by the laboratory tests.

4. ACKNOWLEDGMENTS

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R.I.G., Belgian Geotechnical Institute, Brussels.

Control Organization:

SECO, control office for safety of the construction, Brussels.

Ground Freezing Contractor:

FORAKY s.a., Brussels.

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RÉSUMÉ:

LA TECHNIQUE DE LA CONGELATION EN BELGIQUE

La technique de la congélation est utilisée en Belgique depuis plus de cent ans notamment pour la réalisation des puits de mines. Jusqu'il y a deux décennies, la congélation n'était que fort peu utilisée en génie civil et était souvent considérée comme le procédé de la dernière chance. Depuis, cette technique s'est remarquablement développée dans les sites urbains. En même temps des perfectionnements étaient apportés à la conception des centrales de réfrigération ainsi qu'aux techniques de forage et de mise en place des congélateurs. Les problèmes posés étaient différents de ceux rencontrés lors du fonçage des puits de mines, à savoir la congélation de terrains non saturés, la rencontre d'anciennes fondations, la complexité des formes des volumes à congeler, la nécessité de laisser les parois congelées longtemps non revêtues, etc...

Après ces considérations, l'article décrit un certain nombre d'exemples d'ouvrages exécutés grâce à l'utilisation de la congélation, à savoir:

- *des chantiers du métro d'Anvers (un essai de congélation in situ; le tunnel Van Wesenbeke; le tunnel Opéra-Astrid);*
- *des chantiers du métro de Bruxelles (la station métro sous la gare du Midi; le passage sous les bâtiments de la chocolaterie « Côte d'Or »; la station métro Louise; le tunnel sous la Senne, rivière canalisée);*
- *le chantier d'un nouvel immeuble de la Caisse Générale d'Epargne et de Retraite à Bruxelles;*
- *le puits d'accès d'une profondeur de 215 m donnant accès à un laboratoire souterrain situé dans l'argile de Boom à Mol;*
- *la consolidation du remplissage d'anciens puits de mines.*

Enfin l'article décrit les progrès qui ont été réalisés pour le calcul de la propagation du froid dans le sol et de la tenue des parois congelées en fonction du temps. Ces progrès ont permis d'affiner les projets et de réaliser des économies et des gains de temps appréciables.

D'une part l'analyse thermique de la propagation du froid a permis de mieux évaluer la progression du froid dans le temps et l'énergie requise à cet effet. La méthode des éléments finis est utilisée à cette fin. On arrive, grâce au calibrage des calculs par des observations in situ, à d'excellentes prévisions.

D'autre part des programmes ont également été mis au point pour calculer l'évolution des contraintes et des déformations selon le temps dans les massifs gelés. Les paramètres mécaniques (résistance à la compression) et rhéologiques (fluage) sont déterminés in situ et dans les laboratoires spécialisés en mécanique des sols gelés. Ici également la comparaison avec des observations in situ a permis d'affiner les prévisions.

SAMENVATTING:

BEVRIEZINGSTECHNIEK IN BELGIË

De bevriezingstechniek wordt in België reeds meer dan 100 jaar toegepast, namelijk bij het realiseren van mijnschachten. Tot voor twee decennia werd bevriezing slechts weinig toegepast in de burgerlijke bouwkunde en werd deze methode dikwijls beschouwd als laatste redmiddel. Sindsdien heeft deze techniek zich op merkwaardige wijze ontwikkeld voor toepassing in de steden. Te gerlijker tijd werd verbetering aangebracht aan de opvatting van de koelinstallaties alsook aan de boortechnieken en de plaatsing van de vriespijpen. De voorkomende problemen verschilden van deze ondervonden tijdens het graven van mijnschachten, te weten de bevriezing van niet verzadigde terreinen, het aanstoten van oude funderingen, de ingewikkeldheid van de vormen van de te bevriezen volumes, de noodzaak om bevroren wanden niet landgurig onbeschermd te laten.

Na deze beschouwingen beschrijft het artikel een zeker aantal voorbeelden van werken, uitgevoerd dank zij het gebruik van bevriezing, te weten:

- *werven van de Pre Metro te Antwerpen (een bevriezingsproef in situ; de tunnel van Wesembeek; de tunnel Opera-Astrid);*
- *werven van de Metro te Brussel (het metro-station onder het Zuidstation; de doorgang onder de gebouwen van de chocoladefabriek „Côte d'Or"; het Metro-station Louise; de tunnel onder de gekanaliseerde rivier de Zenne);*
- *werf van de nieuwe gebouwen van de Algemene Spaar- en Lijfrentekas te Brussel;*
- *de toegangsschacht met een diepte van 215 m, welke toegang geeft tot een ondergronds laboratorium in de Boomse klei te Mol;*
- *de consolidatie van de opvulling van oude mijnschachten.*

Verder beschrijft het artikel de vooruitgang welke is verwezenlijkt in de berekening van de uitbreiding van de koude in de grond en van het gedrag van de bevroren wand in funktie van de tijd. Deze vooruitgang laat toe de projekten te verfijnen en hierdoor winst te maken zowel financiëel als op het gebied van uitvoeringstijd.

Enerzijds heeft de thermische analyse van de voortschrijding van de koude toegelaten beter de uitbreiding van de koude in funktie van de tijd alsook de hiervoor vereiste energie te evalueren. De eindige elementen methode wordt hiervoor gebruikt. Men komt, dank zij de afstemming van de berekeningen op de waarnemingen in situ, tot uitstekende voorspellingen.

Anderzijds werden eveneens programma's op punt gesteld voor de berekening van de veranderingen van de spanningen en vervormingen in de tijd in bevroren massieven. De mechanische en rheologische parameters (drukweerstand en kruip) worden in situ en in laboratoria voor grondmechanica bepaald.

Ook hier heeft de vergelijking met waarnemingen in situ toegelaten de voorspellingen te verfijnen.