The extension of the HADES underground research facility at Mol, Belgium

The construction of the connecting gallery

W. Bastiaens, F. Bernier, M. Buyens, M. Demarche,
X.L. Li, J.-M. Linotte, J. Verstricht

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Foreword and summary

Geological disposal—a potential solution for the long-term management of high-level and/or long-lived radioactive waste—has been the object of uninterrupted investigations in Belgium since the mid-seventies. The research programme launched at the time in that framework by the Belgian Nuclear Research Centre (SCK•CEN, Mol, north-eastern Belgium) led it to decide, at a very early stage, to focus its efforts on studying the Boom Clay layer present under its site. To this end, SCK•CEN needed in particular to assess the feasibility of constructing underground facilities at a depth of some 200 metres in this type of clay, in other words, in a poorly-indurated clay. The successful construction of the HADES underground research laboratory in the early eighties showed that it was possible to construct shafts in frozen aquifer sands and to construct the facilities needed for disposal in Boom Clay without having to freeze the clay massif. The research and development programme on geological disposal was, and still is, a methodological programme. Hence, it does not prejudge the site where a deep disposal solution would actually be implemented. A few years later, the construction of the Test Drift, which was the first major extension of HADES, provided an opportunity to compare several construction techniques. The feasibility of constructing galleries in clay using an industrial technique had not yet been demonstrated, though.

The promising results so far obtained, both in technical and in scientific terms, prompted SCK•CEN and ONDRAF/NIRAS, the Belgian Agency for Radioactive Waste and Fissile Materials, to devise an ambitious demonstration project: the PRACLAY project. This project was meant, in particular, to demonstrate the feasibility of constructing underground galleries with an industrial technique—since an industrial technique will be necessary for constructing a real geological repository—and to simulate the disposal of high-level heat-emitting waste in a geological repository in clay. The PRACLAY project required to extend the HADES facility, but compliance with the mining regulations imposed that the existing underground facility be provided with an additional access shaft first. The connecting gallery was going to link this second shaft to the Test Drift and would enable new experiments to be carried out, among which the PRACLAY experiment. The new shaft was excavated in 1997–1999, at 90 metres from the end of the Test Drift. It was the first time that a large-diameter shaft was built in the Boom Clay without freezing the clay layer.

After several years of studies and tendering, which resulted in the adjudication of the contract for the construction of the connecting gallery to the temporary association SCM Tunnel, EURIDICE—the economic interest group formed by ONDRAF/NIRAS and SCK•CEN—finally launched the construction works in 2001. For the first time ever, a gallery was going to be excavated in a poorly-indurated clay at a depth of 223 metres using an industrial technique. This was a challenge, an experiment in itself. It implied indeed to find an optimum between two contradictory requirements: on the one hand, minimising the disturbances to the clay massif, among others by keeping the overexcavation as low as possible and by reaching as high a construction rate as possible and, on the other hand, avoiding that the shield got trapped in the massif by convergence, knowing that the instantaneous convergence of the clay had never been measured accurately in the past. As construction technique, EURIDICE had elected to use a tun-

nelling machine equipped with a road header, in combination with the wedge-block technique for the lining. This technique is fast and minimises the convergence of the massif, since the lining is expanded against the massif.

Having completed the construction of the connecting gallery, EURIDICE has synthesised in the present document the preparation process, the actual construction experience, and the technical and scientific achievements.

The provisional delivery of the connecting gallery in June 2002 has been evidence of the success of the underground construction works, which met the two major challenges: the convergence was minimised while the shield did not get trapped in the clay at any time, and the construction rate was consistently above the target value of 2 metres per 24 hours. The design and the dimensions of the tunnelling machine and of the lining segments were thus adequate in respect to each other, this thanks to the accuracy of the numerical predictions of the displacements of the massif and of the pressures on the lining. Besides, the very thorough preparation of all the practical aspects of the works had enabled some potential problems to be detected in advance and enabled the problems encountered during the works and related to the design aspects to be solved rapidly.

Though the construction technique used has overall been successful, there has been one major, unexpected problem: the extent of the detachment of clay blocks from the front and from the unsupported sidewalls as a result of the presence of excavation-induced fractures. This has been both a safety issue and a construction issue. Using a full-face tunnelling machine instead of a road header would enable a uniform and simultaneous excavation of the front and could drastically reduce the detachment of clay blocks from it. If used in combination with "support fingers" at the upper rear of the shield, such machine could also largely prevent the fall of clay blocks at the back of the shield. The wedge-block technique remains a privileged option for future construction works in the Boom Clay. It could be improved through a few simple modifications.

The measurement and research programmes carried out before and during the construction works have led to a comprehensive characterisation of the fracturation pattern around the excavations and of the instantaneous hydromechanical response of the Boom Clay to an excavation using an industrial tunnelling technique.

The fracture characterisation programme resulted in a description, in terms of orientation and shape, of the fractures around the connecting gallery, and in a better understanding of how these fractures are formed. All the fractures observed at the Mol site appear to have been induced by excavation. They originated at some 6 metres ahead of the excavation front and have a parabolic shape. Coring after the completion of the gallery indicated that the fractures extend up to about 1 metre in the radial direction. An important remaining issue is the impact fractures can have on the long-term performance of geological repositories. This impact will probably be limited by the healing and sealing mechanisms that have already been identified qualitatively in various ways. For instance, although fractures induced by the Test Drift 15 years ago were encountered at about 6 metres before the Test Drift front, signs of oxidation were only seen in the last metre of clay. This observation suggests that the fractures between 6 metres and 1 me-

tre before the front had been sealed and were reactivated by the excavation of the connecting gallery.

The CLIPEX programme has revealed that the behaviour of the Boom Clay massif is characterised by a strong hydromechanical coupling, already noticeable at an unexpectedly large distance from the excavation, and by a clear time dependency. It furthermore confirmed the blind predictions of the displacements and pressures on the lining, the modelling of the evolution of the pore water pressure deep inside the clay massif being still an unresolved issue.

The most significant results of the various types of measurements performed (pore water pressure, displacements, total pressure, pressure on the lining) are the following.

- The measurement of the pore water pressure through piezometers has proved to be a very reliable, accurate, and mature technique. For the first time, the pore water pressure has been measured outside of the zone of influence of the HADES facility. The value measured at 223 metres depth was confirmed to be equivalent to the initial undisturbed pore water pressure, namely about 2.2 MPa. Furthermore, the measurements of the pore water pressure clearly reflected the influence of the fracturation, of the decompression of the massif, and even, in the vicinity of the excavation front, of the excavation phases (alternation excavation/lining). Finally, measurements have revealed that the hydraulically disturbed zone extends up to more than 60 metres from the excavation front, which is important for the proper understanding of the hydromechanical behaviour of the Boom Clay.
- The pressure on the lining increased very rapidly in the first few days after construction, as a result of the reconsolidation of the Boom Clay caused by the drainage of the pore water and the creep of the massif. This too indicates that the hydromechanical disturbances have been minimised. The pressure has been stabilising progressively since.

The total radial convergence of the Boom Clay was about 9 cm on the excavated radius (2.5 metres), which is considered acceptable in terms of hydromechanical disturbances. This total radial convergence is the sum of the measured instantaneous convergence of the Boom Clay, which was about 45 mm on the radius, and the radial convergence ahead of the excavation front, which was also about 45 mm on the radius according to the modelling results and as confirmed by the displacement sensors.

Finally, the petrographic study has shown that the Boom Clay only oxidises on the fracture walls: the only evidence of pyrite oxidation, under the form of newly formed minerals, is indeed on the fracture planes, microfractures, and discontinuities. Such oxidation effects are the visual print of the geomechanical disturbances induced by excavation. Assessing them is important in the framework of the migration studies and for performance assessments.

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1. Introduction

For more than 20 years now, Belgium has been actively studying the long-term management of high-level and/or long-lived radioactive waste. A research programme has indeed been launched by the Belgian Nuclear Research Centre SCK•CEN at Mol, in the north-east of Belgium, as early as in the seventies. This programme was directed to the solution recommended at an international level for isolating such waste from humans and the environment, namely, to dispose of it in a stable geological formation with appropriate characteristics. Quickly, SCK•CEN chose to concentrate its efforts on investigating the Boom Clay layer beneath its own site as a potential host formation. Because of the lack of experience, both at the national and at the international levels, in the excavation of underground facilities at a depth of some 200 metres in this type of clay—a poorly-indurated clay—, one of the main objectives of the SCK•CEN initial research and development programme has been to assess and demonstrate the feasibility of such works. This is why the HADES (High-Activity Disposal Experimental Site) underground research laboratory was constructed at a very early stage in the Belgian programme (Figure 1).

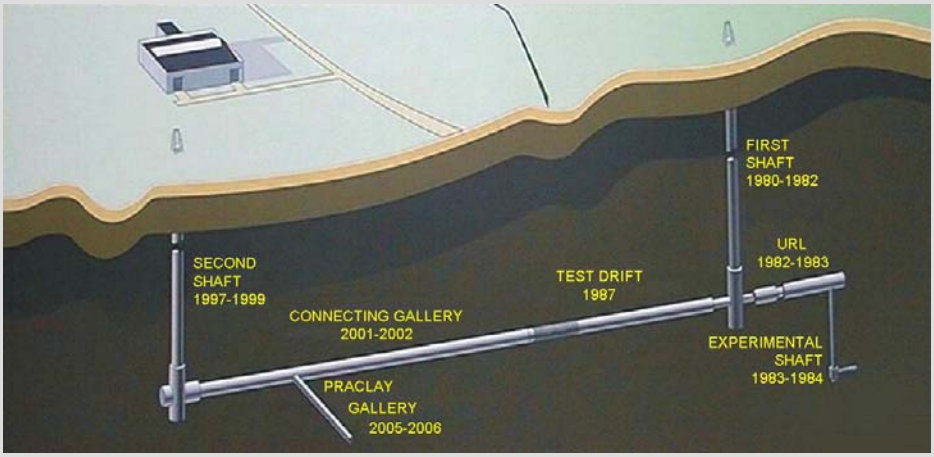


Figure 1: Construction history of the underground research facility HADES. The 80-metre-long connecting gallery connects the recently built second shaft to the already existing facility. The construction of the PRACLAY gallery is foreseen for 2005–2006.

In line with its mission of long-term management of radioactive waste, ONDRAF/NIRAS, the Belgian Agency for Radioactive Waste and Fissile Materials, took over the SCK•CEN project in 1985. The promising results already obtained at the time by SCK•CEN made ONDRAF/NIRAS decide to pursue the research and development work in geological disposal that was being carried out on the Boom Clay beneath the Mol site. The ONDRAF/NIRAS work programme was, and still is, a programme of methodological research and development. Hence, it does not prejudge the site where a deep disposal solution would actually be implemented. In other words, the Boom Clay and the Mol–Dessel nuclear zone are considered as, respectively, a reference

host formation and a reference site. (The Ypresian Clays and the Doel nuclear zone are being investigated as alternative host formation and alternative site since the late nineties.)

From the start, an important aspect of the Belgian geological disposal programme has been the assessment of construction techniques. Geological disposal in clay implies indeed to build specific underground facilities, consisting typically of access shafts, main galleries designed for transporting the waste, and disposal galleries, designed for receiving the waste. Building such repository, with kilometres of galleries, is clearly going to require an industrial technique, combining excavation and lining. The natural convergence of the Boom Clay imposes indeed to use a continuous lining, meant to limit the disturbances in the clay while withstanding its pressure.

The construction works carried out up until 1987 showed that it was possible to construct shafts in frozen aquifer sands and to construct the facilities needed for disposal in Boom Clay without freezing the clay massif. Also in the late eighties, an expert assessment confirmed the ONDRAF/NIRAS conclusions, namely that the poorly-indurated clays, and in particular the Boom Clay under the Mol site, could be considered for the disposal of high-level and/or long-lived waste, since they are able to offer effective protection in the very long term. This clay had indeed been found to have a very low hydraulic conductivity, a plastic character that gives it good self-healing properties, and a high capacity to fix radionuclides and, hence, to delay their migration towards the aquifers and the biosphere. The feasibility of constructing galleries using an industrial technique had not yet been demonstrated, though.

The encouraging results obtained up until the late eighties prompted ONDRAF/NIRAS and SCK•CEN to devise an ambitious demonstration project: the PRACLAY project, primarily aimed to contribute to the demonstration of the feasibility of disposing of heat-emitting high-level waste in geological clay layers. More specifically, this project aimed at the following:

- demonstrating the feasibility, from the technical and economic points of view, of constructing underground galleries using industrial techniques that could be used for constructing a real geological repository;
- demonstrating the feasibility of constructing the intersection between main galleries and disposal galleries, including the use of a reinforcement ring;
- carrying out the PRACLAY experiment, namely simulating the disposal of heat-emitting high-level waste in a geological repository in clay. This was to be done by installing electrical heaters in a 30-metre-long pilot gallery perpendicular to a main gallery and similar in every respect, except the length, to the disposal galleries considered in the reference architecture of the repository.

The PRACLAY project was to be managed by EIG EURIDICE (EURIDICE for short), the economic interest group created to that end in 1995—though under the name "EIG PRACLAY"—and grouping ONDRAF/NIRAS and SCK•CEN. Besides managing the PRACLAY project, EURIDICE is also responsible for managing and exploiting the whole underground research facility, for opening it to international collaborations, and for communicating about its own activities. (The various objectives of the PRACLAY project, and more specifically of the PRACLAY experiment, have been slightly modified in the recent years. This aspect is not treated in the present document.)

Compliance with the mining regulations imposed however that the existing underground facility be provided with an additional access shaft before work began on the excavation of the gallery intended to demonstrate the feasibility of using an industrial construction technique and to enable new experiments to be carried out, more specifically the PRACLAY experiment. This new shaft was excavated in 1997–1999, at 90 metres from the end of the Test Drift, using a jack-hammer mounted on a hydraulic arm and hand-operated pneumatic drills. It was the first time that a large-diameter shaft was built in the Boom Clay without freezing the clay layer. The new shaft was to be connected to the Test Drift by the gallery intended for the new experiments, the so-called "connecting gallery", financed by both Members of EURIDICE, SCK•CEN and ONDRAF/NIRAS.

After several years of studies and tendering, which resulted in the adjudication of the contract for the construction of the connecting gallery to the temporary association SCM Tunnel (a joint venture between SMET-Tunnelling, Wayss & Freytag, and Deilmann-Haniel), EURIDICE finally launched the construction works in 2001. For the first time ever, a gallery was going to be constructed in a poorly-indurated clay at a depth of 223 metres using an industrial technique. This enterprise, one of the milestones of the PRACLAY demonstration project, was a challenge, an experiment in itself. It implied indeed to find an optimum between two contradictory requirements: on the one hand, minimising the disturbances to the clay formation, among others by keeping the overexcavation as low as possible and by reaching a minimum target construction rate and, on the other hand, avoiding that the shield got trapped in the formation because of convergence, though such (immediate) convergence had never been measured accurately in the past.

Having completed the construction of the connecting gallery, EURIDICE has synthesised, for traceability purposes, the preparation process, the actual construction experience, and the technical and scientific achievements.

The present synthesis is structured in the chronological order of the works.

Chapter 2 describes briefly the history of the preparatory works (studies and tendering) that preceded the actual excavation works, and summarises their conclusions.

Chapter 3 provides an overview of the general organisation of the works between EURIDICE and its main contractors, and of the planning of the works.

Chapter 4 describes the preparatory works done before the actual start of the excavation works, and, in particular, presents the results of the auscultation programme of the Boom Clay massif.

Chapter 5 describes the construction works in their chronological order, namely the construction of the mounting chamber, the construction of the connecting gallery, and the connection to the Test Drift.

Chapter 6 presents the results of the measurement and research programmes carried out during and after the construction works and, in particular, those of the European CLIPEX programme.

Chapters 7 and 8 cover respectively the safety and the communication aspects.

Chapter 9 is a little of an outsider: it briefly gives the results of the preliminary studies concerning the possibilities to reinforce the primary lining of the connecting gallery to enable the drilling of boreholes of a certain diameter and the construction of the PRACLAY gallery.

Chapter 10 summarises the main conclusions, evaluates the achievements, and provides recommendations for future similar underground construction works.

Annex 1, finally, provides the reader with a list of additional reading.

2. Preparatory studies, including the calls for tenders

The preparation of the construction of the connecting gallery is based on the NIROND 92-03 report of March 1992 (A. Van Cotthem, *L'expérience de démonstration PRACLAY — Modalités de réalisation des excavations et soutènements*, ONDRAF/NIRAS), which led to the first selection of the technique to be used for constructing underground galleries in the Boom Clay at a depth of about 225 metres (Section 2.1). This preparation was then followed by tendering procedures, which led to adjudicating the contract for the construction of the connecting gallery in early 2001 to the temporary association SCM Tunnel (Section 2.2).

2.1. NIROND 92-03 report

The NIROND 92-03 report has been used by EURIDICE as *starting point* for the studies related to the construction of the connecting gallery. Although this report focuses mainly on the study related both to the excavation and lining of underground galleries and to their crossings, with a view to the PRACLAY demonstration project, this study could indeed be extrapolated to the excavation and lining of the connecting gallery. One of the basic requirements mentioned in the report is that the technique(s) to be selected for constructing underground galleries had

- *to minimise the convergence of the clay massif;*
- *to be reproducible;*
- *to be well-tried*.

According to the NIROND 92-03 report, the extent of the excavation-disturbed zone and the convergence of the clay massif, both during and after the construction of a gallery, are influenced by the construction rate (excavation *and* lining rate), the applied overexcavation, the length of the zone that is not supported during excavation, and the mechanical characteristics of the lining. More specifically, the report concluded that the convergence during and after construction of the gallery could be minimised through

- *maximising the construction rate;*
- *minimising the overexcavation;*
- *minimising the length of the unsupported zone;*
- *chosing a lining that was both stiff and very strong (concrete, but non-reinforced in order to avoid corrosion of the reinforcement and, hence, the production of hydrogen)*.

The NIROND 92-03 report also concluded that the best way to line galleries at a depth of approximately 225 metres in the Boom Clay while minimising convergence is to use the so-called "wedge-block technique" (an English technique used for constructing the London underground) at the back of a shield. Indeed, after studying three well-tried lining techniques, namely

- the *bolted-segments technique*, whereby segments are bolted together under protection of a shield, with subsequent injection of grout in the annular space between the lining and the clay massif (Figure 2a);
- the *pipe jacking technique*, whereby entire rings are pressed in the clay massif (Figure 2b);

Excavation-damaged zone The damaged zone is an evolving zone experiencing geomechanical and geochemical modifications of its state (stress, pore water pressure, porosity, temperature, etc.) and material properties, which might have a negative effect in terms of operational safety (stability) and long-term safety (radionuclide confinement and transport), due to the construction, the operation, and the closure of a repository, and to the post-closure phase. The extent of the damaged-zone is problem dependent (namely, the system and scenarios being considered and the performance measures being addressed in performance assessments). Criteria defining the extent of the excavation-damaged zone and the damage intensity will be site, design, and time dependent.

- the *wedge-block technique*, whereby segments are assembled to form a ring that is then expanded against the excavated clay massif through insertion of one or more key segments or "wedge blocks" (Figure 2c);

the NIROND 92-03 report concluded that

- the bolted-segments technique had to be rejected due to the high disturbances it induces in the clay massif as a result of the convergence of the massif prior to injection;
- the pipe jacking technique and the wedge-block technique are almost equivalent as regards minimisation of the convergence;
- the pipe jacking technique is more difficult to implement at this depth than the wedge-block technique because of the high friction forces between the pipe and the clay. The only possible solution to reduce these forces, the injection of fluid bentonite, was not considered, among others because it increases costs, it increases convergence, and it could cause leaks.

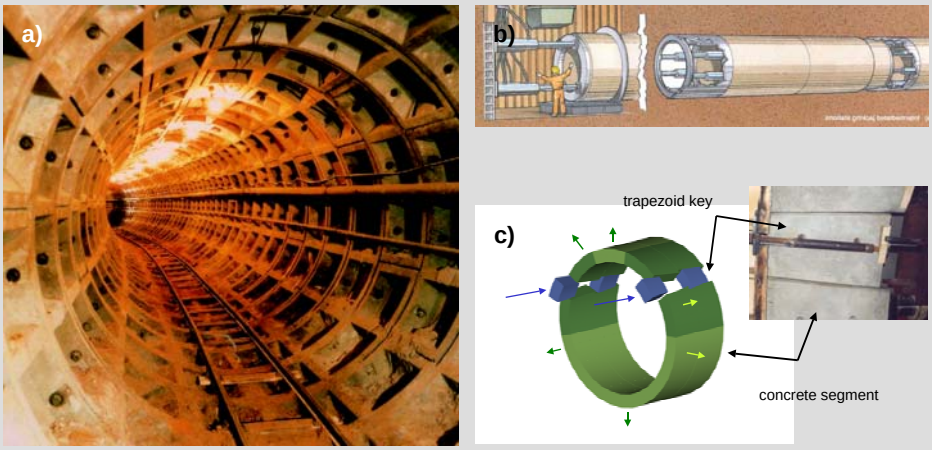


Figure 2: The three lining techniques studied in the NIROND 92-03 report. a) The bolted-segments technique, here with metal segments (Buchan) (concrete segments can be used, too); b) The pipe-jacking technique; c) The wedge-block technique, where the segmental lining is expanded against the massif through insertion of key segments.

2.2. Tendering procedures

The preparation of the construction of the connecting gallery as regards tendering proceeded in two main steps: the launch of a first procedure in 1998, which was later cancelled, and the launch of a second procedure in 2000, which led to adjudicating the contract to the temporary association SCM Tunnel.

The studies for the first tendering procedure were started again in 1997, after the end of the moratorium on the PRACLAY project, which had resulted from delays in the settlement of administrative issues between SCK•CEN and ONDRAF/NIRAS. This procedure covered the construction of both the connecting gallery and the PRACLAY gallery. The connecting gallery was to be constructed using a cast-iron lining, in order to enable the presence of openings for the installa-

tion of experiments; the PRACLAY gallery was to be constructed using the wedge-block technique. The selected adjudication procedure was the negotiation procedure. It was started in February 1998, but was cancelled on 30.06.1999 by the steering committee of EIG EURIDICE—then still EIG PRACLAY—for various reasons, among others the failure of some of the instrumentation used in the OPHELIE mock-up.

Following the cancellation of the first procedure, EURIDICE and its advisers decided very rapidly to investigate how to progress with the underground works and to do so in two phases:

- the so-called "crash programme", launched on 19.10.1999 and completed on 08.12.1999;
- an in-depth study and the setting-up of a new call for tenders file.

2.2.1. Crash programme

The crash programme aimed to enable a number of strategic decisions regarding the continuation of the underground infrastructure works to be taken. It led to the following results.

- *The adjudication of the connecting gallery was split from that of the PRACLAY gallery, the construction of the connecting gallery having to be granted as soon as possible*.
- *The restricted call for tenders procedure appeared as the most appropriate tendering procedure* for the construction of the connecting gallery.
- *The most suitable excavation technique for the connecting gallery was to use a tunnelling machine consisting of a road header under cover of a shield and to secure the clay front by anchors in case of problems.*
- *A double-lining system was selected.* The primary lining (concrete segments to be expanded against an accurately excavated profile according to the wedge-block technique) was to be placed progressively during excavation and had to be strong enough to counter the pressure of the clay massif. The secondary lining consisted of steel or cast-iron reinforcement rings, to be placed after construction of the gallery in those locations were openings for future experiments, such as the PRACLAY experiment, would have to be made.
- *The excavation of the connecting gallery, including the installation of the primary lining* (wedge blocks)*, was split from the realisation and emplacement of the secondary lining* (steel or cast-iron rings). This would enable the reinforcement rings to be fabricated and placed according to the needs, which was going to be cheaper and much more flexible.

2.2.2. Setting-up of the new call for tenders file

The construction of the connecting gallery has been designed in such a way as to limit both the extent of the excavation-disturbed zone and the extent of the excavation-damaged zone. This requirement was to be achieved by reaching a high construction rate and by excavating the massif accurately. Another important aspect was that the works needed to be strictly instrumented and controlled, in order to gain as much information as possible on the surrounding

OPHELIE Large-scale surface mock-up simulating a section of disposal gallery for high-level heat-emitting waste, in order to review several technical aspects of the disposal concept. The main focus of OPHELIE was on the engineered barriers and, in particular, on the thermo-hydro-mechanical behaviour of the backfill material and on its interactions with the other barriers. OPHELIE has been dismantled in 2002.

Excavation-disturbed zone The disturbed zone is the zone experiencing a significant modification of its state (stress, pore water pressure, porosity, temperature, etc.) due to the construction, the operation, and the closure of a repository, and to the post-closure phase. The damaged zone is a part of the disturbed zone. The disturbed zone without the damaged zone has no negative effect in terms of safety assessment.

The granulometric scale is divided into different fractions:

Clay
$Ø_{particles} < 0.002$ mm

Silt $0.002 \text{ mm} < Ø_{particles} < 0.062$ mm

Sand $0.062 \text{ mm} < Ø_{particles} < 2$ mm

Gravel
$2 \text{ mm} < Ø_{particles}$

Septaria Concretion in the shape of a loaf of bread that can attain a height of 30 cm and a diameter of two to three metres, and is characterised by dessication cracks or "septae". These cracks are frequently covered with a film of calcite or pyrite crystals. The septarias have formed in bands rich in limestone under the action of diagenetic processes caused mainly by the precipitation of carbonates around rotting micro-organisms (basic micro-environment). Bands of septarias are used to stratigraphically link different clay pits and deep boreholes.

clay massif and on its influence on the lining, this with a view to the installation of new experiments.

Since the whole design work of the connecting gallery and of the tunnelling machine depended on the geomechanical characteristics of the Boom Clay under the Mol site, the call for tenders file, together with the attached documents, contained a description of the relevant characteristics of the Boom Clay at this location.

The Boom Clay, or Boom Formation, belongs to the Rupelian, that geological part of the Tertiary Period which lasted from 36 to 30 million years ago. It is found at a depth of about 190 metres under the site of Mol, where it has a thickness of about 100 metres (Figure 3). It is surrounded by the overlying Neogene Aquifer (Sands of Mol, Kasterlee, Diest and Dessel, Antwerp, and Voort) and by the underlying Lower-Rupelian Aquifer (Sands of Ruisbroek and Onderdale). The Boom Clay is a silty clay (or argillaceous silt) characterised by a structure of bands that are several tens of centimetres thick, reflecting mainly cyclical variations in grain size (silt and clay content) due to fluctuations in the wave action on the sedimentation medium and to variations in the carbonate and organic matter contents. Typical concretions, known as septarias, are found in the marly bands occurring throughout the thickness of the formation.

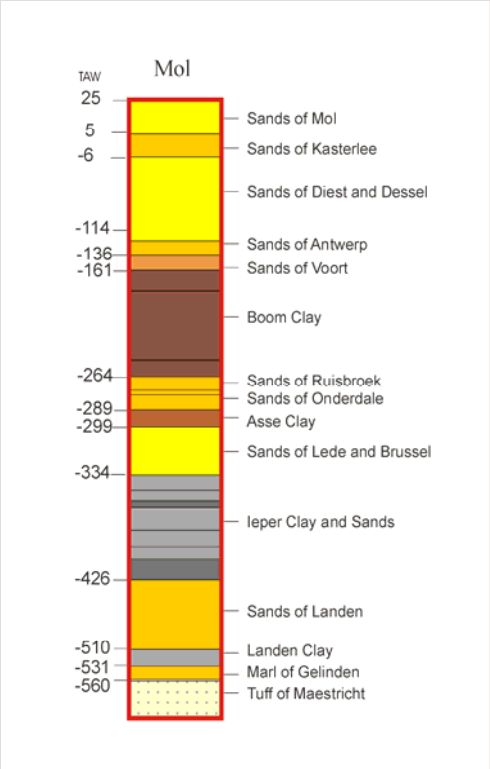


Figure 3: Geological section under the Mol site. (TAW is an acronym designating the reference topographic level (zero level) in Belgium, which corresponds approximately to the sea level.)

As regards mineralogy, the Boom Clay is characterised by a wide variation in the content of clay minerals (from 30 to 70 % volume, dry matter), due to its vertical lithological heterogeneity. In descending order of importance, the non-argillaceous fraction of the sediment consists of

quartz, feldspars, carbonates, and pyrite. The organic matter content ranges from 1 to 3 % weight, dry matter. The water content ranges from 30 to 40 % volume.

The undrained geomechanical characteristics of the Boom Clay at the depth of the existing facilities, assuming that the massif has a perfect elastoplastic behaviour, of the Mohr-Coulomb type, and that it has not been disturbed by excavation, are the following. (The characteristics around the second shaft and the front of the Test Drift can be less favourable.)

- Young's modulus at the origin — E — 200 to 400 MPa
- Poisson's ratio — ν — 0.4 to 0.45
- Angle of friction — φ — 4°
- Cohesion — c — 0.5 to 1 MPa
- Plastic limit — w_p — 23 to 29 %
- Liquid limit — w_l — 55 to 80 %
- Plastic index — IP — 32 to 51 %

The Boom Clay displays furthermore a visco-elasto-plastic behaviour, such that its convergence in the longer term is high. Its hydraulic conductivity is in the order of 10^{-12} m/s.

Besides the characteristics of the Boom Clay, the design work of the connecting gallery depended also on budgetary constraints. Certain features, such as certain automated systems, could indeed only be justified for galleries that are much longer than the connecting gallery (80 metres).

The technical specifications related to the whole construction project covered a range of topics, of which the most straightforward ones were the segmental lining, the tunnelling machine, and the construction scheme of the gallery (Figure 4). Another major topic was the construction of the so-called "mounting chamber", namely the construction, at the bottom of the second shaft, of a cylindrical chamber large enough to enable the tunnelling machine and all related equipment needed for the excavation activities to be assembled and started up. The studies that led to the design of the mounting chamber and of the connecting gallery and to the related technical specifications were performed by BELGATOM, in close cooperation with EURIDICE. Some important items of these specifications are summed up hereafter.

Eurocodes Set of nine Euronorms that contain common structural rules for the design of buildings and civil engineering structures, and are intended to serve as reference documents. Eurocodes are applicable to whole structures and to individual elements of structures and cater for the use of all major construction materials. They have been developed first as European prestandards and are now in the process of being converted into European standards. Eurocode 2 is the standard related to the design of concrete structures.

Requirements concerning the lining

The lining of the connecting gallery, which did not have to be waterproof, has been dimensioned on the basis of conservative hypotheses concerning the long-term behaviour of the clay massif. This dimensioning

- followed the Eurocode 2 and considered the calculations in ultimate limit state;
- had to guarantee that each ring would remain stable for an opening of 100 mm without needing to be reinforced by a secondary lining;
- assumed that the lining had to meet the following constraints:
  - its own weight;
  - the loads due to handling and erection;
  - the horizontal pressures due to the thrust of the jacks;

- the geotechnical ground pressure at equilibrium;

- took into account the thermal expansion of the concrete due to a temperature increase of approximately 11°C. This is the temperature increase that would be generated by a representative heating experiment in the future PRACLAY gallery (Figure 5), and that would thus induce extra constraints on the connecting gallery.

The dimensioning of the lining resulted into the following requirements.

- *Construction* The lining had to be made out of concrete rings consisting of 10 segments each, which were to be expanded against the excavated profile through insertion of two key segments (Figure 2 and Figure 6a).

- *Dimensions* The segments had to be 40 cm thick and 1 metre wide (Figure 6b), except the key segments, which had to have the same thickness while being only 0.850 metre wide, to enable an adequate expansion of the lining, in proportion to their depth of insertion. (The tolerance of the wedge-block technique on the nominal excavated diameter of the massif, for a maximum insertion depth of 150 mm per key, namely a maximum insertion depth of 300 mm for the two keys together, is 19 mm—Section 4.4.) The nominal external diameter of the expanded rings was 4800 mm; their nominal internal diameter was 4000 mm.

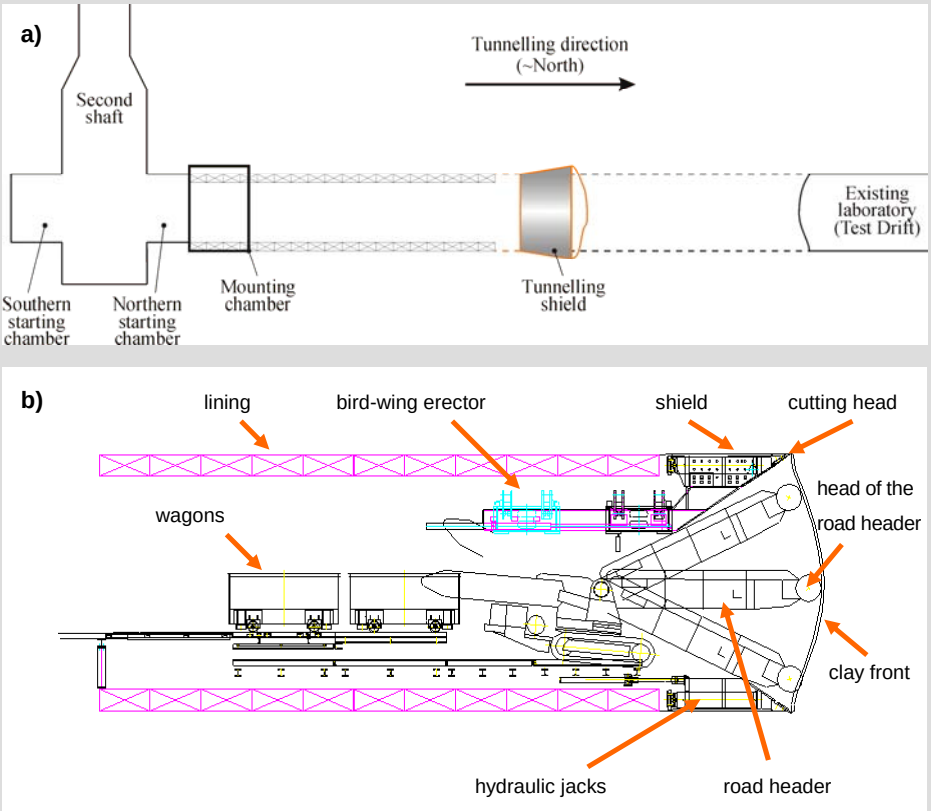


Figure 4: Construction of the connecting gallery. a) Cross-section showing the underground infrastructure; b) Cross-section showing the tunnelling machine in the connecting gallery (see also Figure 22).

- *Materials* The concrete was not reinforced and had to belong to the C65/80 class, according to the Eurocode 2. In other words, it had to have a compressive strength of 80 N/mm^2 (95 percentile) when measured on a cube with a 15-cm side. The gaps due to the fact that the keys were not as wide as the other segments had to be filled with a mortar belonging to the C45/55 class, namely with a mortar having a compressive strength of 55 N/mm^2 (95 percentile) when measured on a cube with a 15-cm side.

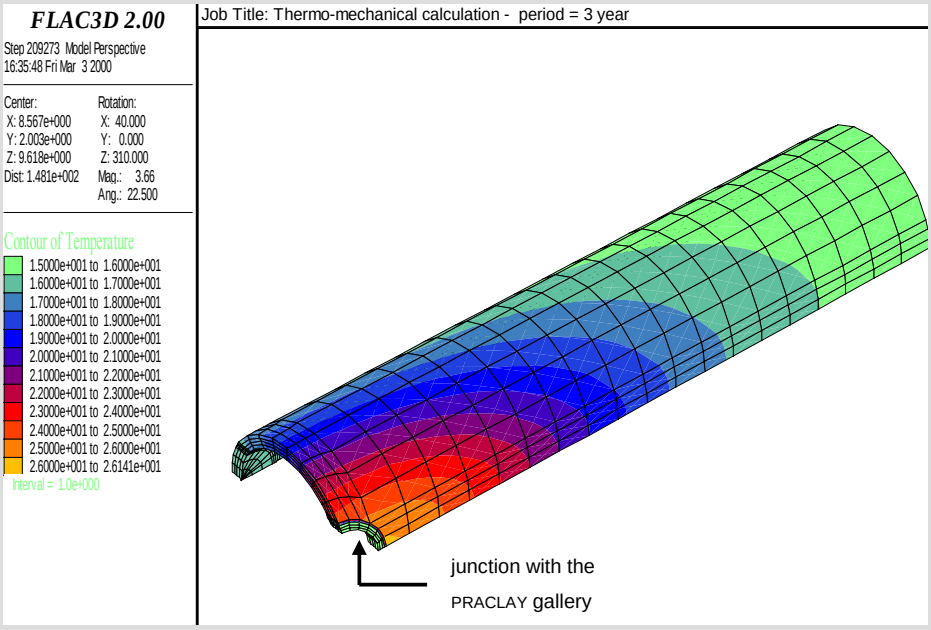


Figure 5: Modelling results of the temperature increase in the connecting gallery induced by a representative heating experiment in the PRACLAY gallery (reference in situ temperature: 15°C) (BELGATOM).

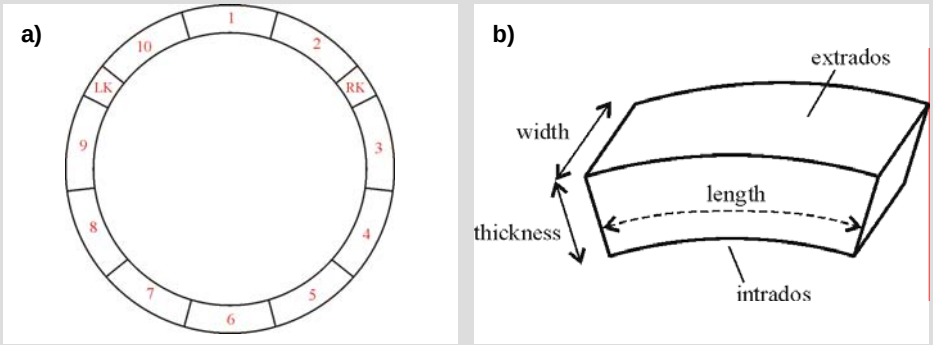


Figure 6: Geometry of the lining. a) Segment numbering (looking north, towards the Test Drift), segments 11 and 12 being the key segments; b) Terminology used for describing the segments.

Requirements concerning the mounting chamber

Fracture General term for a surface along which loss of cohesion has taken place.

The main design features of the mounting chamber (Figure 7 and Figure 8), not to mention the lining, were directly linked to the requirements imposed by the assembly of the tunnelling machine and its correct positioning.

- *Available space* The mounting chamber had to be bigger than the tunnelling shield. Its exact dimensions were to be specified by the selected contractor in function of the final dimensions of the machine.
- *Orientation and vertical positioning of the tunnelling shield* The mounting chamber had to be provided at its end with a first ring—the support ring—made from in situ poured reinforced concrete, at least 50 cm thick, both to orient the shield correctly and to support the clay at the beginning of the excavation of the connecting gallery. It also needed to be provided with a cradle meant to support the shield and to ensure its correct initial vertical positioning.

The construction scheme of the mounting chamber was chosen to minimise the convergence of the clay, and hence the risks of clay blocks coming off from the front and from the excavated profile. Two of the prime requirements set to this end were that the length of unsupported clay at the back of the shield could not exceed 1 metre and that the clay could not be left unsupported for more than 24 hours. The construction featured three main steps:

- the anchoring of the front of the northern starting chamber, as a direct result of the presence of fracture planes at the bottom of the second shaft;
- the excavation of the top half of the chamber, in 1-metre steps, and its progressive lining;
- the excavation and lining of the bottom half of the chamber according to the same scheme.

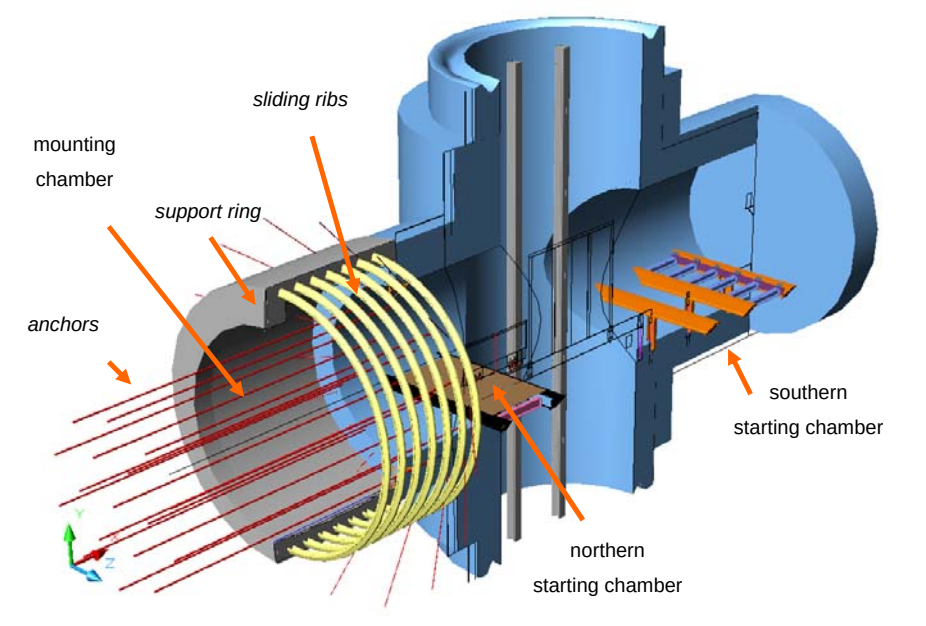


Figure 7: The mounting chamber and the two starting chambers (SCM Tunnel).

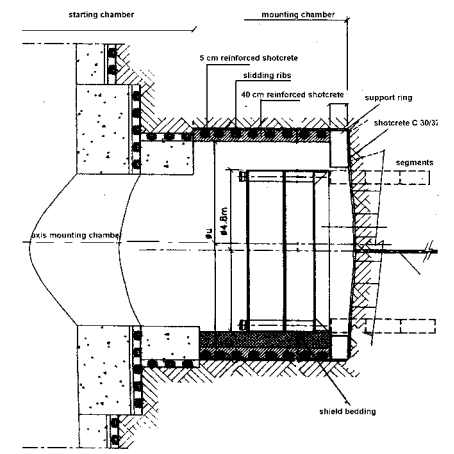


Figure 8: Design of the mounting chamber according to the technical specifications.

Requirements concerning the tunnelling machine

The final design of the tunnelling machine by the contractor to be selected had to take the following into account.

- The machine had to achieve an excavation and lining rate of 2 metres per 24 hours.
- The length of the shield had to be minimised to limit the risks that the machine would get trapped in the clay.
- The cutting head at the frontal circumference of the shield, meant to achieve the suited accuracy on the excavation diameter, had to allow the oversize (see Figure 19) to be adapted in function of the convergence. (Too large an oversize would increase the disturbances and, hence, the convergence; too small an oversize would increase the risks that the shield would get trapped by convergence.)
- The stroke of the jacks, and hence the length of the unsupported zone behind the shield, was to be limited to 1.2 metre, also to limit the convergence.
- The shield had to be provided with a real-time data-acquisition system, aimed to monitor some of its functioning parameters, and with small holes, aimed to enable the instantaneous convergence of the clay to be monitored, among others to modify the oversize if necessary.

Requirements concerning the connecting gallery

The construction of the connecting gallery had to be optimised in order to minimise the convergence and, hence, to limit the excavation-damaged zone. (Such requirements were a lot less relevant for the construction of the mounting chamber, since the clay massif had already been highly disturbed by the construction of the second shaft and since convergence was not really an issue there, whether it be in terms of excavation or in terms of placement of the lining.) To optimise the excavation of the connecting gallery, the selected contractor had to

- minimise the duration of the start-up phase and the length of the start-up zone;
- achieve a minimal construction rate of 2 metres per 24 hours;

- work without interruptions (7 days a week and 24 hours a day).

The construction of the connecting gallery had to proceed by successive iterations of the following sequence:

- excavation over a sufficient distance using a road header and under cover of a shield;
- forward movement of the shield, pushed by jacks resting on the already installed lining, the cutting head ensuring a smooth excavation profile;
- filling of the major surface irregularities;
- application of soap on the excavated profile, to reduce friction when installing the lining segments;
- lining of the excavated profile behind the shield with concrete segments.

The gaps left on the sides of the keys were to be filled after construction of the last lining ring.

Requirements concerning the connection to the Test Drift

The junction between the connecting gallery and the Test Drift was to be made by the shield itself.

Solutions to potential problems

Solutions were also designed to cope with three possible types of difficulties.

- *Insufficient convergence of the massif* Since the standard wedge-block technique could only accommodate a deviation of ± 10 mm on the nominal excavated diameter (4800 mm), the technical specifications included solutions to enable the rings to be built in case the excavated diameter remained up to 3 cm higher than the nominal diameter, because of insufficient convergence. These solutions were to use bigger keys or to place steel plates between the keys and the counter keys. (Some extra solutions were developed during the actual construction works to enable the wedge-block technique to be used in case the true excavated diameter deviated even more from the nominal excavated diameter (Section 4.4).)
- *Reduction of the construction rate* A simple, systematic anchoring of the front was possible in case the construction rate slowed down.
- *Jamming of the tunnelling machine* An ultimate solution was worked out in order to proceed with the works without prolonged interruption in case the tunnelling machine got trapped in the massif. In such case, the excavation was to proceed manually under cover of an anchored front and a temporary lining consisting of heavy HEB steel ribs and glass-fibre reinforced shotcrete. (This solution, the use of which was strictly defined in the technical specifications, was worked out because the PRACLAY experiment was then still scheduled to start at the end of 2003. Later, the decision to revise the repository architecture by the end of 2003 and, hence, the postponement of the PRACLAY demonstration experiment, led EURIDICE to decide not to resort immediately to the ultimate solution in case the shield got trapped. There would indeed then be enough time, after having secured the front, to select the most appropriate technique to finish the construction of the gallery.)

2.2.3. Adjudication

As already mentioned, the selected tendering procedure was the restricted call for tenders with prior announcement. The announcement was released on 28.01.2000. Seven contractors were prequalified and invited to introduce their tender. The criteria used for evaluating the four tenders that had been received by 29.09.2000 were the technical quality, the total amount of the contract (ultimate solution not included—Section 2.2.2), the quality of the site organisation, the total amount of the ultimate solution, and the planning. The analysis of the tenders led EURIDICE to adjudicate the contract for the construction of the connecting gallery on 15.01.2001 to the temporary association SCM Tunnel, a joint venture between SMET-Tunnelling (Belgium), Wayss & Freytag (Germany), and Deilmann-Haniel (Germany).

3. General organisation of the works and planning

The construction of the connecting gallery involved four main parties. Besides SCM Tunnel, contracted for the construction works, EURIDICE had indeed contracted two external offices: the engineering office BELGATOM (a joint venture between Tractebel and Belgonucléaire) and the Technical Control Bureau for Construction SECO. They all worked in very close cooperation throughout the project, managing to meet the schedule quite successfully (Figure 9).

From an organisational point of view, the whole construction project can be divided into four main types of activities, during which the various parties held different roles:

- the adjudication procedure;
- the detailed design work, both before and during part of the construction works;
- the construction works;
- the follow-up activities during the construction works.

3.1. Adjudication procedure

The various aspects of the adjudication procedure belonged to the following parties:

- announcement of the works and prequalification in the framework of the restricted call for tenders procedure: EURIDICE, assisted by BELGATOM;
- setting-up of the technical specifications: BELGATOM, in close cooperation with EURIDICE;
- evaluation of the tenders submitted by the various candidate contractors: EURIDICE, assisted by BELGATOM and SECO;
- administrative tasks: EURIDICE.

3.2. Detailed design work, both before and during construction

The five-month design phase (from 16.01.2001 to 08.06.2001) that followed the contract adjudication was used to turn the existing technical specifications into detailed work plans and procedures. This was done by SCM Tunnel, which was contractually bound to submit to EURIDICE all its documents regarding works to be carried out, such as proposals, plans, procedures, and calculation notes. These documents had to be approved by EURIDICE before the corresponding works could be started.

The SCM Tunnel documents were also checked by BELGATOM, for their technical correctness and their compliance with the technical specifications, and by SECO, for the correctness of the stability calculations. Weekly meetings were organised to discuss their contents.

3.3. Construction works

The construction works have been taken care of by SMET-Tunnelling, a partner of SCM Tunnel, under EURIDICE supervision. (The manufacturing of the segments was subcontracted by SCM Tunnel to Buchan, an English company experienced in the construction of wedge blocks.) The

construction of the mounting chamber, the installation of the shield and of the underground equipment, and the construction of the connecting gallery have all been performed by the same team of workers, under supervision of the construction supervisor of SCM Tunnel, assisted by the engineers who had also been involved in the studies, the design, and the preparatory works.

- The *existing site infrastructure* has been adapted in several ways between February and June 2001 to meet the specific requirements of the underground works to be carried out: the existing lift cage was replaced by one fitted out with the necessary facilities, such as rails, mechanical locking devices, and hoists, to facilitate the transport and handling of the materials and equipment to be used underground, a travelling crane was erected to handle the heavy construction materials, the surface infrastructure, the shaft infrastructure, and the underground loading floors were adapted, and reference points were installed in the shaft.
- The execution planning for the *mounting chamber*, set up by SCM Tunnel, took into account not only the nature of the works, but also the limited transport capacity of the lift and the number and kinds of activities that could be done simultaneously in the limited underground workspace. In practice, the mounting chamber was excavated according to a working regime of 24 hours a day for 5 days a week by two teams averaging six workers per shift. This shift work was necessary to minimise convergence and, hence, the collapse of the front, but since the clay massif had already been substantially damaged by the excavation of the second shaft, weekend interruptions, after securing of the front by a 5-cm layer of shotcrete, were judged acceptable a priori. The excavation activities were completed in 39 working days, namely about the duration foreseen by the initial planning. Prior to the excavation, the anchors had been installed in a normal working regime.
- The *connecting gallery* was also constructed according to a working regime of 24 hours a day, but for 7 days a week instead of 5, and by two teams averaging ten workers per shift instead of six. The main reason for working shifts, 7 days a week, was to reach the target excavation and lining rate. The faster the works progressed, the less risk there was that the tunnelling machine would get trapped in the massif, which would have led to the failure of the project. Weekend interruptions were unacceptable for the same reason. Penalties and bonuses were foreseen in the contract in case the construction rate was lower, respectively higher, than the target rate. Practically, the construction rate reached peaks of 4 metres in 24 hours (Section 6.6). The excavation and lining of the connecting gallery were completed in 37 days, namely less than the duration foreseen in the initial planning. This achievement resulted in the full bonus being granted to SCM Tunnel.

Specialists were also called in for specific tasks.

- Specialists were called in for tasks related to the installation of the shield in the mounting chamber, such as the hydraulic connection of the hydraulic pumps and jacks, the connection of the erector, and the setting-up of the electrical facilities.
- English specialists from Kane Tunneling (two per shift) skilled in the installation of a wedge-block lining assisted the Belgian SCM team during the construction of the connecting gallery.
- BELGATOM hired an English expert during the first weeks of excavation for his expertise in tunnelling using the wedge-block technique.

3.4. Follow-up activities during the construction works

The follow-up activities during the construction works took the form of daily and weekly meetings and of field inspections.

- The *daily meetings* with SCM Tunnel and the *weekly meetings* between all four parties were aimed to overview the progress of the works, to discuss the encountered difficulties and the unsafe situations, and to decide on the actions to take in order not to jeopardise the progress of the works. Two types of daily reports were drawn up.
  - The *activity reports*, which were drawn up and discussed at the daily meetings, included the following types of information: number and identity of all those present on site, weather conditions if relevant, activities, tests, and controls performed, description and availability of the materials and equipment present on site, special events that had occurred during the activities, problems encountered during the works and possible solutions, any safety or technical issue that could influence the progress of the works, and planning follow-up.
  - The *observation reports*, which were part of the activity reports, reported the observations made during excavation and their analysis.
- The *field inspections* were taken care of by EURIDICE, BELGATOM, and SECO. SECO was more particularly in charge of the regular controls of the execution of the civil works with a view to building the file needed by EURIDICE to obtain the decennial guarantee.

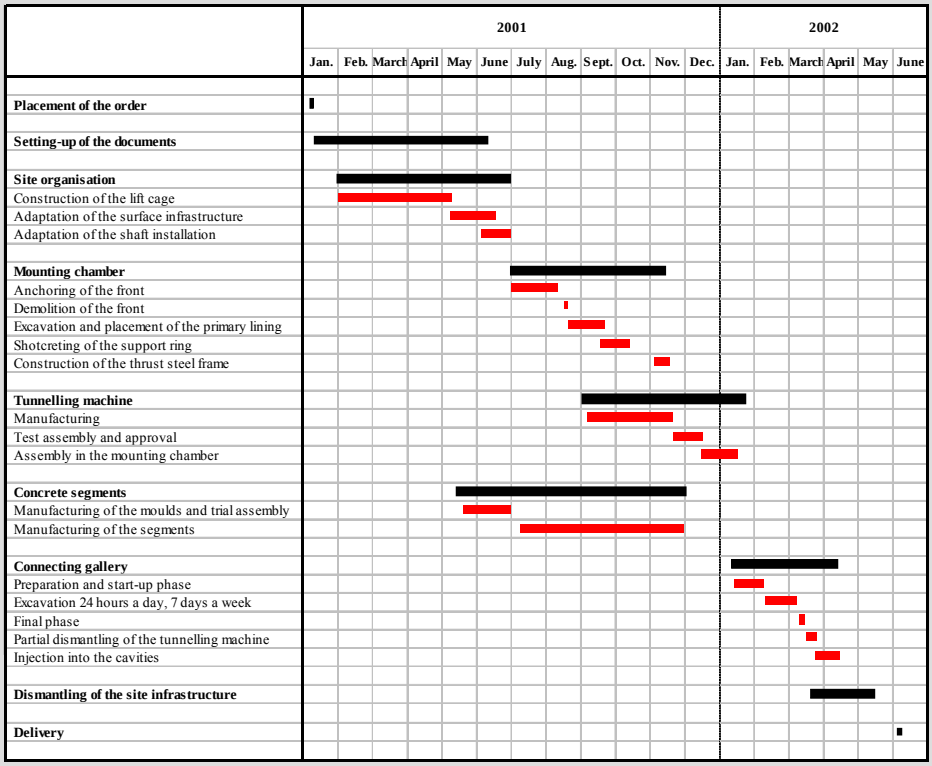


Figure 9: Execution planning of the whole construction project.

4. Preparatory works

The main preparatory works were the following: an auscultation programme of the massif, aimed to characterise the damaged zone around the shaft, the fine-tuning of the shield geometry, the manufacturing of lining segments and the determination of the properties of the concrete used, the development of a decision table aimed to enable fast and adequate reactions in case the excavated diameter of the clay in the unsupported zone behind the shield was beyond the tolerances of the wedge-block lining, and a test assembly of the tunnelling machine.

Shear plane (slip plane, slip surface) The plane over which a unit of rock slips.

Slickenside A rock surface with a polished appearance and fine parallel scratches caused by displacement of the rock during fault formation, where a fault is a fracture surface along which there has been differential movement.

4.1. Auscultation programme

The unexpected large shear planes observed during the construction of the northern and southern starting chambers at the bottom of the second shaft (Figure 10) led to carrying out an *auscultation programme* prior to starting the excavation of the mounting chamber. This programme aimed to collect as many data and as much knowledge as possible concerning the origin and the extent of the damaged zone around the shaft and, in particular, to confirm the hypothesis that the fractures were associated with the construction of the shaft. (The lack of active support during excavation of the starting chambers and the low excavation rate have certainly favoured the opening of the fractures and the detachment of clay blocks.) The shear planes that were observed consisted indeed of an interconnected network of fracture planes inclining at 35° towards the shaft axis, with circular shapes suggesting that they were symmetric around its axis and with slickensides clearly showing a movement towards it.

Figure 10: Large shear planes observed during the construction of the southern starting chamber. They are curved around the shaft axis and dip towards it.

The auscultation programme consisted of seismic measurements, carried out by the German Federal Institute for Geosciences and Natural Resources (BGR), and two cored boreholes (Figure 11). It confirmed the hypothesis that the fractures did not have a natural origin (in other words, that they were not pre-existing), but had been induced by the construction of the second shaft instead. (Later, the construction works revealed that the zone between 0 and 2 metres was a zone with open fractures and that the zone in the next metres was a zone with closed fractures (Section 6.2).)

4.1.1. Seismic measurements

The seismic measurements aimed to reveal disturbances in the clay massif via measurements of the associated variations in the seismic velocity and in the damping of the amplitudes of the seismic waves. There were two types of measurements: microseismic interval velocity measurements and cross hole seismic measurements. Two 20-metre-long boreholes (no. 2000-4 and 2000-5) were drilled to this end. They were horizontal, nearly parallel and separated by a distance of 3.6 metres, and oriented towards the north (Figure 11 and Figure 12).

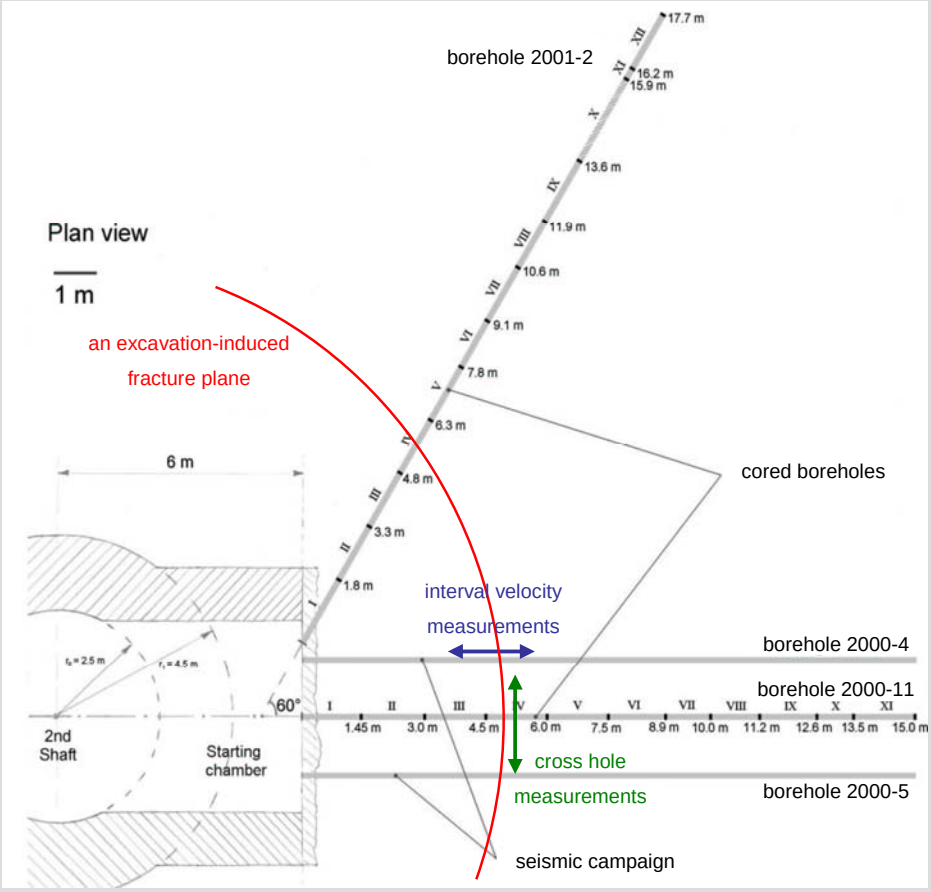


Figure 11: Plan view of the four horizontal boreholes of the auscultation programme, with schematic indication of an excavation-induced fracture plane and of the propagation direction of the seismic waves in both interval velocity measurements and cross hole measurements.

Figure 12: The two boreholes used for the seismic measurements (indicated by the yellow arrows) were located at mid-height of the starting chamber (BGR).

Microseismic interval velocity measurements reveal the seismic velocity around a borehole, which is the propagation speed of seismic waves in the massif around that borehole. It is measured by a probe, equipped with a piezoelectric transducer (source) and three receivers, inserted in the borehole (Figure 13). The distance—or interval—between the transducer and the receivers is fixed. After insertion of the probe in the borehole up to a certain depth, the transducer and receivers are pushed against the borehole sidewall by a pneumatic system. The transducer then sends an acoustic signal into the massif, and the signal is detected by the receivers. The probe is connected to a PC registering the times of emission and of recording of the signals. This enables the seismic velocity to be calculated, since the distance between each of the receivers and the transducer is known. The calculated velocity is attributed to the point at mid-distance between the transducer and the receiver. This procedure is repeated at several depths. Since the length of the wave path is independent from the measurement depth, the signal-to-noise ratio and the frequency content of the signals depend neither on the depth nor on absorbing discontinuities outside of the interval under consideration, which is important for the reliable identification of fractures and small-scale anomalies. The presence of fractures and microcracks caused by excavation is indicated by the variations in wave velocities that are recorded. Since the wave paths are parallel to the borehole axis, they are indeed perpendicular to the fractures and microcracks caused by the excavation of the shaft (fracture planes symmetric around the shaft axis—Figure 11).

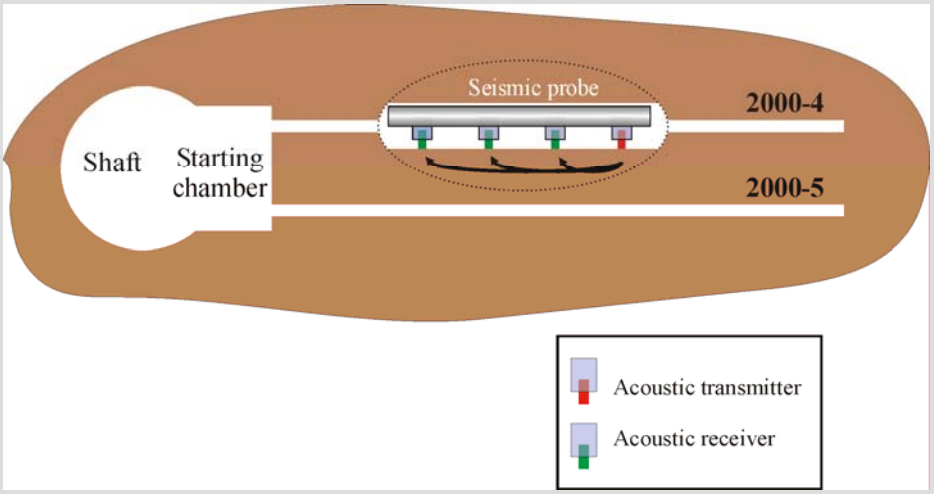


Figure 13: Microseismic interval velocity measurements. A transducer sends a signal into the massif surrounding the borehole and this signal is detected by three receivers.

The interval velocity measurements, which were carried out between 09.10.2000 and 18.10.2000, started at the borehole mouth, the probe being introduced into the borehole in 10-cm steps. They were carried out in two phases. The first phase started when the boreholes were 10 metres long; the second started when they had reached their target length. The sidewalls of both boreholes became unstable at approximately 10 metres, which led to interrupting the measurements at depths of 13.8 and 13.2 metres respectively. They were then cased with

P-wave (acoustic wave, compressional wave, dilatational wave) An elastic body wave or sound wave in which particles oscillate in the direction the wave propagates. P-waves are the waves studied in conventional seismic data. P-waves incident on an interface at other than normal incidence can produce reflected and transmitted S-waves, in that case known as converted waves.

S-wave (shear wave, tangential wave) An elastic body wave in which particles oscillate perpendicular to the direction in which the wave propagates. S-waves can be converted to P-waves. S-waves travel more slowly than P-waves and cannot travel through fluids because fluids do not support shear.

PVC tubes to prevent further damage before the cross hole seismic measurements. The measurements revealed the following features (Figure 14):

- low P-wave velocity and high damping of amplitudes up to about 2 metres (2.8 metres in borehole 2000-4 and 1 metre in borehole 2000-5), with many small-scale disturbances;
- gradual increase of the P-wave velocity between approximately 2 and 5 metres, to an average level of 1900 m/s;
- constant P-wave velocity (about 1900 m/s) between 5 metres and approximately 8.5 metres (9.2 metres in borehole 2000-4 and 7.4 metres in borehole 2000-5);
- very strong velocity variations (1400 to 1800 m/s) and high damping of amplitudes between approximately 8.5 metres and the end of the measurements (13.8 metres in borehole 2000-4 and 13.2 metres in borehole 2000-5).

The results of the interval velocity measurements in the two boreholes have been used to define regions in the clay that are characterised by similar seismic parameters (Figure 14).

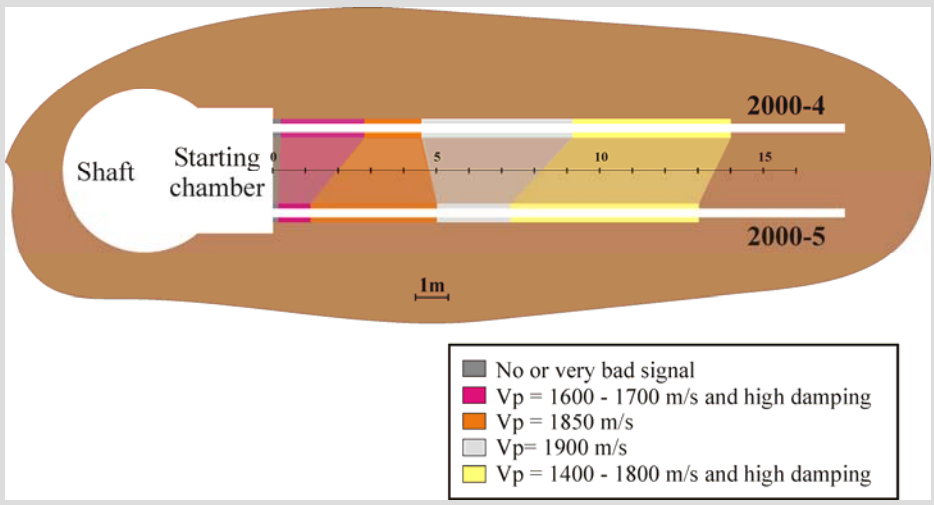


Figure 14: Microseismic interval velocity measurements. The measurements lead to defining regions in the clay that are characterised by similar seismic parameters.

Cross hole seismic measurements have the advantage that they enable a larger part of the clay to be investigated. Such measurements require several (in the present case, two) boreholes at a time. A seismic source (a powerful piezoelectric transducer) is placed in one borehole and receivers (in the present case, the probe used for the interval velocity measurements) are placed in the other one (Figure 15). Again, the transducer and the receivers are connected to a PC. The results are averaged seismic parameters between both boreholes, assuming that seismic waves travel on straight paths between source and receiver. This assumption is not entirely correct, though, and it introduces uncertainties in the results. The calculated velocities are attributed to the points at mid-distance between the transducer and the receivers.

Two configurations have been used successively for the cross hole seismic measurements, which were performed between 07.10.2000 and 10.10.2000: one with the transducer in bore-

hole 2000-4 and the receivers in borehole 2000-5, the other being the mirror configuration. They revealed the following features (Figure 16):

- a very low P-wave velocity in the first 2 metres;
- an increasing velocity between 2 and 7 metres;
- a relatively constant velocity between 7 and about 14.5 metres;
- a slight reduction of the velocity beyond 14.5 metres.

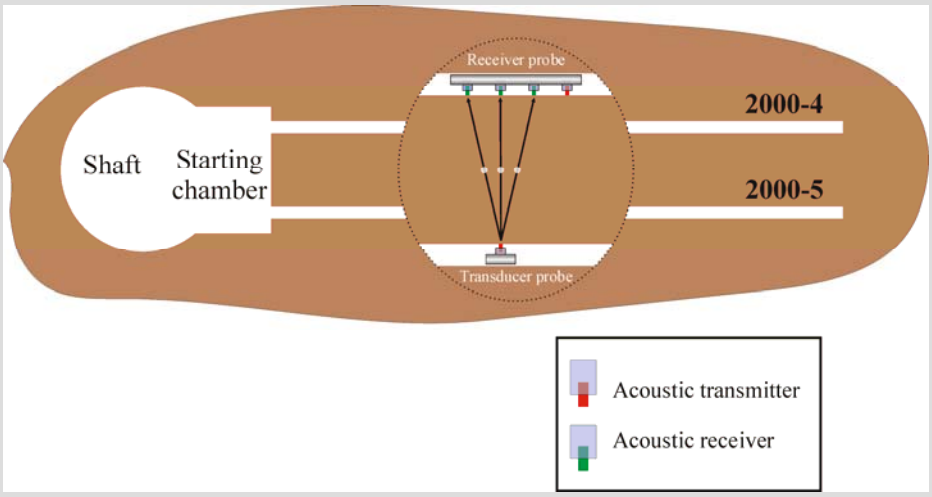


Figure 15: Cross hole seismic measurements. A transducer placed in one borehole sends a signal into the massif and this signal is detected by three receivers placed in another borehole.

Observations of boreholes 2000-4 and 2000-5 with a video camera immediately after drilling provided information about their mechanical stability and revealed macroscopic damages in their deepest part, as did the two corings performed later on (Section 4.1.2). These drilling-induced damages influenced both types of seismic measurements.

- For the interval velocity measurements, which involved the volume of clay within the immediate surroundings of the boreholes, the drilling-induced damages explained the large velocity variations and the high damping of amplitudes between approximately 8.5 metres and the end of the boreholes.
- For the cross hole measurements, which involved the mostly intact volume of clay between the boreholes, the drilling-induced damages explained the slight reduction of the velocity near the end of the boreholes.

The combined interpretation of the cross hole measurements, the interval velocity measurements, and the observations with a video camera led to the following conclusions (Figure 16).

- The excavation-damaged zone ranges up to about 5 metres into the massif (or up to about 11 metres from the second shaft axis).
- The massif features a lot of small-scale disturbances up to approximately 2 metres.
- The cross hole measurements resulted in a rather smooth velocity profile, while the interval velocity measurements showed a lot of disturbances in the first metres. These results emphasise the strong seismic anisotropy within the disturbed massif, which can be related

to the fact that the damages caused by the excavation of the shaft are parallel to the shaft wall: seismic waves travelling perpendicularly to the shaft wall, as in interval velocity measurements, are influenced very strongly by these damages, whereas seismic waves travelling tangentially to the shaft wall, as in cross hole measurements, are less influenced by the (micro)fractures.

- Damages induced by the borehole drilling process itself influence strongly the measurements.

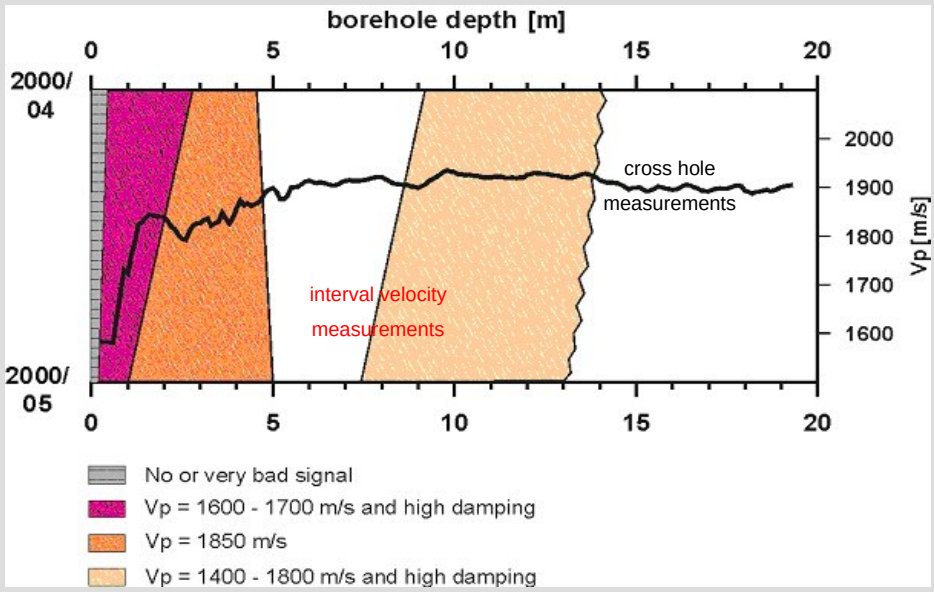


Figure 16: Compilation of the main results of the seismic measurements (based upon a BGR figure). The interval velocity measurements in boreholes 2000-4 and 2000-5 led to defining several regions in the clay, each one of them being characterised by similar seismic parameters, whereas the cross hole measurements (from borehole 2000-4 to borehole 2000-5) led to determining the evolution of the P-wave velocity.

Dip The smallest of the two angles that a planar feature makes with the horizontal plane, as measured in a plane normal to the strike.

Strike The direction of the intersection of a planar feature with a horizontal plane.

4.1.2. Cored boreholes

Two horizontal boreholes (no. 2001-2 and 2000-11) were drilled in two different directions (Figure 11) and the results of the cores taken were correlated with those of the seismic measurements. Depending on their orientation, the cored boreholes revealed different information. The orientation of borehole 2001-2 enabled one to distinguish between fractures caused by drilling and fractures induced by the shaft excavation: two shear planes with dip directions other than the direction of the borehole axis were found (Figure 17). Their strikes were approximately tangential to circles centred on the shaft axis and they were both dipping towards the centre of the shaft (dips: 34° and 38°). The most distant one was located at more than 5 metres from the starting chamber. These two fractures confirm the hypothesis of a pattern of large, curved shear planes, dipping towards the centre of the shaft. All discontinuities at more than 7 metres from the starting chamber had been induced by the drilling process itself. No indication of the presence of natural

discontinuities was encountered. By contrast, the orientation of borehole 2000-11, parallel to that of the two boreholes that were used for the cross hole measurements, made it impossible to distinguish between drilling-related fractures and fractures induced by the excavation of the shaft. The fact that the striations of some shear planes were far clearer and outspoken than others in its first metres also suggested, however, that the excavation-damaged zone extended as far as 5 to 7 metres. Furthermore, all discontinuities between 0 and 7 metres in borehole 2000-11 were dipping towards the centre of the shaft (south), while both dip directions (north and south) occurred further away. However, the core analysis did not allow the high-damping zone (first 2 metres) and the normal-damping zone (2 to approximately 8.5 metres) revealed by the interval velocity measurements to be identified.

The interpretation of fracture planes in cores is complicated by the many fractures induced by the drilling process itself. Two types of drilling effects were observed: the formation of fractures having their dip direction parallel to the borehole axis (Figure 18a), and the expulsion of clay pieces from the sidewalls of the boreholes, at depths between 7 and 15 metres (Figure 18 b and c), as revealed by camera observations. The cross section of the boreholes was therefore very irregular at this depth, which can explain the results of the seismic measurements.

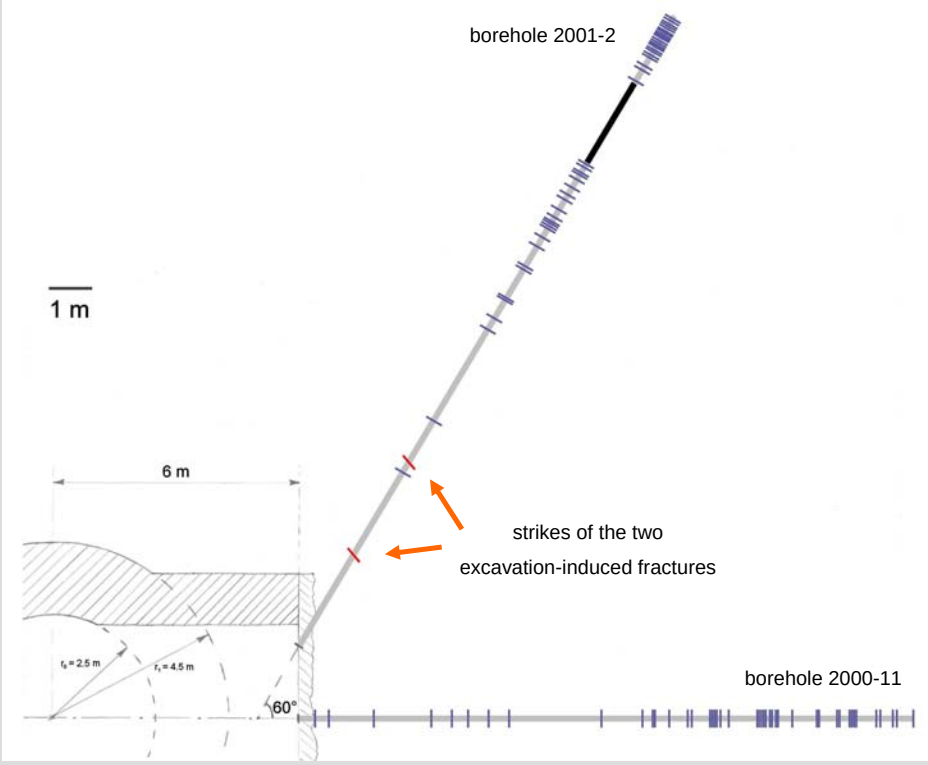
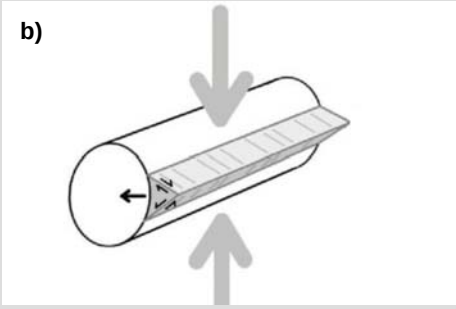


Figure 17: Plan view of the strikes of the discontinuities observed in the cores of boreholes 2001-2 and 2000-11.



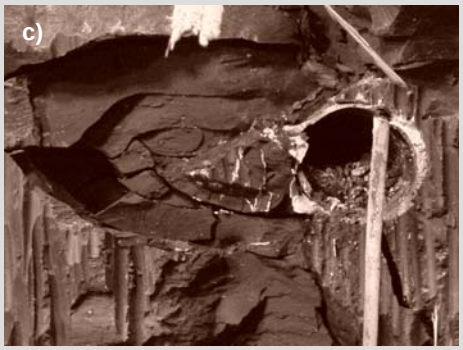


Figure 18: The two types of drilling effects observed in the cored boreholes. a) Intense fracturation, according to the typical pattern induced by the drilling process, observed in the last core taken from borehole 2001-2; b) The mechanism that often leads to clay pieces being expelled from the sidewalls of the boreholes; c) Same type of fracturation as under b), very intense, around borehole 2000-5.

4.1.3. Conclusions

The auscultation programme has been very useful. It enabled the fractured zone around the shaft to be characterised, the excavation-damaged zone ranging up to about 5 to 7 metres into the massif (about 11 to 13 metres from the second shaft axis). It also supported the hypothesis that the fractures had been induced by the excavation of the shaft, as indicated by the orientation of the fracture planes and of the striations. These results suggested that the fractures induced by the shaft (and quantified during the auscultation programme) would be encountered during the first metres of the excavation of the mounting chamber and of the connecting gallery. The excavation of the mounting chamber and of the connecting gallery would in turn also induce new fractures, but their frequency and exact shape were still uncertain at the time. These various conclusions have been confirmed during the excavation of the mounting chamber and during the first few metres of the excavation of the connecting gallery (Section 6.2). Finally, the auscultation programme helped designing the anchors needed to reinforce the front of the northern starting chamber (Sections 2.2.2 and 5.1.1).

4.2. Fine-tuning of the shield geometry

The fine-tuning of the shield geometry concerned three of its main characteristics: the diameter at the rear end, the oversize of the cutting head, and the shape (Figure 19).

- The final diameter of the tunnelling shield was based on modelling results of the response of the massif to excavation. To take account of the convergence of the clay in the unsupported zone behind the shield (15 mm on the radius, as determined with the FLAC soft-

ware, using a two-dimensional axisymmetric model), the diameter of its rear end was increased by 20 mm, namely from 4800 mm, which is the nominal external diameter of the lining, to 4820 mm.

- Since the exact instantaneous convergence of the Boom Clay during excavation with a road header under cover of a shield was uncertain at the time, the cutting head was provided with adjustable knives (Figure 22d). The possible oversizes were to be 0, 10, 20, or 30 mm, the initial intention being to start the excavation with an oversize of 20 mm. This variable oversize was intended to allow the overexcavation to be adapted during the works in function of the real convergence. (Adjusting the oversize was done by inserting steel plates of the required thickness between knives and shield, after unscrewing the bolts keeping the knives in place.)
- To reduce friction between the shield and the massif and to make steering easier, the shield was made slightly conical over its whole circumference (conicity: 10 mm on the diameter).

In a second phase, more detailed modelling with FLAC indicated that the convergence would be larger than initially thought (Section 6.4.2). The initial oversize was therefore set to 30 mm for starting the excavation works. This choice proved suitable, thereby confirming the use of prior modelling.

FLAC (Fast Lagrangian Analysis of Continua) Two-dimensional continuum code for modelling soil, rock, and structural behaviour. It is a general analysis and design tool for geotechnical, civil, and mining engineers. FLAC can be obtained from HCITasca, an independent engineering consulting and software development firm.

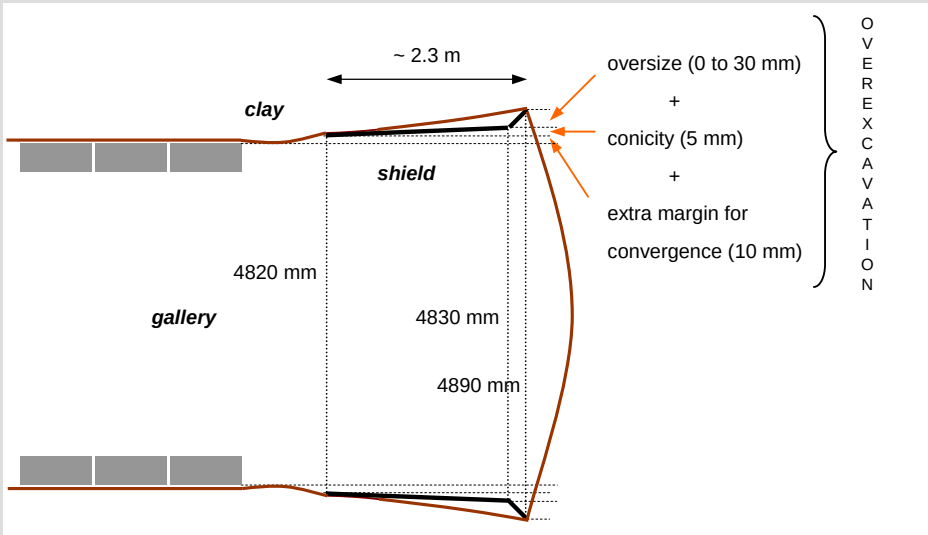


Figure 19: Cross section of the tunnelling shield, showing its conical shape, the adjustable knives providing the oversize, and the total overexcavation (not on scale).

4.3. Manufacturing of the lining segments and concrete properties

The lining segments were manufactured by Buchan, on behalf of SCM Tunnel, using Buchan moulds (Figure 20). These moulds were made out of reinforced concrete, except their extrados, which was made out of a removable steel framework. They were such that the surfaces be-

tween ordinary segments and between ordinary segments and counter keys were flat, but that the surfaces between the keys and the counter keys were slightly helical. The latter was done to ensure an exact and easy radial positioning of the keys and to decrease the risk of damaging them during insertion. A trial assembly of two full rings was performed at Buchan's prior to launching the full fabrication of the segments.

Figure 20: Manufacturing of the lining segments. a) General view of an instrumented segment, also showing its four manipulation holes; b) A mould with its removable steel framework; c) Support frame with instrumentation ready to be installed in a mould; d) Casting of an instrumented segment; e) Trial assembly of two rings; f) Segments stored on site.

The various types of segments that were fabricated can be briefly described as follows (Figure 6a). All were 400 mm thick. The guaranted dimensional tolerances were ± 0.3 mm on the length at extrados, ± 1.5 mm on the thickness, and ± 1.5 mm on the width (Figure 6b). How-

ever, some of the segments did not meet the tolerances on their top side, namely on the side that was free during casting. Whenever this occurred, this side was finished manually.

- *Ordinary segment (segments 1, 4, 5, 6, 7, and 8)*: length at extrados: 1407 mm; width: 1000 mm; weight: about 1262 kg.
- *Left normal key segment (segment 11)* (tapered segment): width: 850 mm; length at extrados: between 687 mm (wedge-block leading edge) and 517 mm (wedge-block trailing edge); weight: about 524 kg.
- *Right normal key segment (segment 12)*: same as left key segment, though there is a difference in the positioning of the manipulation holes.
- *Left counter key segment (segments 9 and 10)*: width: 1000 mm; length at extrados: between 1307 mm (wedge-block leading edge) and 1407 mm (wedge-block trailing edge); weight: about 1218 kg.
- *Right counter key segment (segments 2 and 3)*: same as left counter key segment, though there is a difference in the positioning of the manipulation holes.
- *Longer left and right key segments* (called "special segments") were also manufactured to enable slightly larger ring diameters (Section 4.4).

Two other types of segments were also present on the site: smaller key segments, called "normal sawn keys", both left and right ones (Section 4.4). These were normal keys, of which the longest extremity had been sawn off, to make them only 700 mm wide.

The load tests that were performed on 182 twenty-eight-days-old cubic samples (150 × 150 × 150 mm) made out of the same concrete as the one used for the lining segments (HSR CEM II/B-V 42.5 type, highly resistant to sulfates) indicated that the 95 percentile of the compressive strength was 91.3 N/mm^2, namely a value that was significantly higher than the 80 N/mm^2 requested in the technical specifications (Section 2.2.2).

4.4. Development of a decision table

Since the wedge-block technique only allows a relatively small tolerance on the nominal excavated diameter of the massif (see following paragraph), and since the response of the clay to an industrial tunnelling technique at about 225 metres depth had never been investigated in the past, several methods were designed to enable the wedge-block lining to be emplaced even if the excavated diameter was beyond tolerances (though within certain limits). These methods were then compiled in a decision table for excavation in steady state, aimed to enable fast and adequate choices if ever needed during the works (Figure 21).

The tolerance of the wedge-block technique on the nominal excavated diameter of the massif, for a maximum insertion depth of 150 mm per key, namely a maximum insertion depth of 300 mm for the two keys together, was about 19 mm, as given by the following formula:

$$\Delta \text{ diameter} = \frac{\text{insertion}}{5 \cdot \pi} = \frac{300}{5 \cdot \pi}.$$

Figure 21: Decision table for excavation in steady state.

This meant, in other words, that it would not be possible to build the lining rings using the wedge-block technique as foreseen in case the excavated diameter at the rear end of the shield was below 4 791 mm or above 4 810 mm. The various solutions that were worked out to increase the acceptable range of excavated diameters were the following.

- *If the convergence exceeded 99 mm on the diameter* (4 890 mm at the front end of the shield – 4 791 mm in the unsupported zone behind the shield), the solution was to reduce the width of the keys to 700 mm: these were the "normal sawn keys". This width reduction enabled the ring to be built in an excavated diameter of 4 772 mm (4 791 mm –

19 mm). The acceptable reduction of the excavated diameter due to convergence during the lapse of time between excavation and lining placement thus became 118 mm. Since the keys had to be sawn off with special equipment, 8 sawn keys (4 left keys + 4 right keys) were kept available at all times on the construction yard, to minimise delays in case there was much convergence.

- *If the convergence was smaller than expected or if the installed oversize of the cutting head was too large*, three solutions were available to enable larger ring diameters: placing 10-mm-thick shims (high-density polyethylene plates) between the ring segments, placing 10-mm-thick curved plates (annular plates) on the outside of the ring, or using bigger keys, called "special keys". These special keys were 30 mm longer than the normal keys, which would increase the ring circumference by 60 mm and, hence, the ring diameter by 60 mm/π = 19 mm. Curved plates (1 or 2) were preferred over shims (2, 4, or 6), because they were thought to be better as regards ring stability. To preserve the symmetry of the rings, shims always had to be used in pairs, and the two keys of a ring always had to be of the same type: sawn, normal, or special.

Finally, adjustments to the oversize would have to be considered in case the difference between true and nominal (4800 mm) diameter was consistently too large.

4.5. Test assembly of the tunnelling machine

Prior to being transported to the mounting chamber, all the tunnelling equipment (shield, road header, erector, rail tracks, and hydraulics) was assembled in a nearby building, in the same configuration as for the actual excavation, to check all the functions and the geometry of the machine, separately and as a whole (Figure 22), and to provide an opportunity to make the necessary adjustments prior to starting the works. This test was all the more needed since the limited volume of the mounting chamber was going to make the underground assembly of the tunnelling machine one of the most difficult and dangerous steps of the project. Important tests were the manipulation of a lining segment and the insertion of a key into the corresponding pocket of the shield (see also Section 5.2.4).

The test assembly was officially inspected by EURIDICE on 10.12.2001 and 13.12.2001. It then received a first approval, subject to the successful placement of the first lining ring in the mounting chamber (Section 5.2.3).

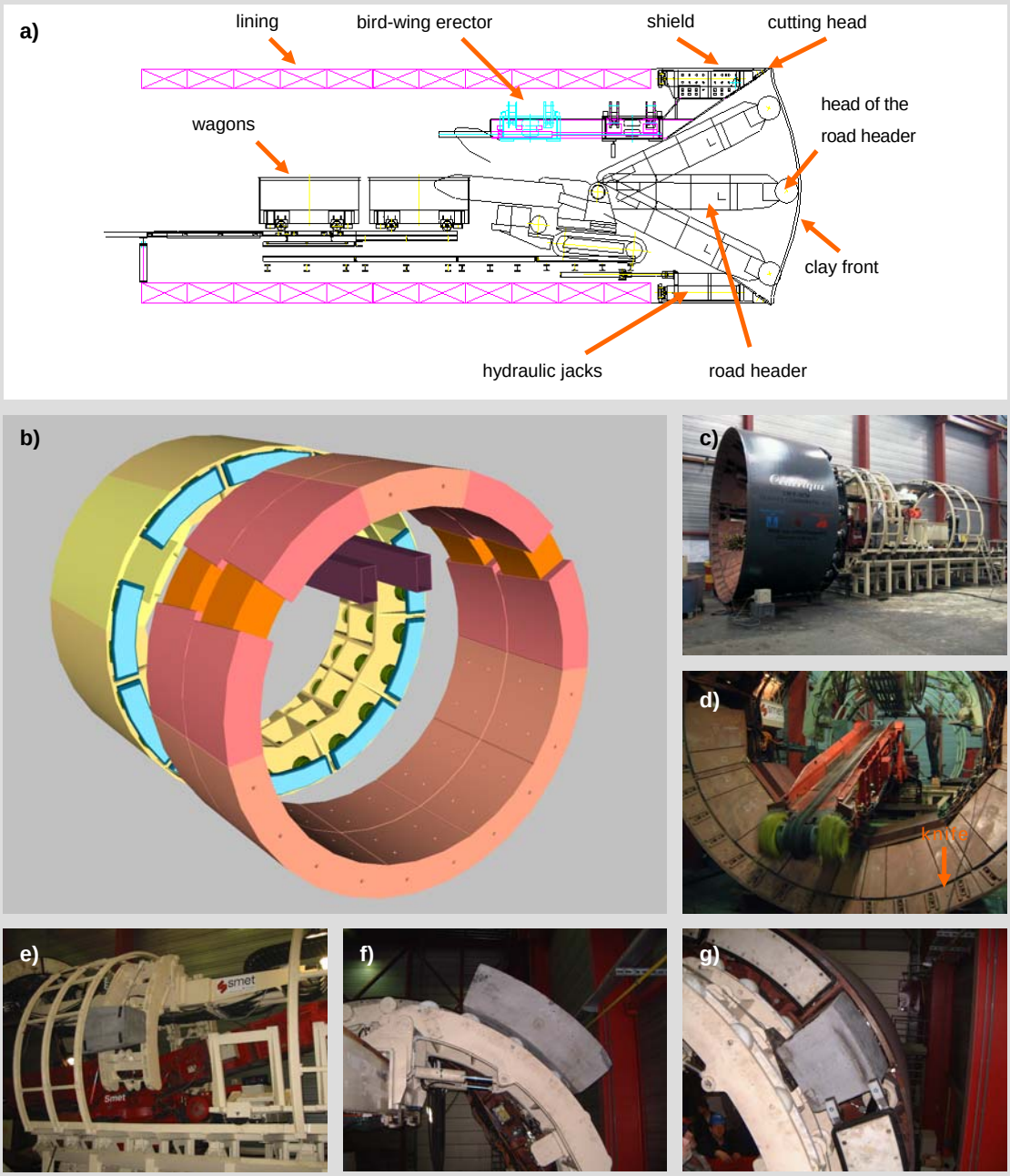


Figure 22: Test assembly of the tunnelling machine. a) Simplified sketch; b) The wedge-block lining at the back of the shield (SCM Tunnel); c) The tunnelling machine; d) The head of the road header and the cutting head with its adjustable knives; e) The bird-wing erector; f) Testing of the manipulation of a lining segment; g) Testing of the insertion of a key into the corresponding pocket of the shield.

5. Construction works

The so-called "construction of the connecting gallery" can actually be divided into three successive phases:

- the construction of the mounting chamber (Section 5.1);
- the construction of the connecting gallery (Section 5.2);
- the connection to the Test Drift (Section 5.3).

They have overall been successful, the technical problems encountered during the whole construction process and related to the design aspects being only minor. There has been one major, unexpected problem, though: the extent of the detachment of clay blocks from the front and from the unsupported sidewalls.

5.1. Construction of the mounting chamber

The construction of the mounting chamber, between 20.08.2001 (beginning of the excavation of the front) and 11.10.2001 (end of the construction of the primary lining), has proceeded without any major difficulties.

The detailed engineering of the tunnelling machine and the design of the mounting chamber, combined with the difficult working conditions, led SCM Tunnel to overdimension the mounting chamber slightly with respect to its minimal dimensions, and to give it an excavated diameter of 6.40 metres and a length of 3.75 metres.

The construction of the mounting chamber (Figure 23) can be split in five phases: the preparatory works before the start of the excavation works, the excavation of the first metre of the chamber, the excavation of the remaining part of its top half, the excavation of the remaining part of its bottom half, and the finishing works, including the construction of the facilities needed to install the tunnelling machine and to start the excavation of the connecting gallery.

5.1.1. Anchoring of the front and adaptation of the site infrastructure

The front of the northern starting chamber was provided with anchors before the start of the excavation works: sixteen 8-metre anchors parallel to the axis of the starting chamber and ten 3-metre anchors at an angle of 45°, equally distributed on the circumference of the front (Figure 24 and Figure 25). Their installation took longer than expected, mainly because of the (expected) presence of pyrite in the clay and because the hand-drilling tool used by SCM Tunnel for installing the first anchor turned out to be too light for drilling in Boom Clay and had to be replaced by a pneumatic drill mounted on a carriage.

The surface and underground adaptations of the working floors, the replacement of the lift cage, and the installation of the reference points are dealt with in Section 3.3.

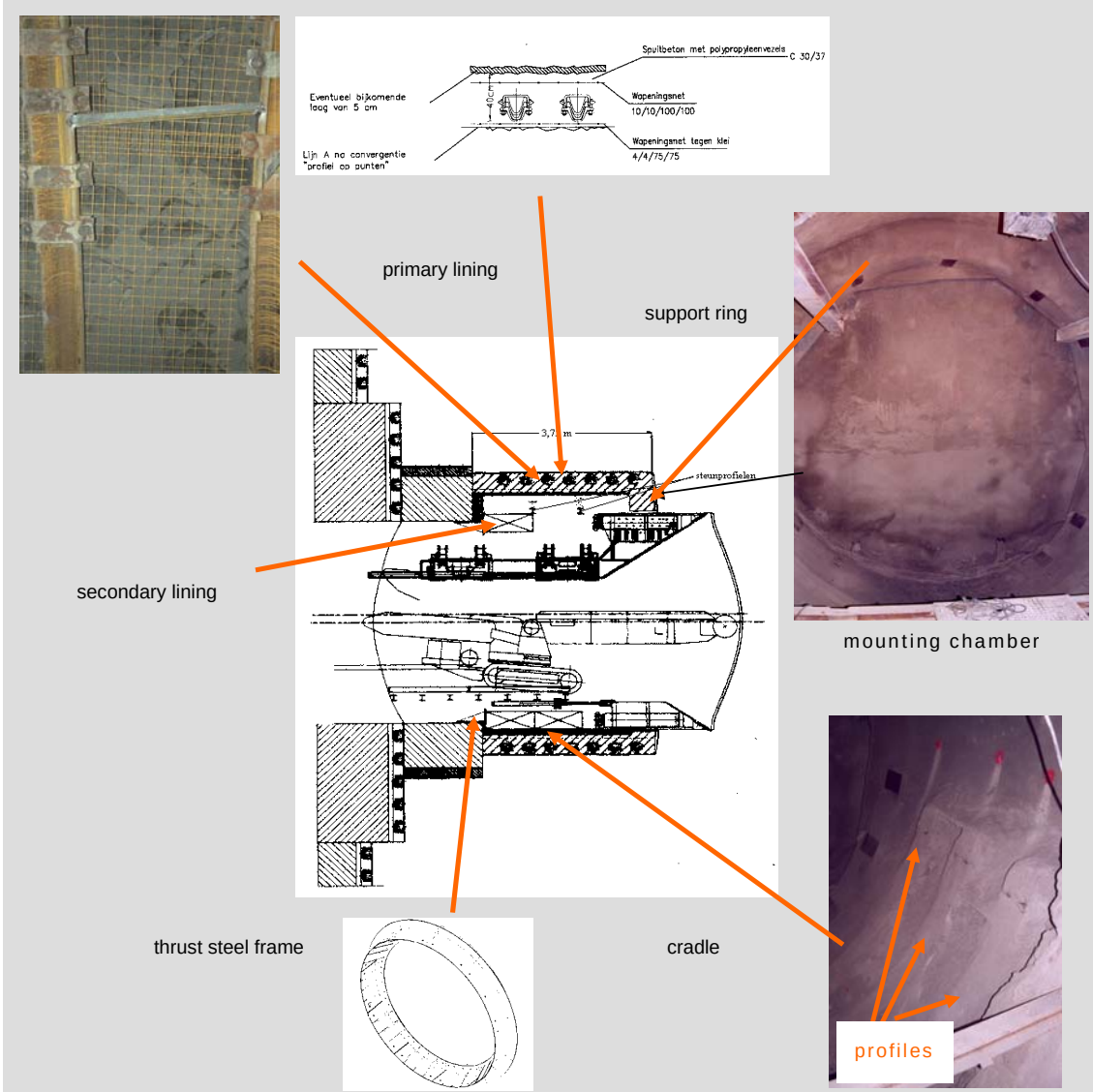


Figure 23: The mounting chamber as built.

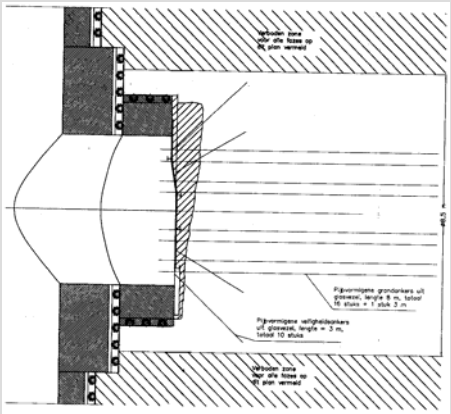


Figure 24: Anchoring of the front of the northern starting chamber.

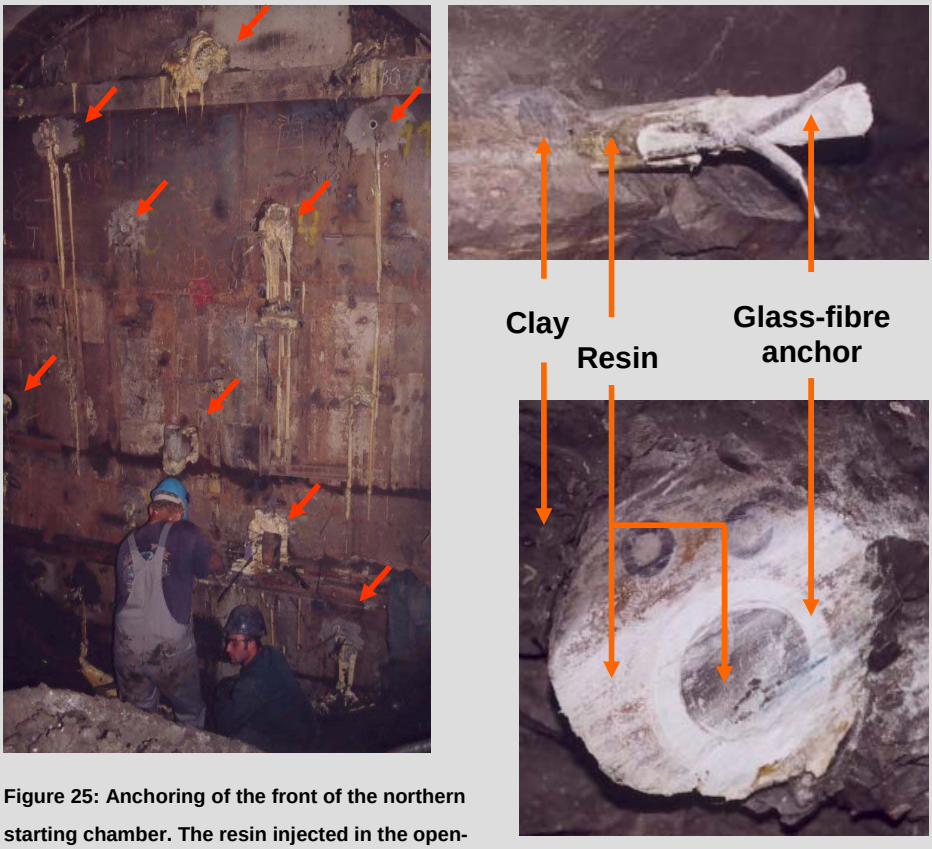


Figure 25: Anchoring of the front of the northern starting chamber. The resin injected in the open-ended tubular glass-fibre anchors ensured a close contact between the anchors and the massif.

5.1.2. Excavation of the first metre

Instead of first excavating the top half of the mounting chamber as initially foreseen, SCM Tunnel proposed to excavate the mounting chamber over its full cross section, in 1-metre steps, using a pick hammer (Figure 26a), and to secure the front by a 5-cm layer of shotcrete before weekends. This excavation scheme was indeed applied for the first metre:

- excavation of the top half of the mounting chamber over a distance of 1 metre;
- placement of a first wire-mesh, with small meshes, against the clay massif, both to protect the workers against falling clay blocks and to improve the adhesion of the shotcrete onto the clay. (This was decided because the many large cavities present in the roof and sidewalls made it impossible to apply the 5-cm layer of reinforced shotcrete as foreseen[1].);
- placement of the top half of the sliding ribs of rings 1 and 2, to support the clay massif (Toussaint-Heintzmann sliding ribs, placed every 50 cm and as soon as possible after ex-

[1] The relatively small quantities of glass-fibre reinforced shotcrete required and the limited underground workspace led to order pre-blended, ready-for-use shotcrete, which was delivered in bags of 25 kg. They indeed did not justify a blending installation on surface, which would also have implied a risk of segregation of the shotcrete due to the pumping height (230 metres).

cavation, weighting at least 44 kg per metre and having a minimum cross section of 56 cm^2);

- shotcreting of the top half of the front, up to 5 cm thickness;
- excavation of the bottom half of the chamber over a distance of 1 metre;
- placement of the first wire-mesh;
- placement of the bottom half of the sliding ribs of rings 1 and 2 and closing of those rings;
- placement of bags of wet cement between the excavated massif and the ribs, in order to ensure a close contact between them;
- shotcreting of the bottom half of the front, up to 5 cm thickness;
- tightening of the sliding ribs of rings 1 and 2 up to a moment of 750 Nm;
- extension of the work platform;
- welding of the sliding ribs of rings 1 and 2;
- shotcreting of the walls, up to 5 cm thickness.

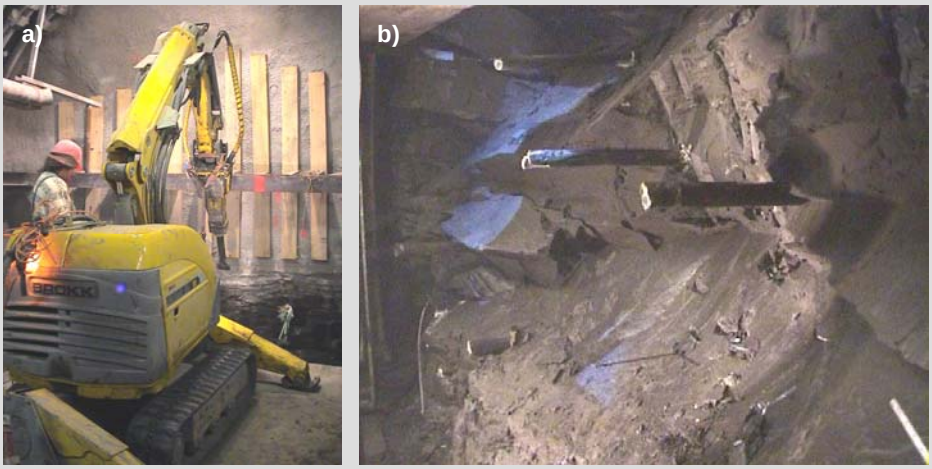


Figure 26: Excavation of the mounting chamber. a) The pick hammer excavation tool and the timbering of the front; b) Cavities in the front (external diameter of the anchors and of the surrounding resin: about 80 mm).

5.1.3. Excavation of the remaining part of the top half

When restarting the excavation activities after the weekend, clay blocks—some of them of up to several cubic metres—came off from the massif because of fracturation (Figure 26b). These fractures were thought to be the consequence of the following three factors: the low excavation rate, the heavy vibrations created by the pick hammer, and the insufficient support of the front during the weekend. The supporting anchors did not perform as efficiently as expected, since a zone of secondary fracturation around them was decreasing the friction with the clay. These secondary fractures have however considerably limited the size of the clay blocks detaching themselves from the massif.

Returning to the initial excavation scheme was necessary to ensure the safety of the workers. It started with the following (Figure 27):

- excavation of the next metre of the top half of the chamber;
- placement of the first wire-mesh;
- placement of the top half of the sliding ribs of rings 3 and 4;
- further excavation of the top half of the chamber up to the required depth;
- placement of the first wire-mesh;
- placement of the top half of the sliding ribs of rings 5, 6, and 7;
- spherical excavation of the top half of the front;
- shotcreting (10 cm, non-reinforced shotcrete) of the top half of the front;
- shotcreting of the walls, up to 5 cm thickness;
- reinforcement of the top half of the front by timbering (Figure 26a).

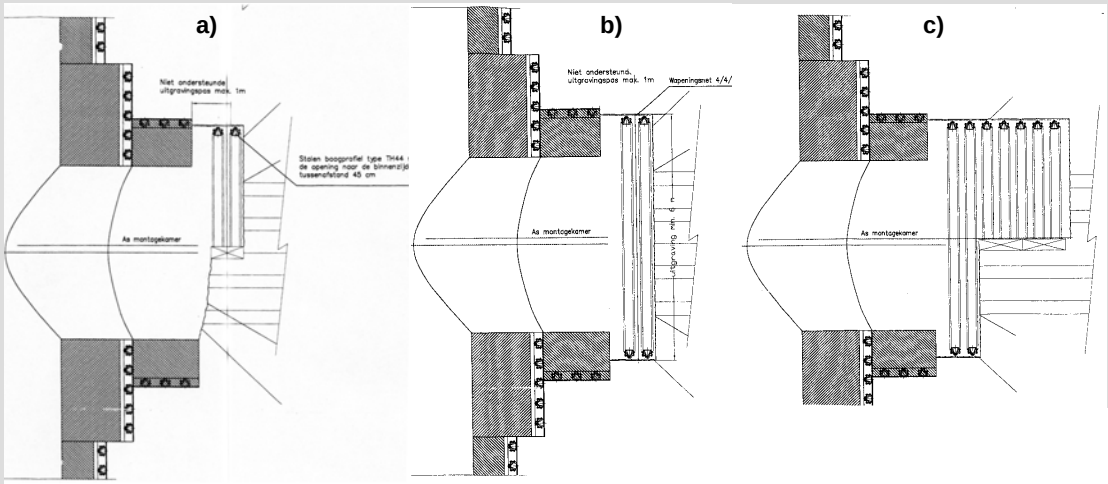


Figure 27: Excavation of the mounting chamber. a) Excavation of the top half over a distance of 1 metre and installation of the top half of the first two sliding ribs; b) Excavation of the bottom half over a distance of 1 metre and closing of the first two sliding ribs; c) Excavation of the complete top half and installation of the sliding ribs. (The excavation of the complete bottom half is not depicted.)

5.1.4. Excavation of the remaining part of the bottom half

The excavation of the remaining part of the bottom half of the mounting chamber proceeded in the same way as the excavation of its top half:

- excavation of the bottom half of the chamber, placement of the first wire-mesh, placement of the bottom half of the sliding ribs of rings 3, 4, and 5, and progressive closing and welding of the ribs to prevent any further displacement;
- shotcreting of the walls between rings 3, 4, and 5, up to 5 cm thickness;
- shotcreting (5 cm, non-reinforced shotcrete) of the bottom half of the front;
- extension of the work platform;

- excavation of the bottom half of the chamber, placement of the first wire-mesh, placement of the bottom half of the sliding ribs of rings 6 and 7, and progressive closing and welding of the ribs;
- further spherical excavation of the front;
- shotcreting (10 cm, non-reinforced shotcrete) of the bottom half of the front;
- shotcreting of the walls between rings 6 and 7, up to 5 cm thickness;
- reinforcement of the bottom half of the front by timbering.

5.1.5. Finishing and preparation of the construction of the connecting gallery

Once the excavation works had been completed, the following activities were still necessary before installing the tunnelling machine:

- the shotcreting of the sidewalls up to the required thickness, namely minimum 40 cm;
- the installation of a support ring at the end of the mounting chamber (Section 2.2.2);
- the installation of a cradle for the shield (Section 2.2.2);
- the installation of a thrust steel frame.

These activities have been done according to the following sequence:

- welding of a second wire-mesh with larger meshes against the sliding ribs;
- placement of the reinforcement of the support ring at the end of the mounting chamber;
- partial shotcreting of the support ring, using the same shotcrete as the one used for the primary lining;
- placement and adjustment of the steel profiles of the cradle, meant to support the shield and to ensure its correct initial vertical positioning;
- placement and adjustment of T-profiles, with a view to the later placement of spacers for orienting the shield accurately;
- partial demolition of the timbering of the front;
- shotcreting of the sidewalls and of the support ring;
- demolition of the timbering of the front;
- construction of a thrust steel frame at the beginning of the mounting chamber, meant to provide a support for the jacks of the tunnelling machine at the start of gallery excavation (Section 5.2.3).

The secondary lining of the mounting chamber has been placed when starting the excavation of the connecting gallery (Section 5.2.3).

5.2. Construction of the connecting gallery

Like the construction of the mounting chamber, the construction of the connecting gallery (Section 5.2.4) proceeded without any major difficulties, between 23.01.2002 and 09.04.2002. Besides the thorough preparation works that preceded the start of the underground construction works (Sections 4.2 to 4.5), some extra preparation was actually needed between the end of the placement of the primary lining of the mounting chamber and the start of the excavation of the connecting gallery (Sections 5.2.1 to 5.2.3).

5.2.1. Control of the tunnelling direction

The tunnelling shield had to be guided in the correct direction by a laser beam parallel to the theoretical axis of the gallery. The laser source was fixed in the southern starting chamber and the direction of the beam was set up on the basis of topographic reference points in the shaft. However, some doubts about the correctness of these points and, hence, of the direction itself prompted EURIDICE to have them checked again. The new measurements and the subsequent calculations led to a direction that was different—but only by about 0.0665 gon—from the one that had been determined previously. This very slight difference was attributed partly to the tolerance on the measurements and partly to possible movements of the reference points. Since the support ring and the shield cradle had already been constructed—on the basis of the initial direction—, slight steering corrections would be necessary during the first few metres of excavation, to re-align the shield on the newly determined direction. These corrections proved relatively easy to perform (Section 5.2.4).

Gon Measurement of plane angles, corresponding to 1/400 of a full circle, thus dividing a right angle in 100. One gon is equal to 9/10 of a degree or $\pi/200$ of a radian.

5.2.2. Assembly of the tunnelling machine in the mounting chamber

After the test assembly in a surface building, the tunnelling machine was dismantled and all its parts were transported to the mounting chamber in a well-organised and thought out manner (Figure 28a) and re-assembled underground (Figure 28b). The initial positioning of the shield was ensured by the cradle constructed on the bottom of the mounting chamber and by the support ring.

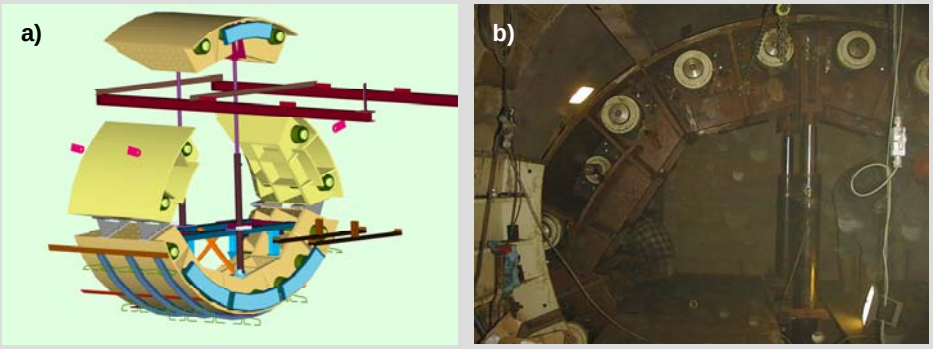


Figure 28: Assembly of the tunnelling machine. a) One of the many illustrations of the story-board of the shield assembly (SCM Tunnel); b) Assembly in the mounting chamber.

5.2.3. Construction of the first lining rings and excavation of the front

The construction of the secondary lining of the mounting chamber was another important preparatory work before starting the construction of the connecting gallery. The lining of the connecting gallery had indeed to be placed against that of the mounting chamber. The secondary lining of the mounting chamber was made out of the same concrete wedge blocks as those used for the lining of the connecting gallery (40-cm-thick non-reinforced segments).

The exact positioning of the first lining ring in the mounting chamber was important, since it conditioned the positioning of all subsequent rings. It depended primarily on the adequate design and construction of two steel devices:

- *steel rings*, meant to compensate for the fact that the inner diameter of the mounting chamber (about 5.3 metres) was larger than the outer diameter of the lining (4.8 metres) and, hence, to enable the first rings to be expanded (Figure 29a);
- *the thrust steel frame*, built at the beginning of the mounting chamber and meant as a support for the hydraulic jacks (Figure 29b) (Section 5.1).

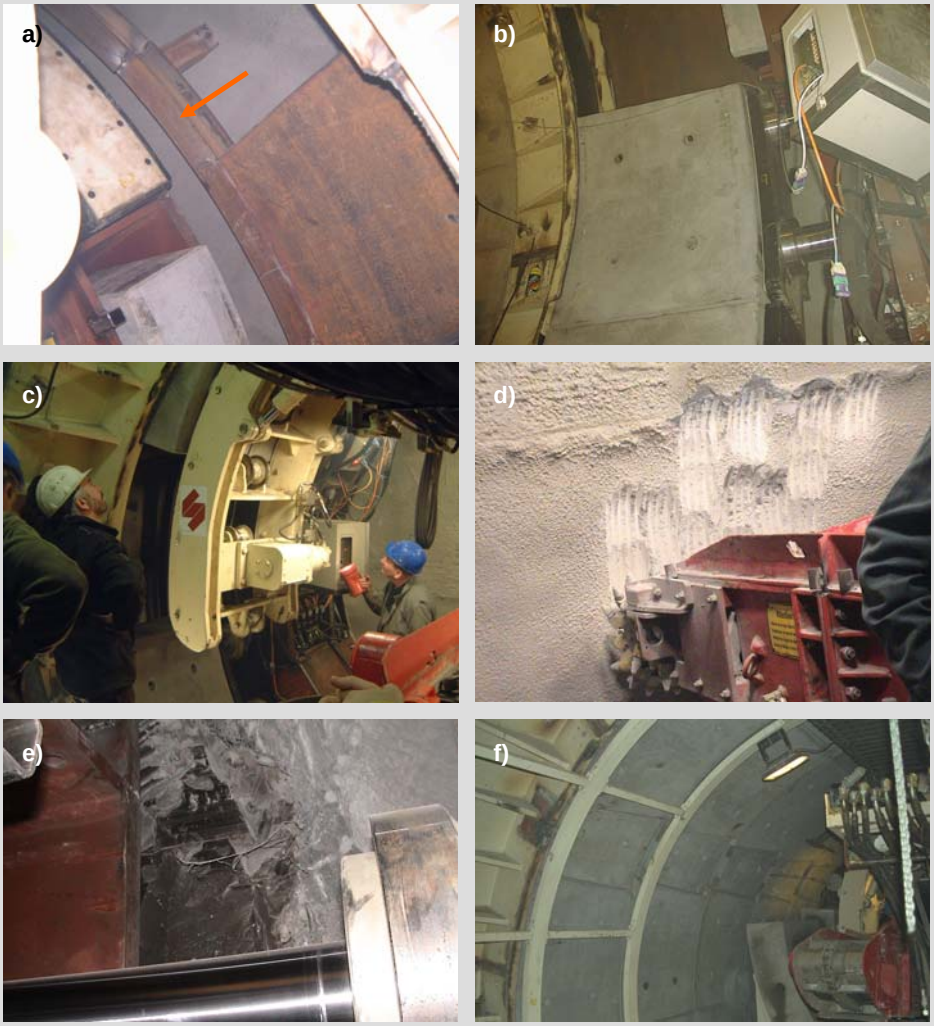
Since the mounting chamber was just long enough to accommodate the shield and one lining ring, it was not necessary to demolish the concrete front to build the first ring. This avoided any risk of large convergence of the clay, should the placement of that ring have taken a while or should adjustments to the equipment have been needed.

The first lining ring was placed and expanded successfully on 25.01.2002, which led to the full approval of the tunnelling equipment (Figure 29 b and c) (see also Section 4.5). Its dimensions were well under control. The concrete front was then excavated: its outer parts were removed manually with pneumatic hammers over the full circumference in order to avoid damaging the Amercoat coating on the outside of the shield, meant to reduce friction when pushing the shield forward; the rest of the front was excavated with the road header (Figure 29d).

The construction sequence of the other lining rings in the mounting chamber was as follows:

- excavation of the massif over such a depth as to obtain a 1.2-metre-long unsupported zone (see next paragraph) behind the shield;
- construction of the second steel ring;
- installation and expansion of the second lining ring;
- excavation of the massif over 1 metre;
- construction of the third steel ring;
- installation and expansion of the third lining ring;
- excavation of the massif over 1 metre;
- installation and expansion of the fourth lining ring. (Ring 4, built at the junction of the mounting chamber and the massif, was indeed the first lining ring directly in contact with the clay. The steel rings were thus unnecessary from then on (Figure 29 e, f, and g). The passage through the support ring was trouble free.)

The length of the unsupported zone needed behind the shield for constructing a ring was 1.2 metre (Figure 30a). Not less, since the rings were 1 metre wide, and since the jacks needed an extra 20 cm in their most retracted position, and not more, to limit convergence. Consequently, there was always a 20-cm-long section of the unsupported zone that was left unsupported for twice as long as the rest of the zone (Figure 30b). This zone had therefore more time to converge, which could be observed on the sidewalls. Similarly, the sidewalls also bore the print of the rear end of the shield whenever the shield remained in the same position for some time.



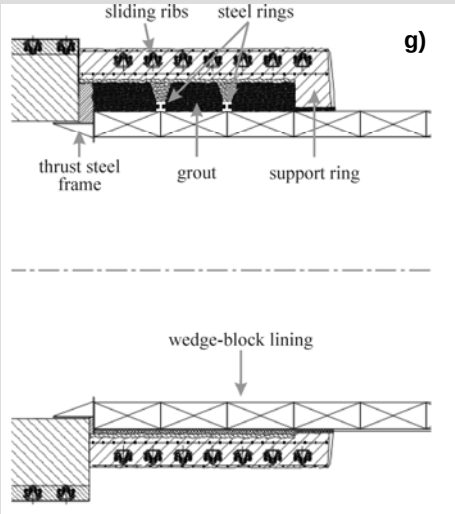


Figure 29: Construction of the secondary lining of the mounting chamber. a) The first steel ring after the thrust steel frame; b) Construction of the first lining ring, against the thrust steel frame; c) Placement of a lining segment of the first ring with the bird-wing erector; d) Start of the excavation of the concrete front of the mounting chamber with the road header; e) Transition between the primary lining of the mounting chamber and the clay; f) The first four lining rings; g) Cross section of the first four lining rings, the fourth ring being partly in the mounting chamber and partly in the connecting gallery.

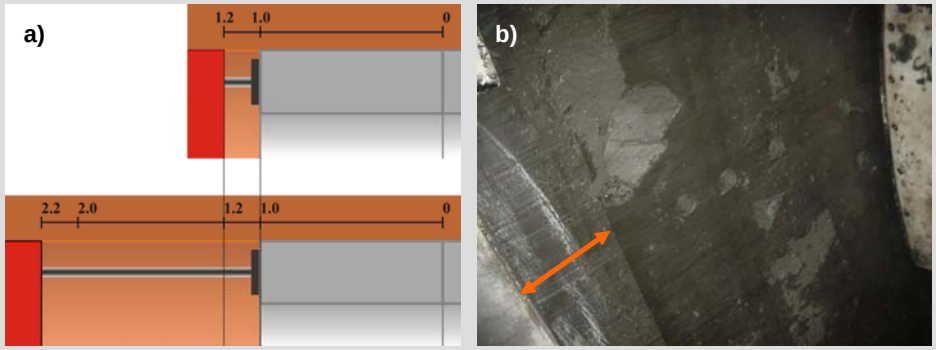


Figure 30: The unsupported zone at the back of the shield. a) A 20-cm-long section of clay was always left unsupported for twice as long as the rest of the unsupported zone; b) Close-up of the sidewall, showing differential convergence of such a 20-cm-long section.

The gaps between the secondary lining (the first three rings and part of the fourth one) and the primary lining of the mounting chamber were injected with grout after completion of the connecting gallery, through a hole drilled from the northern starting chamber through the upper part of the thrust steel frame, and after the gaps on both sides of the keys had been filled with mortar.

5.2.4. Construction of the connecting gallery in steady state

The construction of the connecting gallery (Figure 33f)—eighty-three lining rings in total, including those in the mounting chamber—, between 02.02.2002 and 09.03.2002, has progressed according to expectations. The tunnelling technique and the wedge-block technique proved to be satisfactory, though there is room for improvement (Section 10.4). The dimensions of the shield and of the lining, in particular, appeared to have been well chosen in relation to each other. Overall, the adjustments needed during the works were only minor.

Overall sequence of the works

The construction of the rings of the connecting gallery in steady state (namely excluding the construction of the secondary lining of the mounting chamber (Section 5.2.3) and the connection to the Test Drift (Section 5.3)) proceeded according to the following sequence.

- *Excavation of the clay over a distance of one metre*, the total length of the unsupported zone thus being 1.2 metre. This entailed several iterations of the following three steps:
  - excavation of the Boom Clay by the road header over a distance of 20 to 30 cm (Figure 31a), with simultaneous evacuation of the clay by the conveyor of the road header and direct loading into wagons (Figure 31b) that were pushed by hand on the rail tracks built at the rear of the tunnelling machine as it progressed;
  - forward movement of the shield, pushed by the jacks resting on the already installed lining, with simultaneous smooth trimming of the excavated profile by the cutting

head (Figure 31 c and d), the cuttings falling onto the bottom of the gallery and being then scooped up with the head of the road header. The shield could be steered if necessary by extending some or all of the jacks.

- evacuation of the cuttings.

Figure 31: Excavation of the clay. a) Excavation of the front; b) Evacuation of the clay by wagons; c) Measurement of the length of the unsupported zone; d) The smooth excavated profile.

- *Measurement of the excavated diameter and transport of the segments to the front*
  - determination of the diameter of the ring to be placed, by mutual agreement of EURIDICE and SCM Tunnel, on the basis of measurements of the diameter of the unsupported zone and of the last installed rings. The corresponding ring configuration was deducted from the decision table (Section 4.4) and had to be passed on to the surface workers at least two hours before starting the construction of the new ring, to enable them to start transporting the necessary segments. It was very important to know well enough in advance the required diameter, since the limited underground workspace and the limited possibility of extra transports with the lift would have made it very difficult to bring changes once the segments had been transported to the bottom of the shaft.
  - filling of the large cavities, if any, in the excavated profile with wood or cement bags (sometimes also done while placing the segments) (Figure 32a);
  - trimming of the sidewalls if necessary (Figure 32b);
  - application of soap on the excavated profile, just before the placement of each segment, and after drawing in the corresponding jacks (Figure 32c);
  - transport of the segments from the surface on purpose-built carts designed to carry two segments each (Figure 32 d and e). These carts were pushed manually. The

transport sequence had to match the placement sequence, namely segments 6 and 7 (left rail track), 5 and 4 (right rail track), 8 and 11 (key) (left rail track), 12 (key) and 3 (right rail track), 9 and 10 (left rail track), and 1 and 2 (right rail track) (Figure 32f).

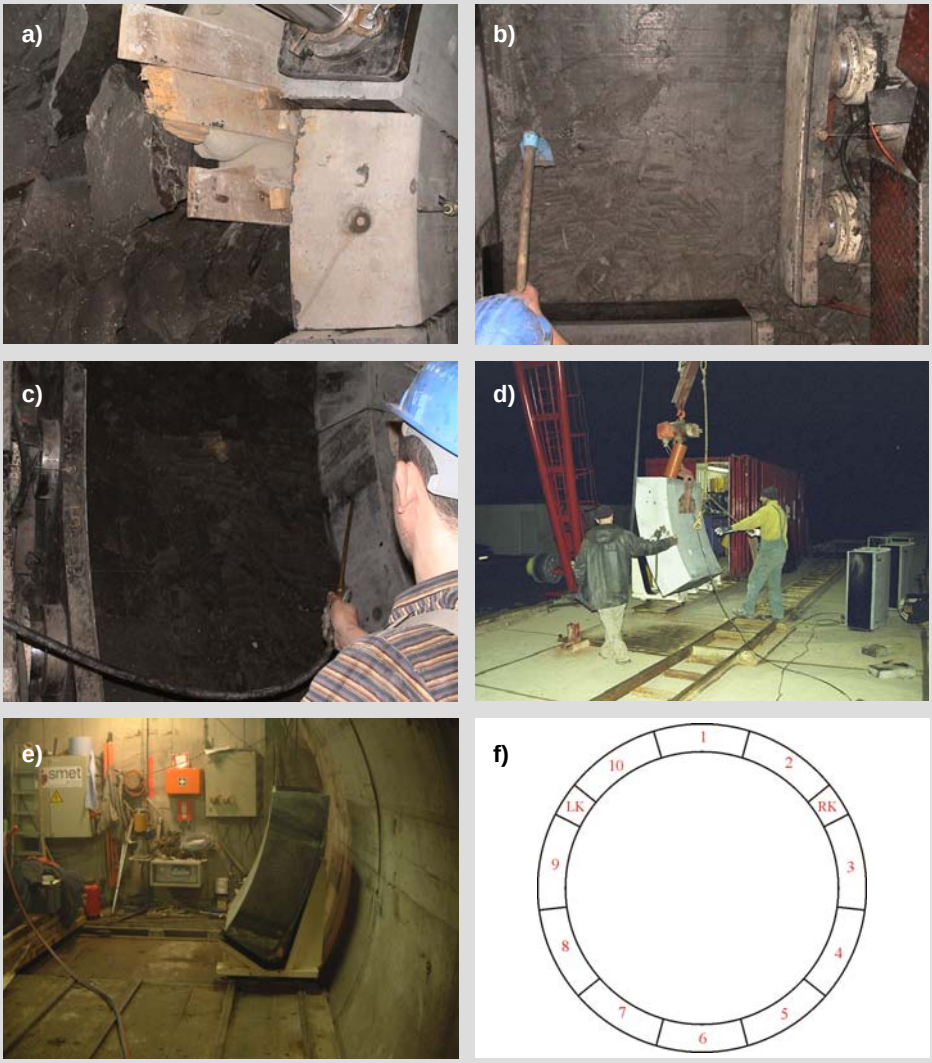


Figure 32: Preparation of the construction of a lining ring. a) Filling of large cavities with wood; b) Trimming of the gallery sidewalls; c) Application of soap on the excavated profile; d) Segment manipulation on surface; e) Segment cart; f) Segment numbering, looking towards the Test Drift.

- *Placement of the segments*
  - lifting of segment 6 from the cart by one wing of the erector, using a triangular steel plate equipped with two fingers designed to grab the segments in their manipulation holes;
  - placement of segment 6 by the erector on the bottom of the gallery, the segment being pulled into place via the steel plate and fingers of the other wing (Figure 33a),

and then held in place against the corresponding segment of the previous ring by the thrust pressure of the jacks;

- repetition of these first two steps (using only one erector wing, though) for segments 7, 5, 4, and 8;
- pulling of the two keys (11 and 12) in the pockets of the shield (see also Figure 22g);
- placement of the two segments directly under the two keys (3 and 9) (Figure 33b);
- partial insertion of the two keys;
- placement of segments 10, 1, and 2, followed by the placement of the anti-fall system for each of the three segments (Figure 33 c, d, and e);
- full insertion of the two keys up to their final position, with expansion of the ring.

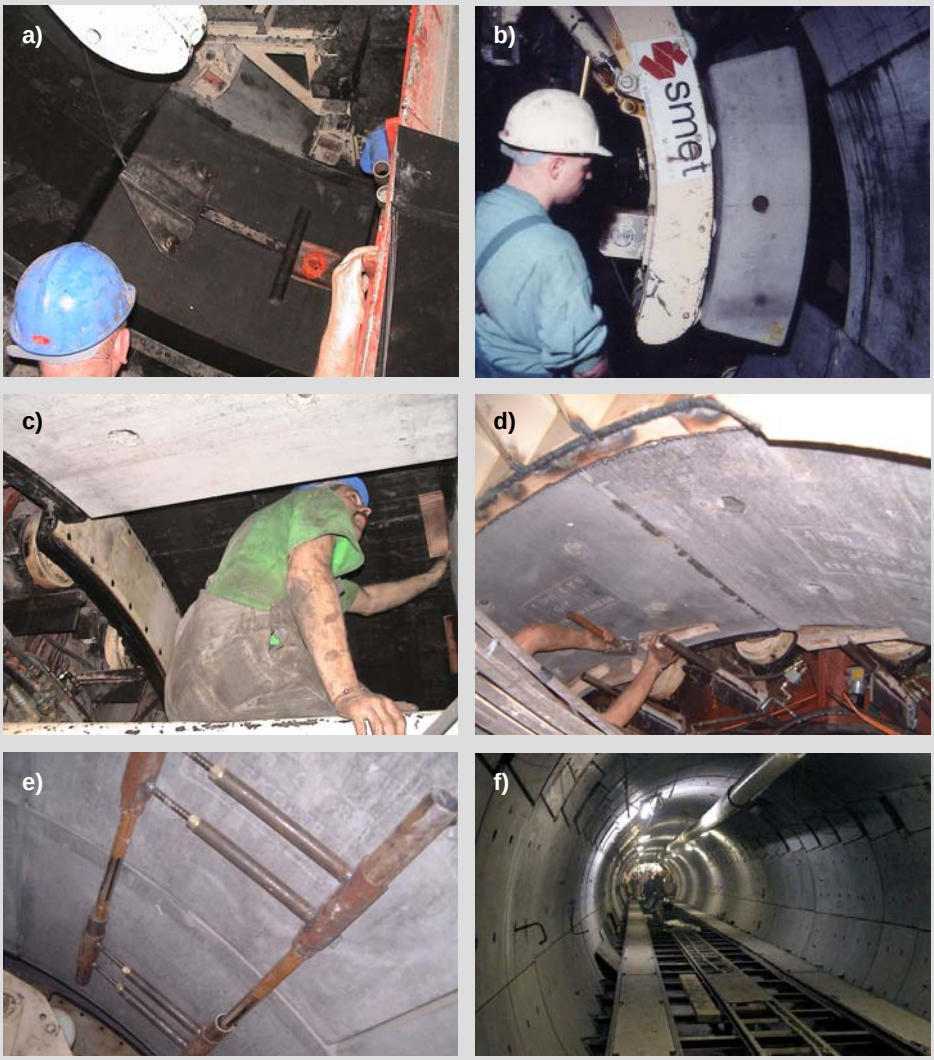


Figure 33: Placement of the lining segments. a) Placement of segment 6 with the help of the triangular steel plate; b) Placement of segment 3; c) Segment held in place by the corresponding jacks; d) and e) The two parts of the anti-fall system, one being attached to the shield, the other to the previous ring; f) General view of the connecting gallery.

Lambert coordinates
A geographic system of coordinates of latitude and longitude, using the Lambert projection system.

A form with the following type of information was filled out for traceability for all rings: position and orientation of the shield just before ring construction, time of construction, type of keys used and their depth of insertion, production date of each segment and mould used, orientation of the ring with respect to the shield, and diameters (Section 6.5).

Ideally, the intrados of each lining ring should have been perfectly smooth. However, misalignments of up to 5 cm were reported, mainly due to the many cavities in the excavated profile (Figure 34) (see hereafter "Problems encountered and their solutions" and Section 6.2.2). These cavities, which have been described qualitatively for each ring, made it very difficult to build smooth rings and were likely to have an influence on the external pressures on the lining in the short and long terms. After completion of the gallery, the steps between the upper segments of all the rings of the gallery were measured. (The working floor did not allow easy measurements in the lower half.) A topographic survey was then performed in December 2002 and January 2003 to measure the exact position of each segment (namely, the Lambert coordinates of six points per segment, determined with an accuracy of about 1 mm).

Figure 34: Misalignments between segments. An injection point for filling the cavities between the lining and the massif is also visible (see hereafter "Problems linked to the clay massif").

The achieved construction rate complied with the technical specifications in steady state (minimum average construction rate of 2 metres every 24 hours), since it was between 2 and 4 metres each day (Figure 35). The limiting factor was the transport of the excavated clay to the surface. Safety rules prohibited indeed the simultaneous transport of personnel and materials or equipment in the shaft, and the useful capacity of the lift was limited to 3.8 tons. Furthermore, the carts transporting the segments derailed frequently, especially towards the end of the works.

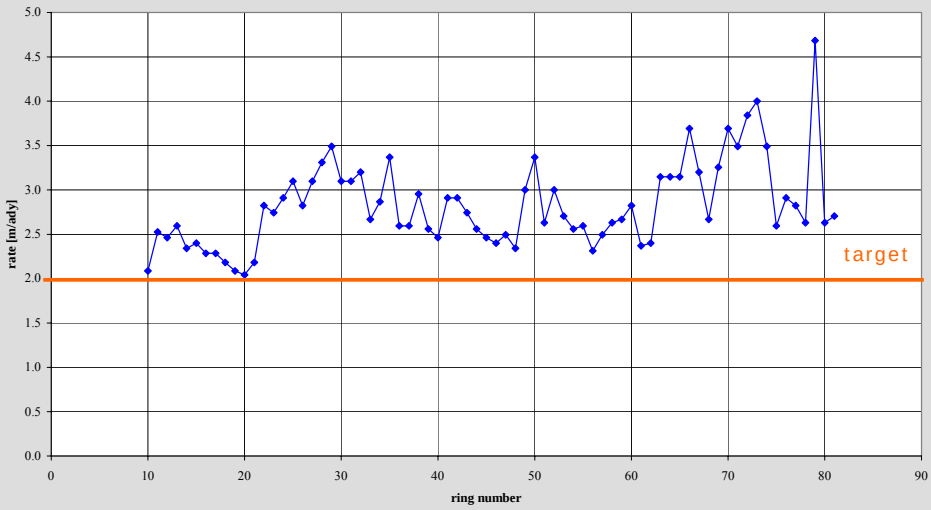


Figure 35: The construction rate (based on SCM Tunnel data) was consistently above the target construction rate of 2 metres per 24 hours.

Problems encountered and their solutions

Most of the problems encountered during the construction of the connecting gallery received a satisfactory solution on the spot.

Problems linked to the clay massif

Safety issue and construction issue due to the large clay blocks coming off from the massif Excavation-induced fractures (Section 6.2) caused large blocks of clay to come off from the excavation front, thereby also inducing cavities in the roof and sidewalls.

> *Remedy* (i) To excavate the upper part of the front (over 0.5 to 1 metre) before its lower part, which reduced the number of blocks coming off and, subsequently, the number of cavities, (ii) to remove loose blocks and, (iii) whenever necessary, to place timber in the upper part of the gallery, between the hydraulic jacks and the roof or the sidewalls (Figure 36). (Risks to workers near the front were limited to when they had to repair the road header.)

Presence of cavities in the roof and in the sidewalls (usually in their upper part)

> *Remedy* (i) Before placement of the lining ring, to fill the large cavities with pieces of wood or cement bags (Figure 32a), the very small cavities being no problem. The force needed to insert the keys was kept relatively low when large cavities were present, to prevent the segments from sinking into them. (ii) After placement of the lining, to inject the remaining cavities with grout after sealing the joints between the segments making up the corresponding rings with silicone, to prevent the grout from entering the gallery.

Figure 36: Timber placed between the jacks and the sidewall to prevent clay blocks from coming off.

Problems linked to the shield and the road header

Broken teeth and tooth-holders of the road header After the excavation of the concrete front of the mounting chamber, the original head of the road header (Figure 37a) was replaced by one specially designed for excavation in clay (Figure 37b). The Boom Clay contains indeed pyrite (FeS_2) and septaria layers at the level of the HADES facility (Section 6.3.1). Though the septaria layers, the septaria nodules—no large nodules were encountered—, and the thin pyrite layers (about 1 cm thick) were no problem, the big pyrite concretions of up to 30 cm in diameter that were also encountered were a source of problems: if not noticed in due time and attacked with full force by the head of the road header, such concretions caused teeth and tooth holders to break (Figure 37 c and d). New holders had then to be welded onto the head, and this was a relatively dangerous operation, since it had to be carried out near the front, where clay blocks sometimes came off (Figure 37e). (The road header could be moved backwards, but only on less than one metre.)

Remedy To try to avoid attacking the pyrite concretions with the head of the road header, and to remove them by hand instead after having removed the clay around them.

Broken metal scrapers The conveyor transporting the clay from the excavation front to the back of the tunnelling machine got damaged too: the metal scrapers broke because of pieces of concrete from the front of the mounting chamber, pieces of pyrite, or simply wear (Figure 38).

Remedy To replace the scrapers.

Need to place the target for the laser beam manually at the front of the shield (Figure 39) *and to use a plumb line to check whether the shield was still horizontal*

Remedy None at the time.

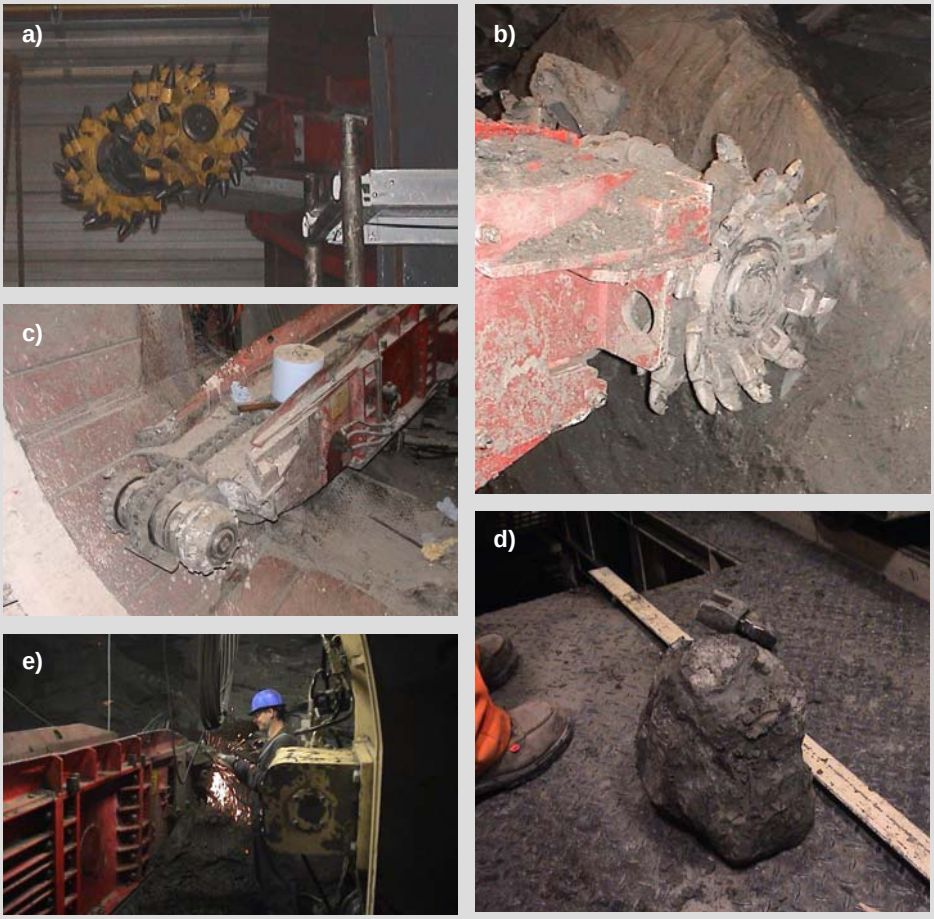


Figure 37: Head of the road header. a) Original head, used to excavate the concrete front of the mounting chamber (see also Figure 29d); b) Head for clay; c) Same head with almost no teeth anymore; d) Pyrite concretion with a tooth-holder (and a tooth) that broke on it; e) Welding work on the head.

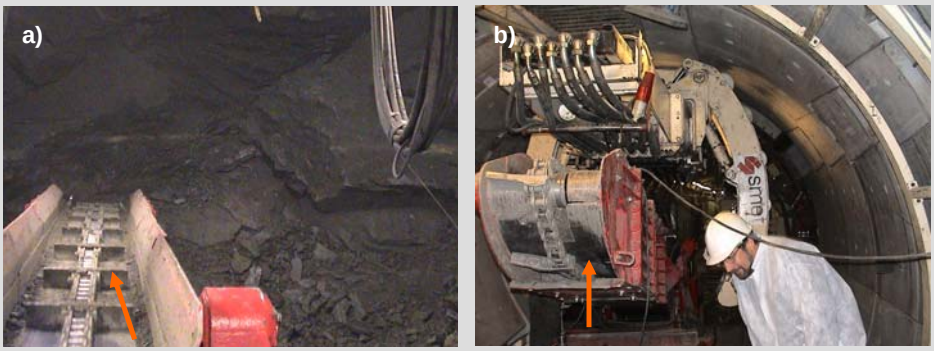


Figure 38: The conveyor transporting the clay from the excavation front to the back of the tunnelling machine. a) Intact metal scrapers; b) Two partly broken metal scrapers at the rear end of the conveyor.

Impossibility to use the target placed near the front of the shield during its forward movements as foreseen, since pieces and blocks of clay were then falling down onto it

Remedy To place a small laser source on the rear end of the shield and a target on a recently built ring (Figure 40).

Impossibility to adapt the jack pressures gradually during the forward movements of the shield
Though steering was possible by setting up the pressure of each pair of jacks separately, the valves used to regulate the jack pressures, which were in the shield, could not be reached while it moved.

Remedy To switch off specific pairs of jacks temporarily. Corrections could not be too brusque, though, to avoid creating "steps" on the sidewalls (Figure 41), which made the placement of the lining more difficult. Brusque corrections also often meant "over-corrections" and, hence, the need for a quick correction in the other sense. (This problem was encountered in the first metres of the connecting gallery.) Steering to the left or to the right proved to be more difficult than steering upwards or downwards. This observation was consistent with the fact that the horizontal convergence exceeded the vertical convergence in the unsupported zone behind the shield (Section 6.5).

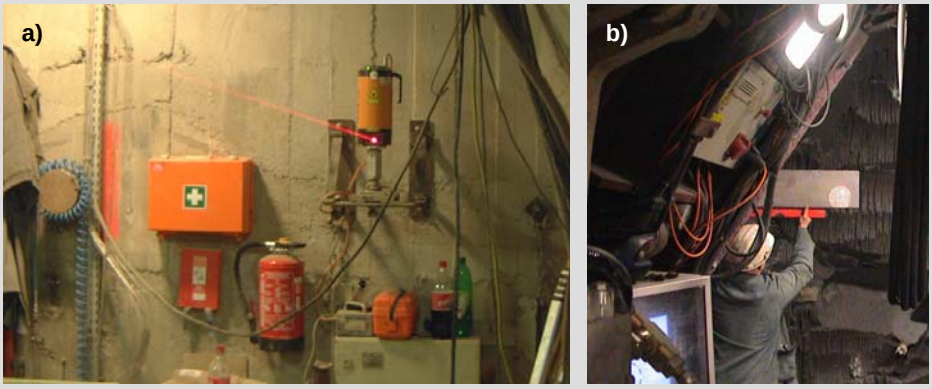


Figure 39: Laser source in the southern starting chamber (a) and target manually placed near the front of the shield (b), used to check the shield direction.

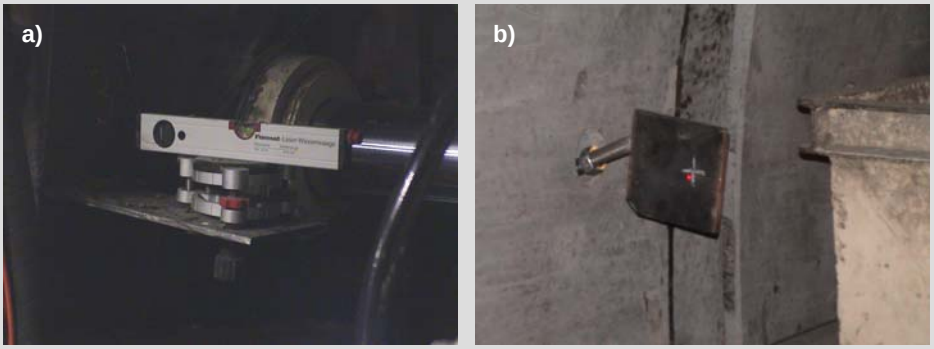


Figure 40: Laser source (a) and target (b) used during the forward movements of the shield.

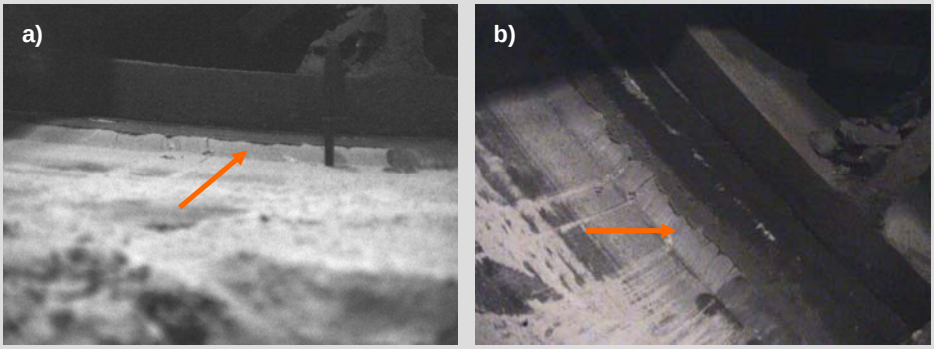


Figure 41: Step on the sidewall, about 5 to 10 mm high, due to a brusque steering correction.

Slight roll of the shield during excavation

Remedy To fix a lining segment on one wing of the erector as a counterweight to compensate for the roll. (Should more problems have arisen, another solution foreseen was to mount the jacks slightly out of their axis, in such a way that pushing them forward would have induced a torque compensating for or correcting the roll.)

Problems linked to the bird-wing erector

Non conform design of the erector The initial design of the bird-wing erector, by the English company Specialist Plant Associates Ltd, on behalf of SCM Tunnel, did not fully comply with the Belgian legislation. On the one hand, the required safety factor of the steel cables was not met: they were designed for the weight of one segment, whereas the winch could provide more force and could thus break them if a segment got stuck. On the other hand, the design did not consider dynamic loads.

Remedy To use cables with a larger diameter.

Breaking cable wires The cables that were used initially, which were both left hand lay and right hand lay, featured broken wires, probably because they were not flexible enough, and had therefore to be replaced.

Remedy To use two different cables for the left and right winch (a left hand lay one and a right hand lay one).

Insufficient diameter of the cable drums The diameter of the cable drums at the extremities of the erector wings was too small with respect to the diameter of the cables themselves, given the fact that these cables had to make almost a 180° turn (Figure 42a) and that their diameters needed to be increased.

Remedy To increase the diameter of the cable drums (Figure 42b).

Need to replace some of the erector jacks The fact that the four hydraulic jacks (Figure 43) moving each erector wing outwards were connected to the same oil duct caused problems when they were not all submitted to identical loads. This was in particular the case when there

were cavities in the roof or in the sidewalls, since the segment would then tilt partially around the edge of the cavity.

Remedy None at the time.

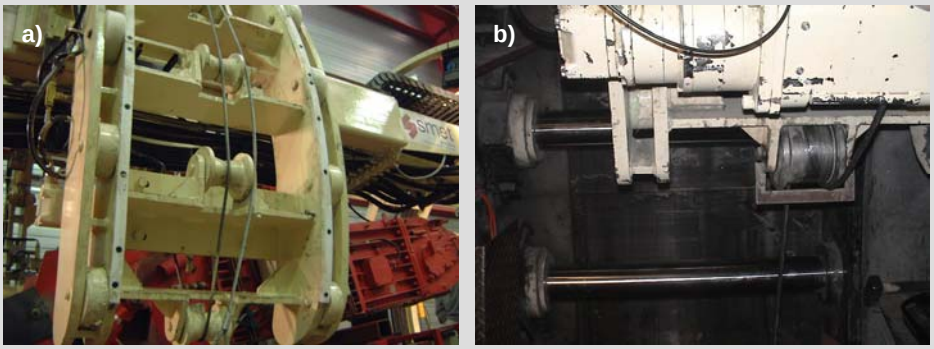


Figure 42: The cables and drums on the erector. a) The original cables and cable drums used during the test assembly; b) Larger-diameter cables and larger bottom drums were used during the actual construction works.

Figure 43: The hydraulic jacks used to move the erector wings. The thrust steel frame is also visible.

Problems linked to the lining

Risk to see keys being squeezed out of their ring by the neighbouring segments

Remedy To fix each key to the previous one, the keys of ring 1 being fixed to the thrust steel frame (Figure 44 a and b). Each time a new ring was built, the keys of the previous ring were held in place by wooden wedges placed between them and the below segments of the new ring (Figure 44c). The rods securing the keys of the previous ring were then detached and moved one ring further, to secure the new ring to this previous one.

Risk to damage segments when pushing the shield forward Since it was impossible to align the segments perfectly with respect to each other, even with the stringent production tolerances placed on them, some segments got damaged when the shield was being pushed forward against the lining (Figure 45a). There also seemed to be some correlation between these damages and both the presence of cavities in the sidewalls and the fact that the pressure used to insert the keys was low. All the damaged segments and the types of damage were listed up.

Marks were placed at the tip of the fissures, to enable their propagation to be followed up with time, this follow up being a monthly visual control. No disturbing fissure growth has been reported between completion of the gallery and the end of 2003.

Remedy To insert high-density polyethylene plates between segments of adjacent rings (Figure 45b).

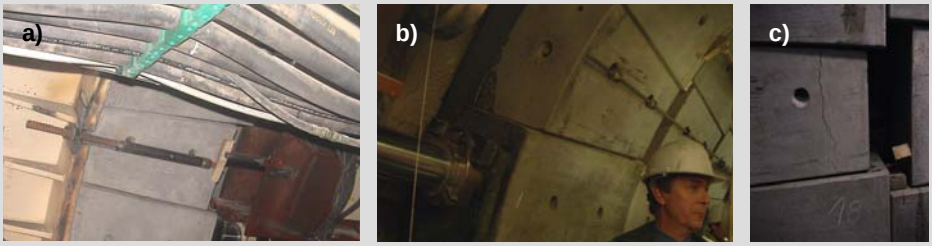


Figure 44: Methods used to prevent the keys from being squeezed out of the rings. a) Key of ring 1, fixed to the thrust steel frame; b) Subsequent keys, always secured to the previous one; c) Key of the previous ring held in place by wooden wedges.

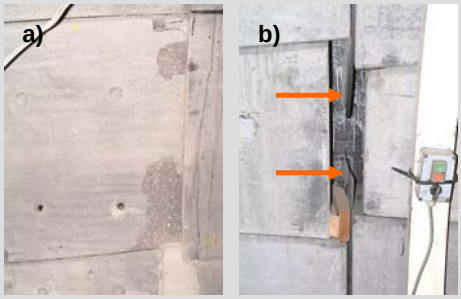


Figure 45: Damaged segment (a) and plates of high-density polyethylene inserted between adjacent segments to avoid damages (b).

5.3. Connection to the Test Drift

The connection to the Test Drift has been trouble free and proceeded as follows. Instead of stopping the shield when it reached the Test Drift, the last two rings of the Test Drift lining were removed (2/3 of a metre), which enabled an extra wedge-block ring to be placed behind the shield, thereby minimising the distance between the rear end of the shield and the last ring of the connecting gallery. The final position of the shield coincided almost with its intended position (Figure 71). The shield was not dismantled, but was used instead as lining for the last few metres of the gallery, where it also serves as witness of the past works, both for members of the public and for specialists. The jacks and some parts of the hydraulic system were left behind too, whereas other reusable parts, such as the hydraulic ducts, the electronics, and the erector were removed.

Since the front diameter of the shield (4 890 mm, including the oversize) was 190 mm larger than the outer diameter of the Test Drift lining, there is apparent Boom Clay at the interface between the connecting gallery and the Test Drift (Figure 46 a and b). A custom built 200-mm-wide steel ring was installed in the final narrow unsupported zone (Figure 46 c and d). Finally,

the annular gap between the shield and the massif was filled with grout identical to that previously used to fill major cavities in the roof and sidewalls. This grout was injected through the holes foreseen in the shield for measuring the instantaneous convergence (Section 6.4.2).

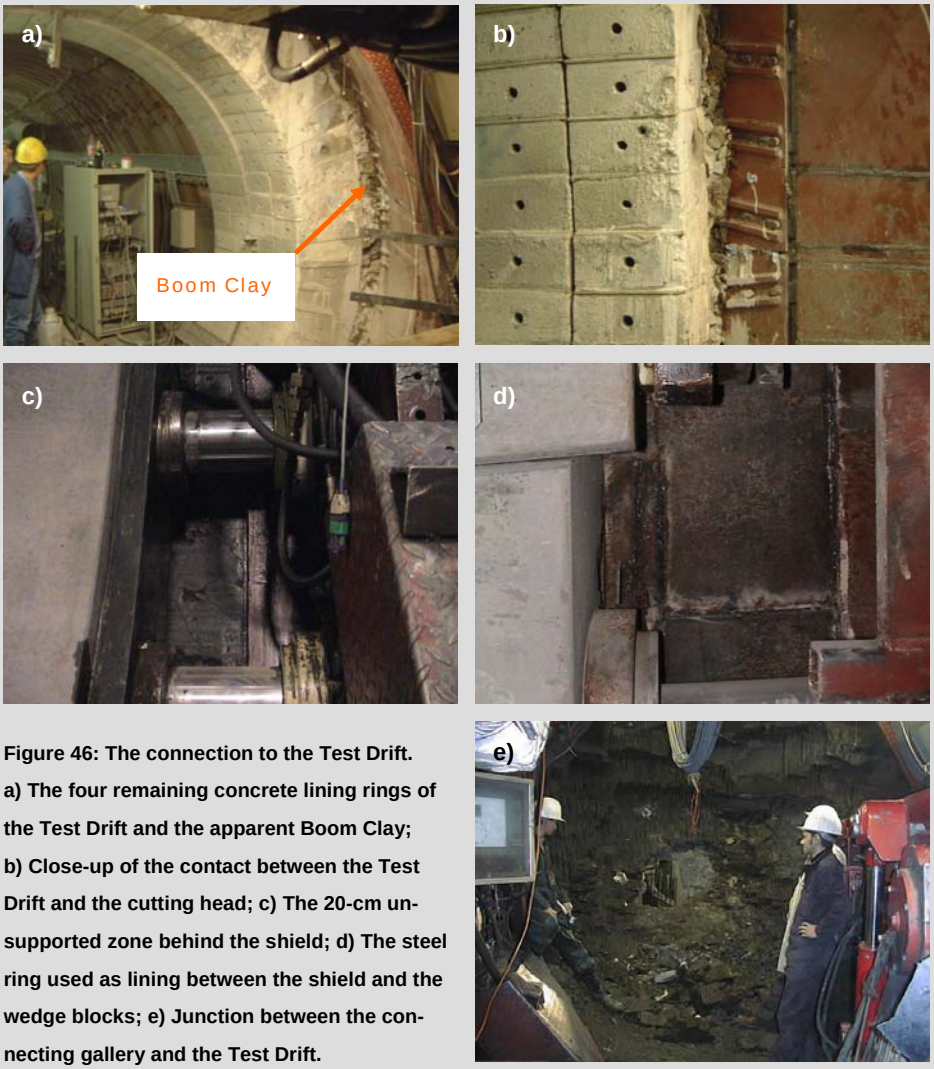


Figure 46: The connection to the Test Drift. a) The four remaining concrete lining rings of the Test Drift and the apparent Boom Clay; b) Close-up of the contact between the Test Drift and the cutting head; c) The 20-cm unsupported zone behind the shield; d) The steel ring used as lining between the shield and the wedge blocks; e) Junction between the connecting gallery and the Test Drift.

Although one could have expected problems with the stability of the front (or at least a change in stability) during the last metres before the connection to the Test Drift, its stability remained almost unchanged: it even improved slightly. This improvement was probably due to a redistribution of the stresses (a kind of arching effect between the Test Drift lining and the gallery lining and shield) and to an anchoring effect of the CLIPEX instrumentation tubes (Section 6.1). The front did not collapse spontaneously when it had become less than half a metre thick and when there was already partial connection (Figure 46e). Signs of oxidation were only seen in the last metre of clay (Figure 63). The construction of the connecting gallery increased however the pressure on the extremity of the Test Drift (Section 6.5).

6. Results of the measurement and research programmes

Several measurement and research programmes were defined, to be carried out during and after the extension of the underground research facility HADES. They aimed to gain as much information as possible on the behaviour of the clay massif and on its impact on the lining. This information was to be gained through observation, measurements on the massif and on the lining, and measurements on the tunnelling machine. Together with the modelling work that was and is still being performed, these data were to contribute to understanding better the behaviour of the massif, to improving the tunnelling technique, and to preparing the installation of future experiments in the connecting gallery. They have provided EURIDICE with a deeper insight into the response of the Boom Clay to geomechanical disturbances and, in particular, to an excavation using an industrial tunnelling technique.

6.1. CLIPEX programme

The European CLIPEX programme (Clay Instrumentation Programme for the Extension of an Underground Laboratory) aimed at a better understanding of the instantaneous hydromechanical response of the Boom Formation to excavations, through observation and prediction of the reactions induced in the clay by the excavation of the connecting gallery. The excavation progressing from the second shaft to the existing facility, it provided indeed EURIDICE with a unique and original opportunity to monitor hydromechanical parameters ahead of the advancing front.

An initial characterisation programme had already been carried out. It had been designed to obtain or validate several hydromechanical parameters necessary for blind predictions of the behaviour of the massif.

The subsequent blind prediction exercise was based on the assumed excavation parameters (excavation rate and sequence, excavation radius, and lining radius) and on various rheological models. Each modelling team (four in total) had had to provide the evolutions of the hydromechanical response of the massif to the excavation of the connecting gallery as a function of the position of the excavation front. The effect of the pre-existing infrastructures (second shaft and Test Drift) had been taken into account as well; the presence of the mounting chamber had not.

The experimental part of the CLIPEX programme consisted mainly in acquiring and monitoring pressure and displacement data. Three main zones were instrumented (Figure 47):

- *the zone behind the front of the Test Drift* (in May 1998)
  - four boreholes (A1 to D1), instrumented to monitor displacements
    - The *extensometer* (in borehole A1, aligned with the axis of the future connecting gallery) consisted of six anchors connected to displacement transducers located at the Test Drift front, installed in a 30-metre-long borehole. The transducers had a measuring range of 200 mm.
    - The two *inclinometers* (in borehole B1, located just above the gallery, almost parallel to it, and in borehole C1, located above the gallery, at a larger angle

with it) were chains made up of ten 3-metre-long beams, each one of them equipped with a uni-axial tilt-sensor having a measuring range of ± 3° and installed in a 30-metre-long cased borehole.

  – The *deflectometer* (in horizontal borehole D1, at an angle with borehole A1) was a chain made up of nine 3-metre-long beams and two end beams of 1.5 metre each, installed in a 30-metre-long cased borehole. A biaxial sensor between each pair of beams measured the vertical and the horizontal angles between the beams, in a range of ± 3°.

- four boreholes (A2 to D2), instrumented to monitor pore water pressure and total pressure
  - The *pore water pressure* was measured by piezometric filters, in 6 locations in each borehole (8 locations for C2), at depths of up to 30 metres. These filters were stainless steel filter cartridges integrated in the borehole casing. The pore water collected was brought to the pressure transmitters at the Test Drift front. The piezometers also enabled water sampling and measurements of other hydraulic parameters, such as permeability.
  - The *total pressure* was measured by miniature total pressure sensors: diaphragms mounted almost flush with the outer surface of the borehole casings. There were 4 to 9 of them in each of the four boreholes.

- *a zone near the bottom of the second shaft* (above the future gallery and parallel to it)
  - one borehole (E1, June 2001), instrumented to monitor displacements, located at about 5 metres above the axis of the future connecting gallery;
  - one borehole (E2, October 2000), instrumented to monitor pore water pressure, located at about 6.5 metres above the axis of the future connecting gallery.

- *lining segments in the connecting gallery*
  - strain gauges, embedded in the 30 lining segments of three different rings (Section 4.3). Each segment contained 6 or 12 sensors, meant to measure intrados and extrados strains separately and, hence, to provide indications on the interactions between the lining and the massif and elements for revising, if appropriate, the number and the size of the openings that will be allowed in the non-reinforced lining.

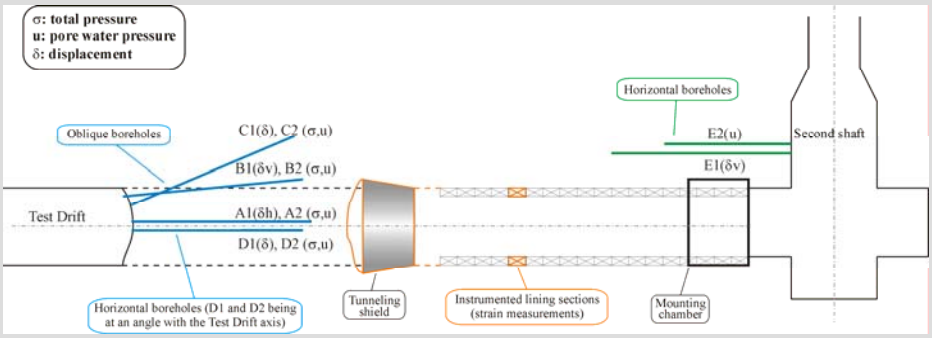


Figure 47: Location of the instrumentation in the CLIPEX programme.

The excavation of the connecting gallery had to provide for the careful retrieval of the pieces of the CLIPEX instrumentation that were in the way of the shield as it approached the front of the Test Drift, and piezometers B2, C2, and D2 had to be reconnected to the data-acquisition system after the construction of the gallery.

The performance of the sensors of the CLIPEX programme ranged from excellent to bad: the pore water pressure sensors and the strain gauges embedded in the lining segments performed as expected, some problems were encountered with the displacement sensors, and the total pressure sensors performed poorly: they recorded indeed values that were almost identical to those recorded by the pore water pressure sensors.

6.1.1. Mounting chamber

The effects of the excavation of the mounting chamber have been clearly followed up with the CLIPEX instrumentation. They were similar to the effects observed later on when starting the excavation of the connecting gallery (Section 6.1.2).

- The *pore water pressure* in the massif varied immediately after the start of the excavation of the mounting chamber (Figure 48): the closest filters showed a reduction of the pore water pressure, whereas the more distant ones showed an increase. Regarding (total) stresses, two main effects are indeed present when excavating. On the one hand, excavation always entails a certain decompression (lower total stresses and lower pore water pressures) of the surrounding massif. On the other hand, further into the massif, total stresses increase due to stress redistribution. Pore water pressures also increase, due to the combined effects of the compression of the clay, which tends to expel the water, and of the low permeability, which makes this phenomenon very slow. Both effects have been observed. The filters up to 15 metres showed a decrease of the pressures (decompression); deeper filters showed an increase (stress redistribution). The filter at 15 metres experienced both phenomena: the pressure increased slightly, before decreasing after further excavation. After the excavation of the mounting chamber, the evolution of the pore water pressures resumed its previous (increasing) trend. This increasing trend before and after the excavation of the mounting chamber is due to reconsolidation effects after the decompression caused by the construction of the second shaft, to the installation of the piezometer itself, and to the collapse of an adjacent borehole. The low values of the pore water pressures measured before excavation (the in situ undisturbed pore water pressure is about 2.2 MPa) and the gradient (deeper filters show higher pressures) are caused by the presence of the second shaft.
- The *settlement of the clay* above the future connecting gallery followed the excavation (Figure 49). The sensors in the first 2 metres from the shaft intrados, located above the starting chamber, showed limited settlement. The sensors at 4 and 6 metres, located above the mounting chamber, and those at 8 and 10 metres, located just beyond the end of the mounting chamber, showed larger settlements (up to about 25 mm) during excavation of the mounting chamber. The sensors further away from the mounting chamber showed less settlement.

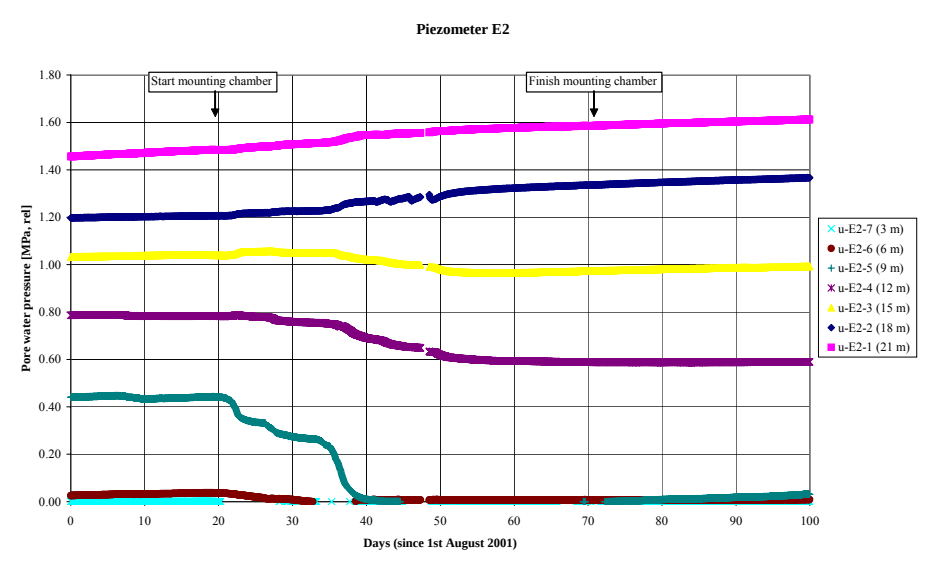


Figure 48: Evolution of the pore water pressure before, during, and after the construction of the mounting chamber (piezometer E2, at about 6.5 metres above the axis of the future connecting gallery). The positions of the filters are measured from the intrados of the second shaft. The 3.75-metre-long mounting chamber is located between about 3.5 and 7.25 metres from the intrados.

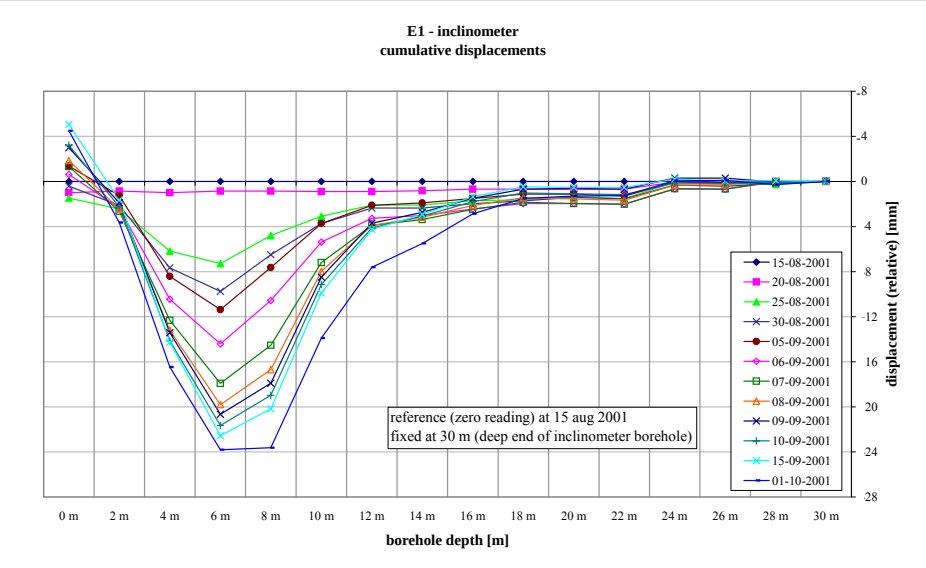


Figure 49: Vertical displacements at the beginning of the excavation of the mounting chamber, located between about 3.5 and 7.25 metres from the intrados of the second shaft (inclinometer E1, at about 5 metres above the axis of the future connecting gallery). The positions of the sensors are measured from the intrados of the second shaft.

- The *total pressure sensors* that had been installed between the lining of the northern starting chamber and the massif and that, strictly speaking, were not part of CLIPEX, also showed the redistribution of the total pressure in the massif (Figure 50). This redistribution resulted from the decompression around the excavation front. The recorded values differed, since it is impossible to install the sensors in a fully reproducible manner. (The comparatively low pressure measured by sensor N1 can be explained by the fact that this sensor was installed about 1 metre closer to the shaft than the other three sensors.) All sensors showed a rather sharp pressure increase as a result of both the construction of the mounting chamber and the construction of the connecting gallery. With time, the recorded pressures tended to stabilise at a higher value. This evolution of the total pressure reveals the creep of the massif.

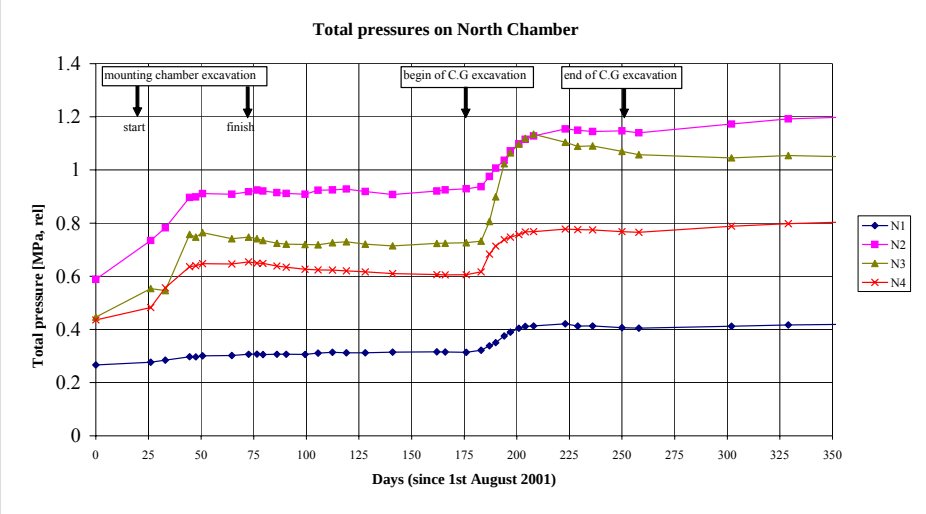


Figure 50: Readings of the total pressure sensors that were installed behind the lining of the northern starting chamber (N1: eastern sidewall, half height; N2: roof; N3: western sidewall, half height; N4: bottom).

6.1.2. Connecting gallery

The new data obtained during the construction of the connecting gallery have evidenced various characteristics of the behaviour of the Boom Clay. Comparison of these data with the blind prediction results indicated that the hydromechanical coupling behaviour of the Boom Clay is very complex.

Strong hydromechanical coupling

The measurements during the construction of the connecting gallery, especially those of the evolution of the pore water pressure, have clearly shown the strong hydromechanical coupling behaviour of the Boom Clay.

- All sensors installed behind the front of the Test Drift registered a similar evolution of the pore water pressure: a progressive increase, followed by a sharp drop as the excavation front approached very closely (Figure 51). A tendency towards re-equilibrium was then registered by the sensors that were reconnected after excavation (piezometers C2 and D2). Although the pressure increase had been predicted, its extent was larger than expected.

  According to the elasto-visco-plastic theory, the variation of the pore water pressure during the underground excavation is linked to the volumetric deformation of the massif via the coupling effect. The increase of the pore water pressure was most likely due to the undrained contractant behaviour of the clay. Stresses increased ahead of the excavation front, compressing the clay (decrease of the pore volume). Since the clay has a very low permeability, the water could only be expelled very slowly, and the pore water pressure increased. The drop phenomenon close to the front resulted from the high decompression of the massif and from the resulting fractures and volumetric dilatations (increase of the pore volume).

  As a corollary, the measurements of the pore water pressure carried out outside of the zone of influence of the HADES facility—a première—allowed the pore water pressure of the undisturbed clay at about 225 metres depth to be confirmed to be about 2.2 MPa.

- A large deformation of the massif (δv-E1) induced a strong variation of the pore water pressure (u-E2) (Figure 52). The sharp drop of the pore water pressure (dilatant behaviour) reflects the high decompression caused in the massif by the close proximity of the excavated profile.

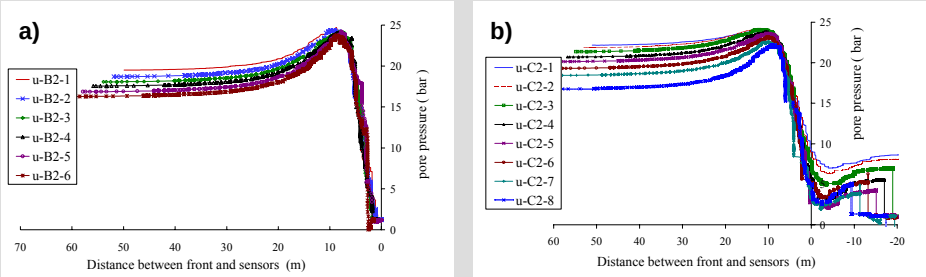


Figure 51: Evolution of the pore water pressure with the distance to the excavation front. a) Piezometer B2, just above the gallery; b) Piezometer C2, at a larger angle above the gallery.

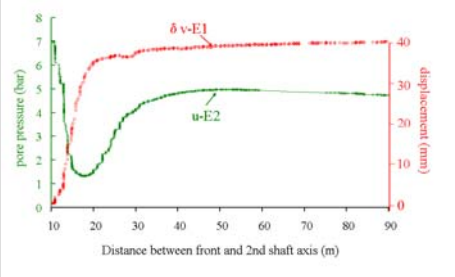


Figure 52: Evolution of the pore water pressure and of the displacements around the connecting gallery, measured above the gallery, at 12 metres from the second shaft intrados.

Furthermore, displacement measurements near the bottom of the second shaft showed that the settlement in the clay massif followed closely the gallery excavation (Figure 53).

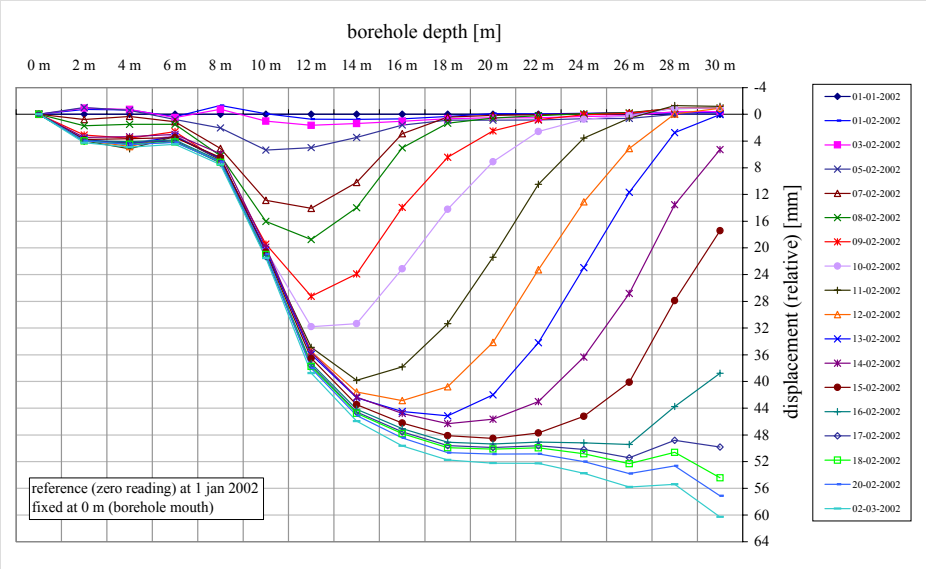


Figure 53: Evolutions of the vertical displacements above the connecting gallery both during and after excavation (inclinometer E1). The shape of the plots reflects the evolution of the deformation of the inclinometer with time.

Presence of fractures around the connecting gallery

The presence of fractures around the connecting gallery (Section 6.2.2) has been confirmed by the following observations.

- *Sudden re-equilibrium of the pore water pressure with the atmospheric pressure* The suction created at about 3 metres ahead of the excavation front (Figure 54a) because of the strong hydromechanical coupling has then been followed by an abrupt recovery of the pore water pressure up to the atmospheric pressure as the front was coming closer. This sudden re-equilibrium corresponded with the sudden connection of the gallery with the fractures in the massif ahead of the excavation front up to a distance of about 2 to 3 metres along the gallery axis. (A similar phenomenon had been observed in another experiment, the RESEAL in situ shaft-sealing test, during removal of the RESEAL shaft lining.) The fractures are actually formed further away from the front, at about 6 metres (Section 6.2.2).
- *Decrease of the pore water pressure during the installation of the lining (when the excavation front was close to the sensors)* This ongoing evolution (Figure 54b) was mainly related to the highly disturbed state of the massif (fracturation, volumetric dilatation, etc.).

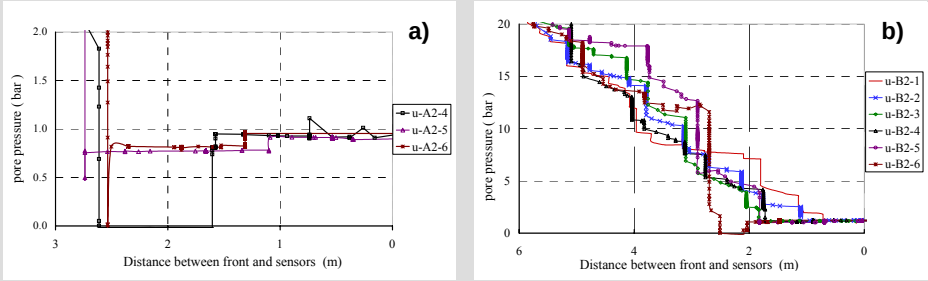


Figure 54: Evidences of the presence of fractures. a) Sudden re-equilibrium of the pore water pressure with the atmospheric pressure as fractures connect with the gallery (axial piezometer A2); b) Decrease of the pore water pressure during the installation of the lining (when the excavation front is close to the sensors) (piezometer B2, just above the gallery).

Time-dependent behaviour

The strains measured in the three instrumented lining rings have allowed the pressure build-up on the gallery lining to be estimated, after determination of the Young modulus and of the creep deformation.

- The strain-stress relationship of the concrete has been derived from laboratory tests by the Magnel Laboratory. The average value of the Young modulus, as determined by the secans method on four cylindrical samples, was 49.2 GPa. The tested samples were then loaded till failure, which led to an average compressive strength of 110 N/mm^2 (minimum of 104 N/mm^2), namely a value that confirmed the 95 percentile of the compressive strength previously determined on cubic samples: 91.3 N/mm^2 (Section 4.3).
- Creep tests were carried out on the concrete of the segments to enable one to distinguish the creep deformation in the total deformation measured by the strain gauges. The creep tests carried out on 2 samples showed that corrections of up to 20% had to be applied to the strain gauge readings from the instrumented segments.
- The external pressure was then calculated for each segment, assuming a perfect ring with nominal dimensions, assuming that the average strain in that segment was the strain in the whole ring, and taking the creep correction into account. This led to values comprised between 2.7 and 3.6 MPa at the beginning of June 2003 (Figure 55).
- Finally, the strain measurements were analysed in depth by BELGATOM. The actual ground pressure on the lining was calculated combining the strain information and the relative position of each lining segment, calculated on the basis of the topographic survey of the connecting gallery (Section 5.2.4). This analysis concluded that the ground pressure was comprised between 2.1 and 3.1 MPa on 06.05.2003 (Figure 55). This result was then used to calculate again the maximal stress in the segments, using the same method as the one used for the original design of the gallery. The maximal stress (22.98 MPa) is well below the allowable stress (36.83 MPa).

The increase of the external pressure exerted by the Boom Clay on the lining segments since their installation shows the time-dependent behaviour of the clay through hydraulic diffusion processes and viscosity of the skeleton. In reality, due to the structure of the lining, the clay wall can be considered as a drained hydraulic boundary. The reconsolidation of the Boom Clay caused by both the drainage of the pore water and the creep of the massif results in an increase of the pressure on the lining. This pressure builds up quite rapidly and to a relatively high value compared to the lithostatic pressure (4.5 MPa). Such high value indicates that the construction of the connecting gallery has disturbed the clay a lot less than the excavation and lining of the second shaft: external pressure measurements on the shaft lining are indeed only of the order of 1 MPa after 4 years.

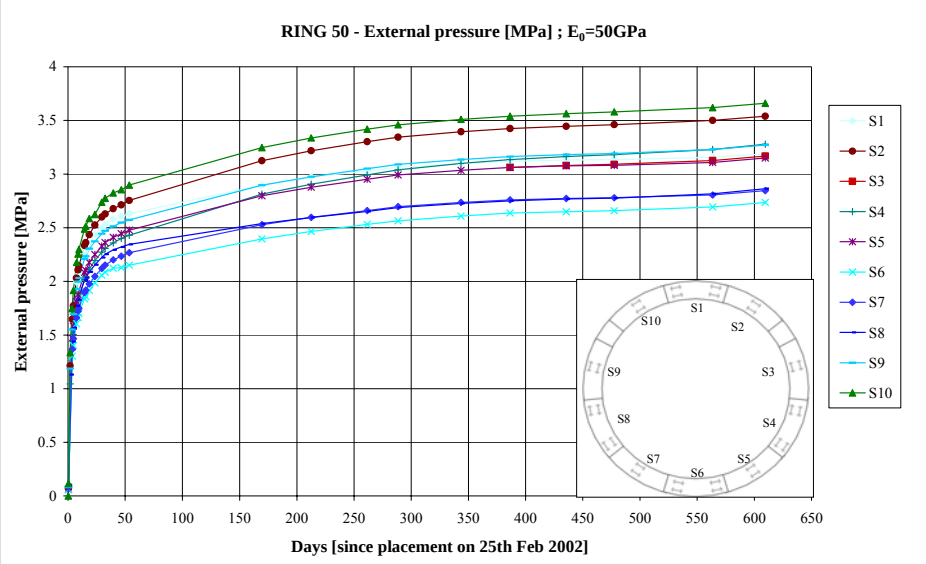


Figure 55: Time-dependent behaviour. Calculated external pressures on the segments of ring 50, located about in the middle of the connecting gallery, since their installation on February 25, 2002 (in days). The in-depth analysis of the data up to 06.05.2003 (day 436 since installation) led to concluding that the ground pressure on the lining is between 2.1 and 3.1 MPa.

Very extended disturbed zone

The zone disturbed by excavation extended much further in the Boom Clay than expected: both piezometer sensors and displacement sensors reacted almost instantaneously to the excavation in spite of their distance from the front, with variations of the pore water pressure and displacements being recorded at, respectively, more than 60 metres (Figure 56a) and more than 75 metres (Figure 56b) from the front.

This far-field behaviour remains difficult to explain. It was probably partly associated with the following two factors.

Characteristic time
Duration after which a given phenomenon starts to play a noticeable role. The characteristic time gives an idea of the order of magnitude of the speed of that phenomenon.

- *The apparent increase of the excavation radius* The fractured zone around the connecting gallery can be seen as an apparent increase of the excavated radius, thereby extending the zone influenced by excavation. The associated increase of the permeability in the disturbed zone certainly reinforced the phenomenon.
- *The viscosity of the skeleton* Viscous effects may appear very soon after the Boom Clay has been subjected to a hydromechanical solicitation, such as excavation, and contribute thus to the far-field response. Recent research has indeed shown that the characteristic time for the viscosity of the Boom Clay (about 10 days) is much shorter than the hydraulic characteristic time of the intact Boom Clay (about 4.2 years).

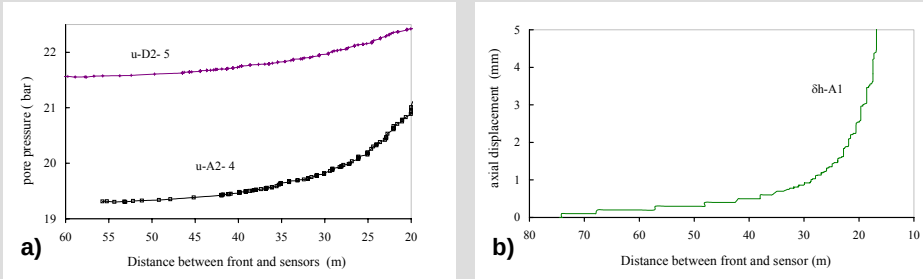


Figure 56: Evidences of the extent of the disturbed zone. a) Very early reaction of the piezometers; b) Very early reaction of the extensometer.

Blind predictions

Generally speaking, the tendency of the pore water pressure evolution during excavation can be predicted, but the calculated values diverge largely from the measurements: all decreases and increases were predicted, but were more important in reality than according to the modelling. The pore water pressure variation due to excavation depends essentially on the non-elastic behaviour of the massif, since there is no pore water pressure variation in the elastic zone, due to the absence of volumetric deformation. Consequently, modelling correctly the plastic behaviour is very important.

The tendency of the predictions of the displacements was in good agreement with reality for extensometer A1 and inclinometer E1. However, the extent of the excavation-disturbed zone predicted by the models was much smaller than in reality. Comparing modelling results with the other displacement data (B1, C1, and D1) was meaningless, because of the difficulty to interpret these data.

One of the conclusions of the comparison of the blind prediction results with the in situ data is that the hydromechanical coupling behaviour of the Boom Clay is very complex. A classical elastic perfectly plastic model is certainly not sufficient. More sophisticated elastoplastic models are therefore being investigated.

6.2. Fracture characterisation

The fracture observation programme carried out during the excavation of the mounting chamber and of the connecting gallery aimed to put to the test the conceptual insights recently acquired during the auscultation programme (Section 4.1) and to get more quantitative data on the fractures. The front of the mounting chamber and the front and excavated profile of the gallery were thus observed, mapped, and photographed as the excavation progressed. The fractures were characterised by measuring their dip and dip direction whenever it was possible to do so safely.

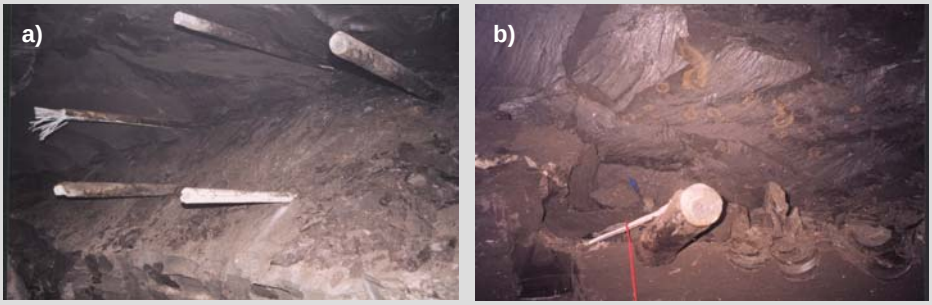
Fissure A fracture with a component of displacement normal to the fracture plane.

6.2.1. Mounting chamber

The characterisation of the fracturation in the volume of clay that was going to be excavated and around the future mounting chamber confirmed the results of the auscultation programme. Two main types of discontinuities were observed: large shear planes and tension fissures, the combination of which led to several clay blocks coming off during excavation. Small shear planes and tension fissures, associated with movement or detachment of clay blocks, desiccation, and convergence were also observed.

- *Large slickensided parallel shear planes* of more than 3 m^2 were the main discontinuities observed in the front. They were dipping towards the shaft axis (Figure 57 a and c). Their spherical geometry confirmed that they were related to the excavation of the shaft. They were separated by an average distance of 70 cm along the axis of the mounting chamber and their dip increased on average from 30° up to 70° with increasing distance from the shaft. The fractures were open up to a depth of 2 metres behind the initial front of the starting chamber, as demonstrated by the oxidation of pyrite on the shear planes and by the seismic measurements (Section 4.1.1).
- *Two large, irregular planes, consisting apparently of a junction of different tension fissures* that had all grown towards each other, were also observed in the upper third of the excavation front (Figure 57 b and c). They also seemed to be separated by less than 1 metre and their dip decreased towards the roof, on average from 30° to 20° north. Their average strike was east–west. The fissures were up to 3 cm wide, and oxidation had occurred up to 2 metres behind the initial front. These discontinuities resulted clearly from the decompression and the gravitational forces due to the convergence caused by the excavation of the second shaft and of the starting chamber.

Though they were not as efficient as expected in stabilising the clay massif, the anchors could nevertheless offer some interest for future fracture characterisation (Figure 58). The construction of the mounting chamber showed indeed that the resin that had been injected in the anchors and had filled the voids between the anchors and the massif had also penetrated into the fissures of the massif caused by the excavation of the second shaft and by the drillings themselves.



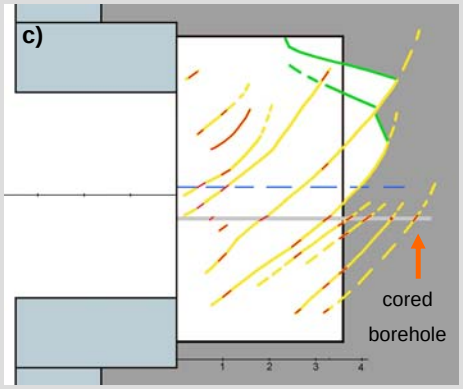


Figure 57: The two main types of discontinuities in and around the mounting chamber. a) A large shear plane covering the width of the excavation front, with the anchors sticking out of the clay (about 80 mm in diameter, including the resin); b) Oxidation spots on the surface of a tension fissure in the upper right corner of the excavation, at a depth of about 2 metres; c) Vertical cross section through the middle of the chamber. In red, all the measurements of shear planes, both before the excavation of the mounting chamber (cored borehole 2000-11, auscultation programme—Section 4.1.2) and during its excavation. In yellow, all the observations of shear planes. In green, tension fissures. Dotted lines are extrapolations.

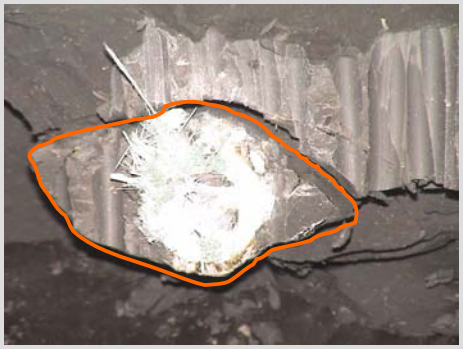


Figure 58: An anchor surrounded by resin that has partly penetrated into drilling-related fractures in the massif.

6.2.2. Connecting gallery

The front and excavated profile of the connecting gallery were observed as systematically as possible during excavation, taking into account the safety aspects linked to the presence of unstable clay blocks. The fractures were photographed (Figure 59 a and b), characterised, and mapped. The main focus was on shear planes, recognisable by their shiny, slickensided surface. (The other features were not studied systematically: these were for instance the large vertical tension fissures seen on the front, as a consequence of the detachment of large clay blocks, and the horizontal decompression fissures observed in its lower part.) The obtained fracture map,

which covers the entire gallery, reveals a fracturation pattern that is constant along the gallery, except in its first and last metres, because of the influence of, respectively, the second shaft and the existing facility (Test Drift). The average dip direction is parallel to the gallery axis.

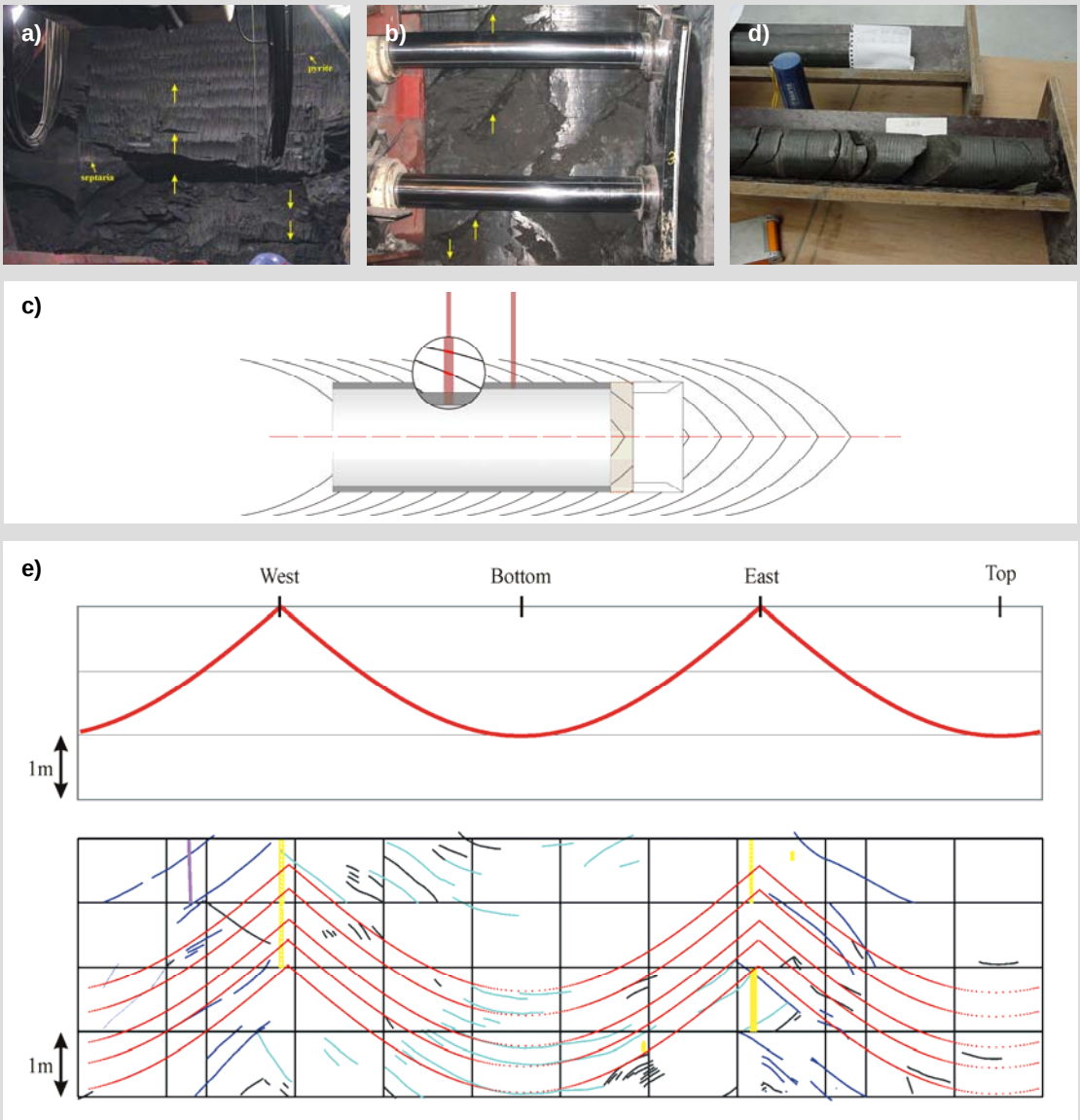


Figure 59: Fractures observed during the construction of the connecting gallery. a) Excavation front; b) Eastern sidewall, unsupported zone; c) Vertical cross section through the gallery showing the fracturation pattern around it, as deduced from the observations; d) Detail of a clay core obtained from a borehole drilled from the gallery (as shown on c); e) Theoretical traces on the excavated profile (represented as an unfolded cylinder) of a (slightly simplified) fracturation pattern and part of the fracture map (one band corresponds to 1 metre). The theoretical and observed traces are similar. Pyrite (purple) and septaria (yellow) layers are shown, too.

Pre-existing fractures were not observed at the Mol site, though it is impossible to prove their absence. Such natural fractures do occur in outcrops, where the clay burial history is completely different from that at Mol.

The fracturation pattern consists of two conjugated fracture planes: one in the upper part, dipping towards the excavation direction (north), and the other in the lower part, dipping towards the opposite direction (south) (Figure 59c). They intersect at mid-height of the gallery, where their dips reach 60° to 70° (north or south); those dips decrease to values as low as 30° with increasing vertical distance from the axis. These fracture planes are curved: their intersection, both with the vertical plane passing through the gallery axis and with horizontal planes, is a curve, but this curve is much more pronounced vertically than horizontally. The dip direction is parallel to the gallery axis near the centre of the front; it changes towards the eastern and western sidewalls. The distance between successive fractures is a few decimetres, and they originate at about 6 metres ahead of the front (Figure 62). Coring performed shortly after the construction of the connecting gallery in order to assess the radial extent of the fractures revealed the presence of fractures up to approximately 1 metre into the clay (Figure 59 c and d).

If the fracturation pattern is simplified to two flat fracture planes (one dipping 50° north, the other 50° south) having their strike perpendicular to the gallery axis, the theoretical trace on the excavated profile in the unsupported zone can be calculated as the intersection of the planes with a cylinder. The observed fracture traces in the unsupported zone are quite similar (Figure 59e). All the information on the fracture map has also been digitised, to enable average trace orientations to be calculated in order to determine the shape of the fracturation pattern quantitatively.

A fracturation pattern similar to the one induced by the excavation of the connecting gallery had been observed during the construction of both the second shaft (Section 6.2.1) and the Test Drift (1987) (Figure 60). The fracture planes encountered near the connection to the Test Drift and due to its excavation were dipping towards the south in the upper part and towards the north in the lower part.

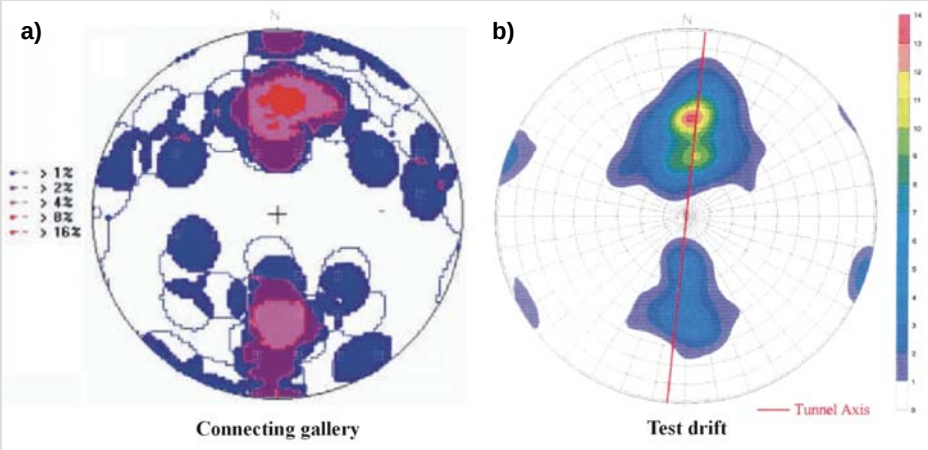


Figure 60: Stereographic plots of the fractures encountered during the construction of (a) the connecting gallery and of (b) the Test Drift, showing strong similarities in the patterns.

Fractures had also been observed in cores and around boreholes during the auscultation programme (Section 4.1.2). The pattern observed in the last core of borehole 2001-2 (Figure 61a), which is typical of the drilling process, is similar to that encountered around the connecting gallery (Figure 61b). Another fracture type observed around boreholes (Figure 18 b and c) has not been observed during the excavation of the connecting gallery. An important difference between an (uncased) borehole and the connecting gallery, though, is the fact that the gallery is lined almost immediately after excavation. The second type of fractures encountered around boreholes is thus possibly caused by the lack of support, and the presence of such fractures around the connecting gallery is uncertain. Fracture observation programmes during the excavation of a perpendicular gallery, such as the PRACLAY gallery, should provide more information in this respect.

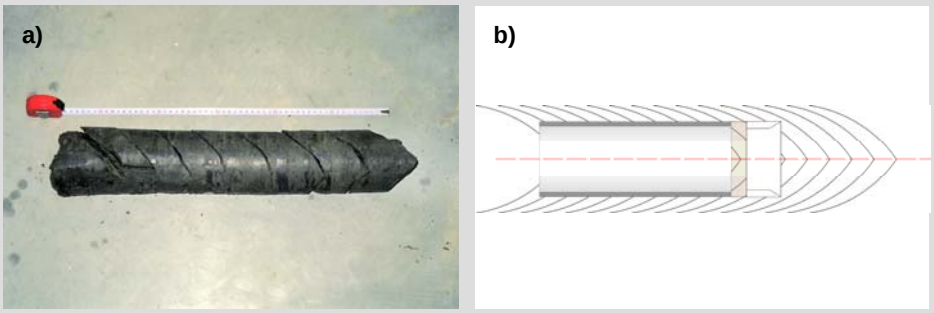


Figure 61: Similarities between the fracturation patterns observed (a) in the cores taken in the framework of the auscultation programme and (b) during excavation of the connecting gallery.

The shape and orientation of the observed fracture planes has been understood theoretically: it can be explained by the high level of the stresses ahead of the excavation front and by the differential stresses (Figure 62). The fractures planes originate at about 6 metres ahead of the front. Their curved shape corresponds with the modelled plastic zone, which is approximately spherical around the front. The horizontal intersection of conjugated fracture planes is explained by the in situ stress state: in the undisturbed clay, the largest principal stress is vertical.

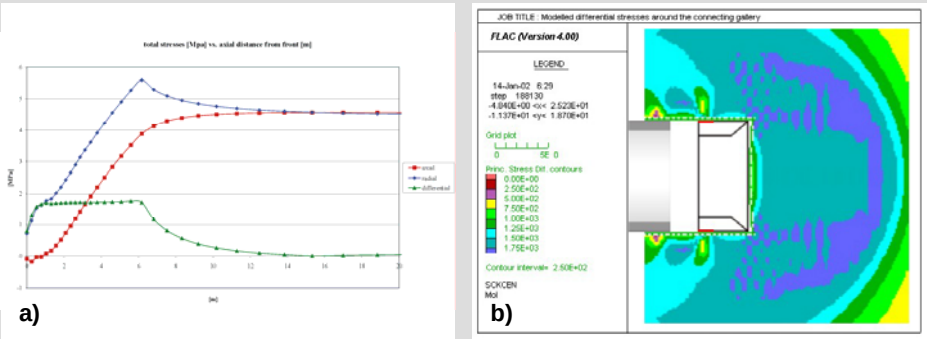


Figure 62: Modelled stresses along the gallery axis ahead of the excavation front. a) Axial, radial, and differential stresses, the plastic zone extending up to 6 metres ahead of the front; b) Differential stresses.

The shape of the fracturation pattern can be used to explain the anisotropic convergence of the clay around the excavation (Sections 6.4 and 6.5). Since the dip directions of the fractures are roughly parallel to the gallery axis, one can assume—if considering a vertical cross-section of the gallery—that vertical de-stressing is partly ensured by fracturation prior to excavation (which reduces the vertical convergence afterwards). On the contrary, horizontal de-stressing is not ensured, so the clay to the east and to the west of the gallery converges as if no fracturation had occurred, which results in a larger horizontal convergence.

An important remaining issue is the impact fractures can have on the long-term performance of geological repositories. This impact will probably be limited by the healing and sealing mechanisms that have already been identified qualitatively in various ways. It has been impossible, for instance, while excavating the first metres of the connecting gallery, to locate visually borehole 2000-11, which had been drilled from the northern starting chamber, parallel to the gallery axis (Section 4.1.2). Similarly, although fractures induced by the Test Drift 15 years ago were encountered at about 6 metres before the Test Drift front, signs of oxidation were only seen in the last metre of clay (Figure 63). This suggests that the fractures between 6 metres and 1 metre before the front had been sealed and were reactivated by the excavation of the connecting gallery. Finally, sealing effects have also been observed around instrumentation tubes in the massif.

Figure 63: Oxidation traces were only visible within one metre of the Test Drift front.

6.3. Petrographic and geologic studies

Some general petrographic and geologic studies were decided during excavation.

6.3.1. Mounting chamber

The petrographic study has shown that the Boom Clay only oxidises on the fracture walls (Figure 57b). On the one hand, there was macroscopic and microscopic evidence of pyrite oxidation, in the form of iron oxide or hydroxide precipitate, of gypsum, and possibly also of jarosite, all these newly formed minerals being always present on the fracture planes, microfractures, and discontinuities (worm tubes, etc.). On the other hand, there were no visible effects of oxidation within the clay matrix: instead, much framboidal pyrite, a very reactive species of pyrite, was observed. The high concentrations of SO_4^{2-} revealed by the geochemical analyses even at a distance of 25 mm from the main fractures might have been caused by microfractures.

6.3.2. Connecting gallery

The geological layers at Mol are nearly horizontal: they dip slightly towards the north. Since the Boom Clay itself has a layered structure, it has been possible to follow up its dip along the connecting gallery. The vertical position of septaria layer mS 140 was measured at different locations, leading to an average (apparent) dip along the gallery axis of 0.4° (Figure 64). These measurements are compatible with earlier observations in the second shaft and in cored borehole 2001-4, an ancient borehole from the Test Drift.

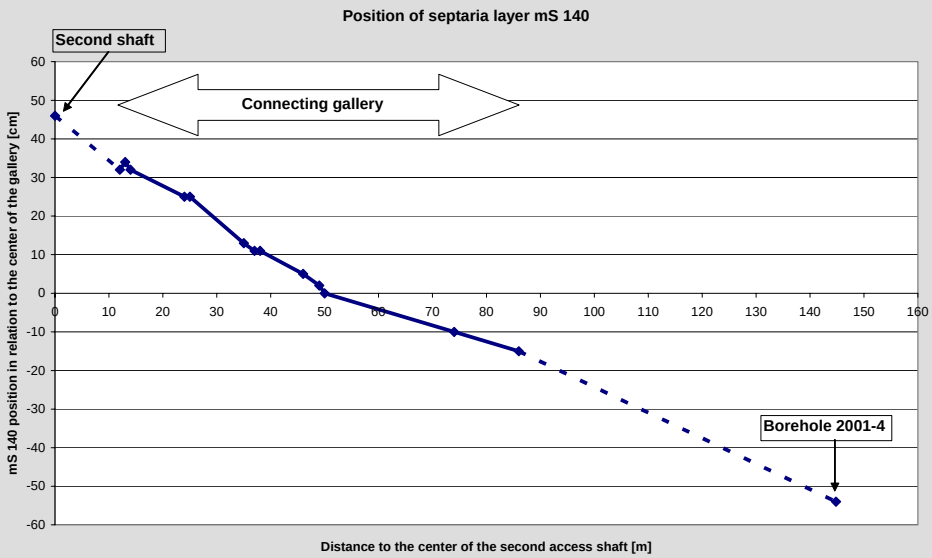


Figure 64: Vertical position of septaria layer mS 140 as a function of the distance from the shaft axis.

Thin pyrite layers (about 1 cm thick) and big pyrite concretions (lumps of up to 30 cm in diameter) were found (see also Section 5.2.4). Several types of fossils, such as crustaceans, shellfish, fish, and foraminifera, were also identified.

6.4. Convergence measurements

The instantaneous radial convergence of the clay was to be measured both in the mounting chamber and in the connecting gallery.

6.4.1. Mounting chamber

The convergence measurements foreseen in the mounting chamber were measurements both of the sliding of the ribs and of the convergence of the shotcrete layer. These measurements, carried out by SCM Tunnel, did not lead to any conclusive results.

- *The measurements of the sliding of the ribs* caused by the increasing pressure had to be abandoned quite rapidly after the start of the excavation works, since the ribs were welded together as soon as possible in order to secure the excavated zones and to stop or, at least, to minimise the detachment of clay blocks.
- *The measurements of the convergence of the shotcrete layer* were listed in tables and visualised in graphical format, but were not really suitable to determine the instantaneous convergence of the clay, since the presence of the shotcrete layer was interfering with the clay behaviour. They were, however, useful for the construction works. Five sections of the mounting chamber had been equipped with 7 measurement points consisting of light reflectors (20 × 20 cm), glued on metal plates and anchored in the shotcrete. The coordinates (x,y,z) of each measurement point were calculated on the basis of their angles and distances measured using a theodolite with respect to three fixed calibrated reference points with identified coordinates chosen in the shaft area.

6.4.2. Connecting gallery

The convergence measurements foreseen in the connecting gallery were straight measurements performed manually, just before and just after the last forward movement of the shield before placement of a new ring, through nine small holes in the upper 120° of the shield (Figure 65). These measurements indicated that the shield was in contact with the clay at its rear, over a distance of about one third to one half of its total length. They agreed very well with the modelled convergence: both the measured and the modelled convergence were about 35 mm on the radius at the level of the rear end of the shield (Figure 66).

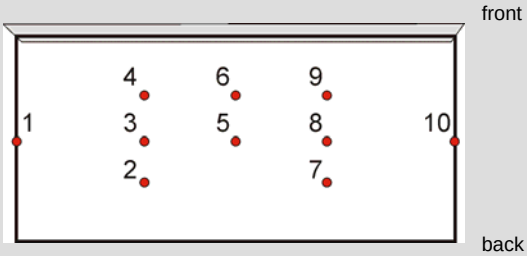


Figure 65: Location of the openings in the top half of the shield. The distances between the holes and the front of the shield were approximately 0.65, 1.30, and 1.70 metre.

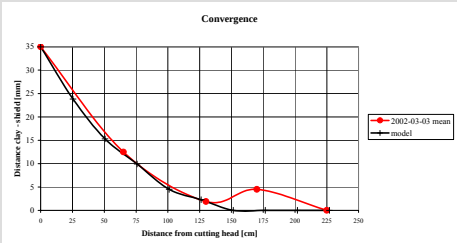


Figure 66: Comparison of a modelling result with a measurement result of the instantaneous convergence of the massif.

The radial convergence in the unsupported zone between the rear end of the shield and the lining has also been modelled with the FLAC software, using the exact dimensions of the shield and taking into account the alternation of the excavation and lining phases (Figure 67). This modelled convergence was of about 20 mm on the radius, though the maximum convergence allowed by the design was only 15 mm on the radius (difference between the shield diameter at its rear and the minimal lining diameter using normal keys: 4 820 mm − 4 791 mm = 29 mm—Section 4.4). In reality, the average measured convergence has been of only about 10 mm on the radius though: the average internal diameter of the rings was 4 000 mm (Figure 68). The relatively high value of the modelled convergence with respect to the measured convergence can be explained by the following three aspects.

- *Fracturation of the massif* The modelling assumed a continuous medium, whereas the massif is not continuous. It has been fractured by excavation, these fractures influencing, among others, the convergence behaviour. The presence of fractures is indeed thought to lead to vertical de-stressing ahead of the shield and, hence, to less convergence than expected after excavation.
- *Presence of cavities in the sidewalls* The fractures also caused clay blocks to come off from the front, thereby creating cavities in the sidewalls that were sometimes beyond the tolerance of the wedge-block technique. Higher diameters were thus observed than what would have been the case without the cavities, which implied that the convergence was underestimated.
- *Necessity to "trim" the sidewalls* It was necessary to "trim" the sidewalls over a portion of the gallery. The horizontal diameter was indeed too small for normal keys to be used; the vertical diameter, however, was large enough. A clay layer of 10 to 20 mm thickness was therefore removed manually from the sidewalls (Figure 32b). This was a rather basic method, but a fast and effective one: it took 5 to 10 minutes per ring. Sawn keys were nevertheless used for rings 25 and 31, because their diameter was believed to be too small. (It appeared afterwards that normal keys could have been used, too.)

Modelling results furthermore indicated that the convergence ahead of the front was about 45 mm on the radius.

The total radial convergence of the clay thus amounts to about 90 mm, namely

- 35 mm on the radius at the level of the rear end of the shield;
- 10 mm on the radius in the unsupported zone;
- 45 mm on the radius ahead of the front.

This value seems plausible in view of the CLIPEX measurements of the total settlement, namely up to about 60 mm at 5 metres above the axis of the connecting gallery (Figure 53).

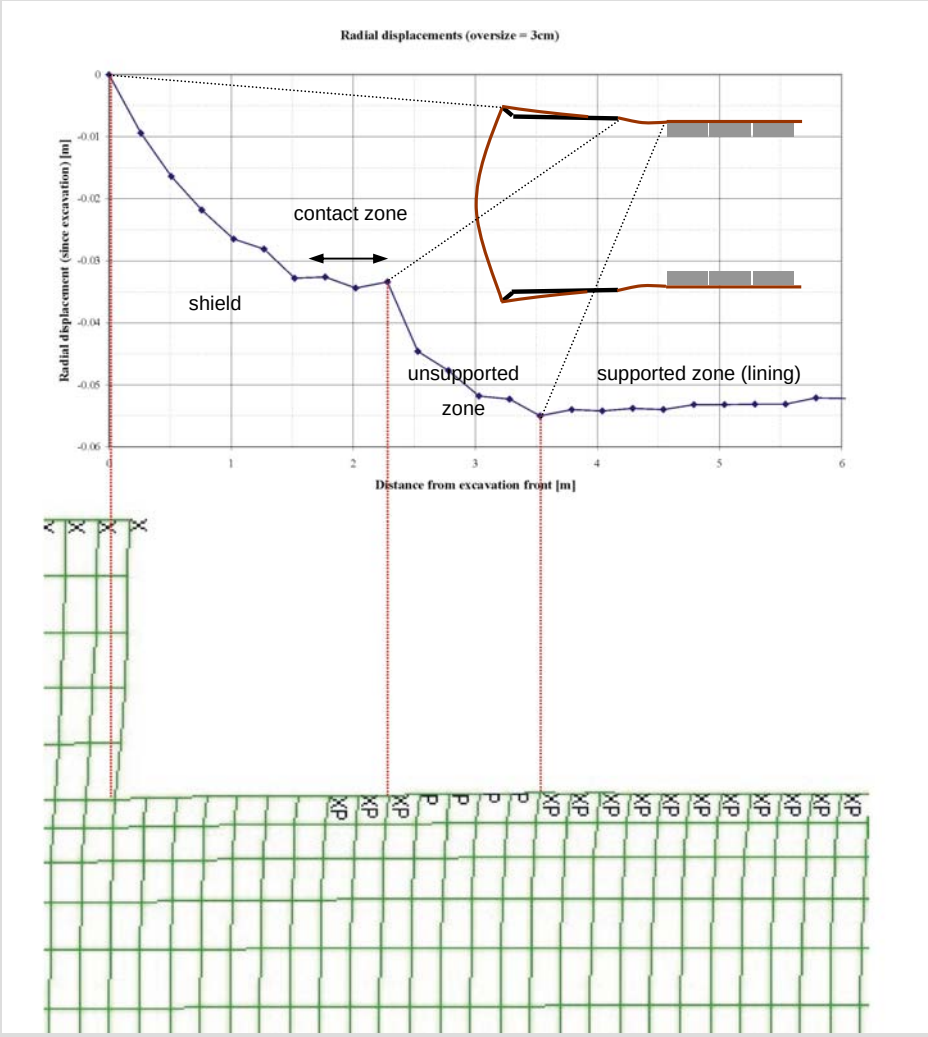


Figure 67: Modelling results showing the radial displacement of the grid points of the sidewall with, below, the corresponding part of the mesh. There is only shield-clay contact over a distance of about one third to one half of the total length of the shield.

6.5. Deformation of the lining

The global deformation of the lining of the connecting gallery was to be measured in two ways: manual measurements of ring diameters immediately after placement of each ring, and measurements of distances with Invar wires using reference points on the intrados of the segments. These two types of measurements provided complementary information.

Invar Nickel iron alloy having a very low coefficient of thermal expansion.

- The manual measurements of the internal diameters (for 1 to 4 orientations) of most rings, immediately after placement of each ring, gave the following results (Figure 68):
  - the diameters in the horizontal plane were smaller than those in the vertical plane;
  - the average diameter was equal to the nominal diameter (4000 mm);

- the diameters measured in the first and second quadrants (at 45° with the horizontal plane) were closest to the average.

- Afterwards, the regular measurements of distances with Invar wires in order to follow up the evolution of the diameter and of the shape of the lining did not show any movements at all. These measurements were carried out at a frequency that was adapted according to the observations. The initial Invar measurements (16 distances) of a selected ring were performed as soon as possible after the ring had been built, using markers fixed on all the segments. Each time a new ring was measured, two distances of all the previously selected rings were measured again. More distances were inspected when there was a significant difference between consecutive measurements. Practically, the diameters that were measured were those of rings 6, 10, 15, 20, 30, 40, and 50.

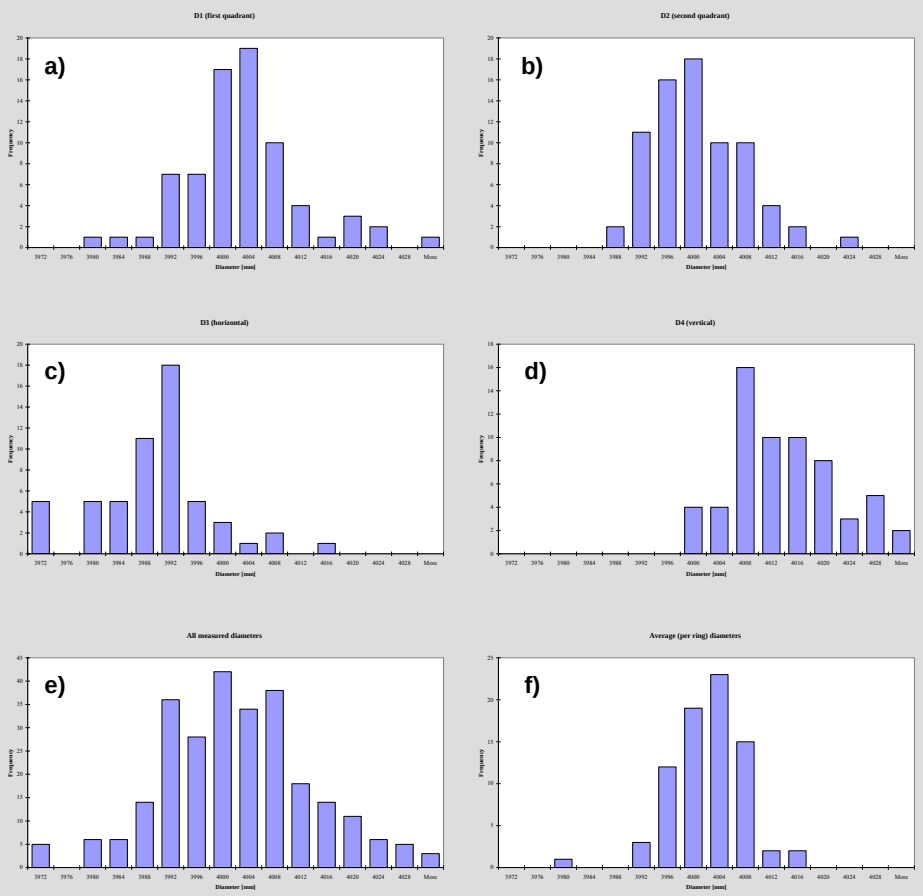


Figure 68: Histograms of the internal diameters of the lining rings. a) to d) Results for the four measurement orientations; e) A combined histogram for all measurements; f) A combined histogram of the average diameter per ring.

Towards the end of the construction of the connecting gallery, the pressure on the extremity of the Test Drift, which is supported by sliding ribs (ANDRA gallery), increased as a result of the

arching effect between the Test Drift lining and the gallery lining and the shield (Section 5.3). This increased pressure led to a faster decrease of the diameter of the ANDRA gallery than previously (Figure 69). This evolution normalised itself after the end of the works. Also as a result of the construction works, the length of the section of the Test Drift supported by sliding ribs increased by about 12 mm during the works, because of the decompression of the massif behind the front of the Test Drift. The lining rings remained vertical during the process, though, which is important as far as stability is concerned.

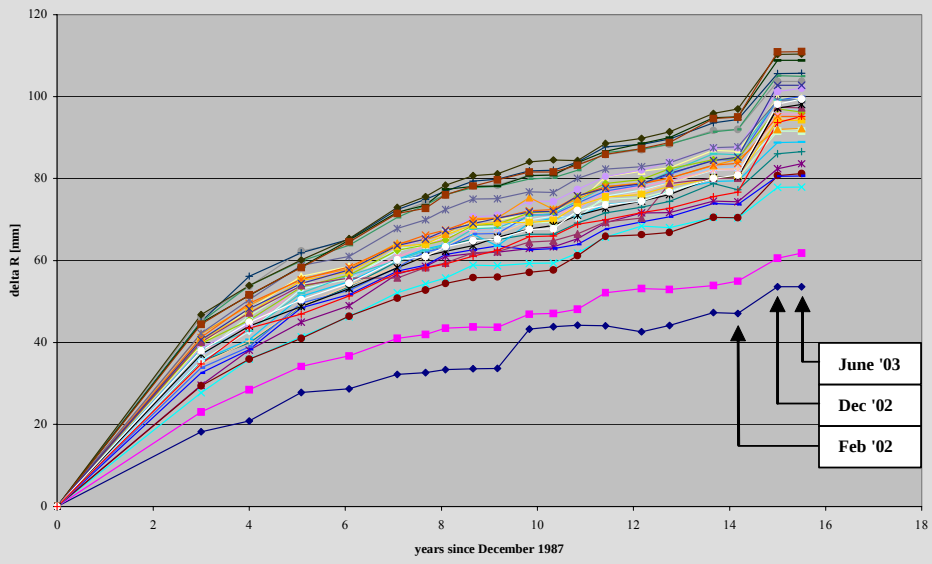


Figure 69: Decrease of the radius of the sliding ribs of the Test Drift since December 1987. The construction of the connecting gallery led to a faster decrease than previously.

6.6. Measurements related to the shield

The shield was equipped with a real-time data-acquisition system aimed to

- the continuous control and registration of the forces in all jack pairs, except the key jacks;
- the continuous control and registration of the stroke of 4 of the 10 jack pairs;
- the control and registration of the axial and tangential strains and of the deformations of the shield structure (at least once an hour for each strain gauge and once at each expansion phase of the lining).

The various types of measurements related to the shield led to the following results.

- The measurements of the forces exerted by the hydraulic jacks were useful during the excavation works themselves. Most of the time, the total force required to move the shield forward was comprised between 2000 and 4000 kN (200 to 400 tons) (Figure 70), namely more than ten times below the nominal total force of 40 000 kN, distributed over 20 hydraulic jacks. (The substantial overdimensioning of the thrust force of the tunnelling machine resulted from the impossibility to predict accurately the behaviour of the massif at

the time.) The maximum force ever used was about 7 500 kN. The construction of the connecting gallery proceeded without any long delays, though, so the shield did not get trapped at any moment. Should the works have been interrupted for a few days for whatever reason, a higher force would have been needed to free the shield afterwards.

- The continuous control and registration of the stroke of the jacks has been useful to follow up the length of the unsupported zone (Figure 31c).
- The measurements of the shield deformation via strain gauges were hard to interpret because of the complex structure of the shield.
- The position of the shield, which was compared with the theoretical axis of the gallery several times each day, to enable the necessary steering corrections to be made, indicated deviations that were consistently below 35 mm, except in the first few days (Figure 71).

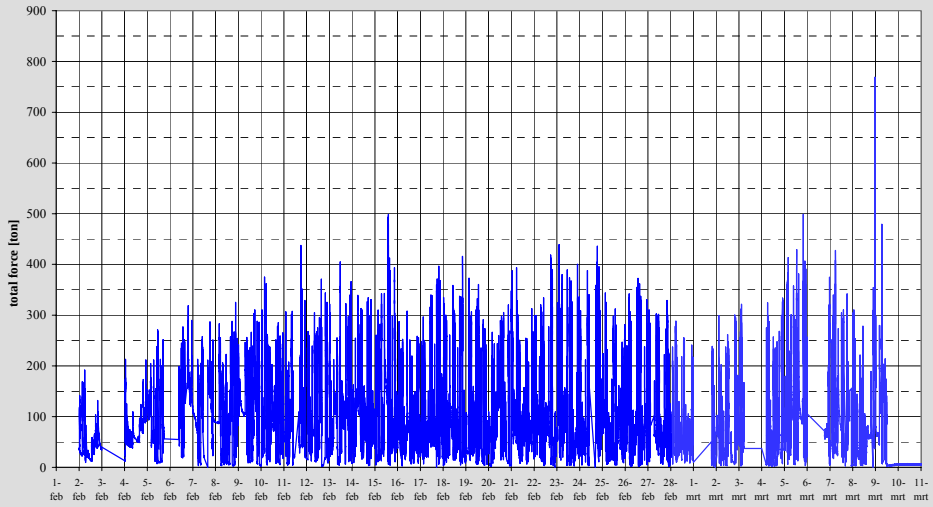


Figure 70: Logging of the total force exerted by the 20 hydraulic jacks, each peak corresponding to a forward movement of the shield. This total force was well below the nominal total force of 40000 kN.

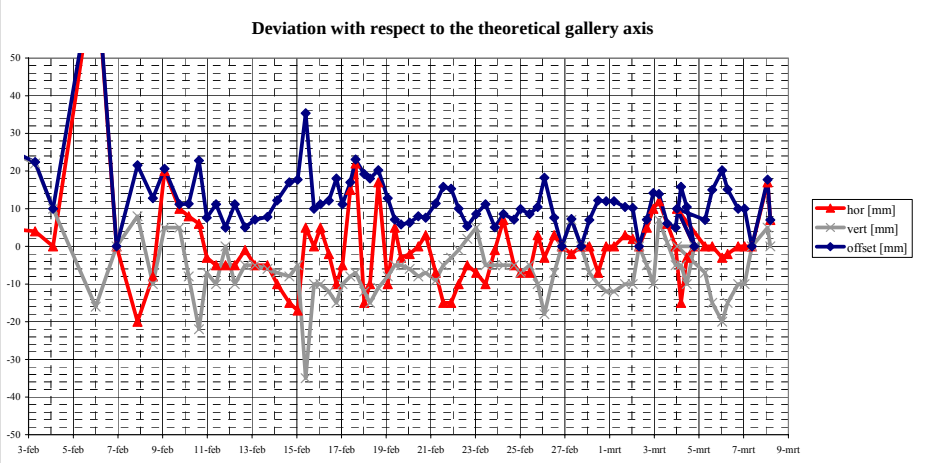


Figure 71: Distance between the actual and theoretical positions of the shield over time. This distance has been consistently very low.

7. Safety aspects

Generally speaking, underground activities entail a higher level of risk than similar activities carried out above ground. To minimise and/or to control those risks, special requirements and instructions are imposed by the authorities in charge of the supervision of underground activities, namely by the Department of Quality and Safety of the Ministry of Economic Affairs up until 01.01.2003, and by the Ministry of Work and Employment since then. In practice, it is the responsibility of the safety officer to take all the necessary measures to ensure safety, some of these measures being explicitly required by law, as for instance the ban on the simultaneous transport of persons and materials in the shaft.

The main safety issues related to the excavation of the mounting chamber and the connecting gallery and the measures taken in this respect were the following:

- *risks associated with falling clay blocks, either from the front or from the unsupported zone*:
  - reinforcement of the front of the northern starting chamber with anchors (Figure 72a);
  - regular detailed inspections of the front, with removal of all unstable clay blocks, and permanent inspection during interventions close to the front in order to call the workers back on time in case of doubt about its stability. (Such interventions were for instance measurements of the exact positioning of the shield or the replacement of parts of the head of the road header.);
  - timbering of unsupported zones (Figure 72b);
  - revision of the excavation scheme of the mounting chamber (excavation in half cross sections instead of excavation over the full cross section—Section 5.1);
  - excavation of the upper part of the front of the connecting gallery before its lower part (Section 5.2.4).
- *risks associated with the assembly of the shield (10 parts of about 2 tons each) and the excavation equipment in the mounting chamber*, where all parts had to be assembled from the inside, whereas they could be assembled from the outside above ground, where it was also possible to use a forklift or a jenny:
  - development of an assembly scenario based on the experience gained during the above-ground assembly and illustration with a full storyboard meant for the underground workers (Figure 72c);
- *risks associated with the placement of the lining segments (1.2 ton each) with the purpose-built bird-wing erector, the main risk lying with the top segments*:
  - use of additional support whenever necessary;
  - verifications of the proper functioning of the erector and of all its parts by SCM Tunnel before the placement of each lining ring and detailed weekly inspections by AIB-Vinçotte, an authorised inspection organisation;
- *risks associated with the transports in the shaft*:
  - ban on the simultaneous transport of persons and materials in the shaft;
  - for the transport of personnel, compulsory use of trapdoors in the lift;

- for the transport of the wagons meant for carrying the clay and of the carts meant for carrying the lining segments, use of a special double-locking system in the lift (one in the floor and one at both exits);
- request not to fill the wagons above their edge, to avoid any risk of pieces of clay falling out during transport.

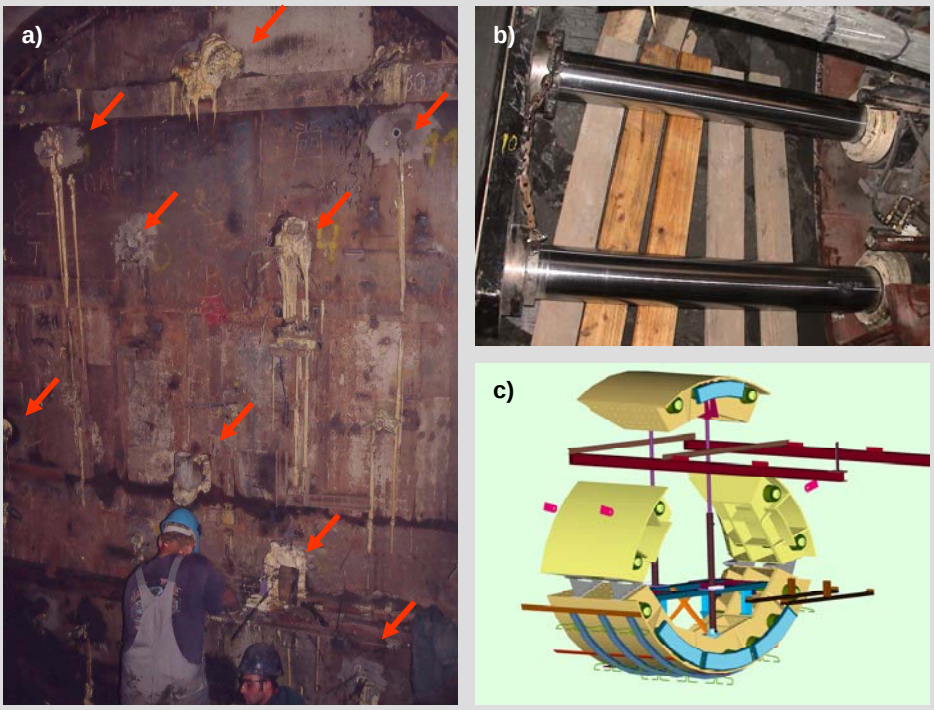


Figure 72: Some of the safety measures taken during excavation. a) Anchoring of the front of the northern starting chamber; b) Timbering placed between the jacks and the sidewall to prevent clay blocks from coming off; c) One of the many illustrations of the storyboard of the shield assembly (SCM Tunnel).

8. Communication aspects

The construction of the connecting gallery has been the object of a communication campaign, decided by EURIDICE for the following two reasons:

- according to its articles of association, EURIDICE must communicate about its own activities;
- the principle of public participation, which is highlighted in the first federal plan (period 2000–2004) on sustainable development[2], will be applied—according to the SAFIR 2 report—to the long-term management of intermediate and high-level waste. (It is already being applied to the ongoing site selection process for short-lived and low-level waste.)

EURIDICE and its Members—SCK•CEN and ONDRAF/NIRAS—therefore considered it very important to go public with the major underground excavation works that were to be conducted in the reference host formation and on the reference site considered by ONDRAF/NIRAS in its methodological research programme.

The information campaign stressed the following two aspects:

- the construction of the connecting gallery was an experiment in itself and a world-wide first as regards tunnelling in a poorly-indurated clay and at this depth;
- the tunnelling technique selected was the one that was best suited to limit the disturbances in the massif, but not the one that was the least liable to failure during construction.

The information campaign started in January 2002 with the diffusion by EURIDICE of a leaflet that was sent to local and national authorities, national industrial partners, and local and national media. Several local and national newspapers and TV stations reported on the subject. At completion of the works, the information campaign was closed by the diffusion of a second leaflet and by the inauguration of the gallery, a ceremony to which authorities, the industrial partners, and the press were invited.

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[2] Belgium has fully committed itself to the path of sustainable development by the law of 05.05.1997 coordinating the federal policy on sustainable development. The initial plan drawn up for the period 2000–2004 and published in the form of a royal decree highlights five sustainable development principles taken from the list of 27 principles agreed upon during the Rio Conference (1992). One of them is the principle of public participation.

9. Reinforcement for large openings

Since the primary (wedge-block) lining of the connecting gallery has been dimensioned to allow one opening of up to 100 mm per ring (Section 2.2.2), it will need to be reinforced by a secondary lining whenever openings larger than 200 mm are necessary. (Openings of up to 200 mm are indeed possible provided they are centred on the join between two adjacent rings and provided the step between those rings—if there is one—does not exceed 20 mm.)

A feasibility study performed by BELGATOM concludes that it is possible to construct steel reinforcement rings and proposes a preliminary design for three different configurations (Figure 73). These configurations are the following:

- a 500-mm opening (horizontal borehole at the axis level) surrounded by two 150-mm openings in the same vertical plane (The corresponding reinforcement ring allows this configuration on one or on both sides of the gallery.);
- a 800-mm opening (horizontal borehole at the axis level);
- a 2500-mm opening (horizontal gallery at the axis level). This configuration is the configuration that will probably be needed for the excavation of the future PRACLAY gallery, which will be constructed perpendicularly to the connecting gallery (Figure 1).

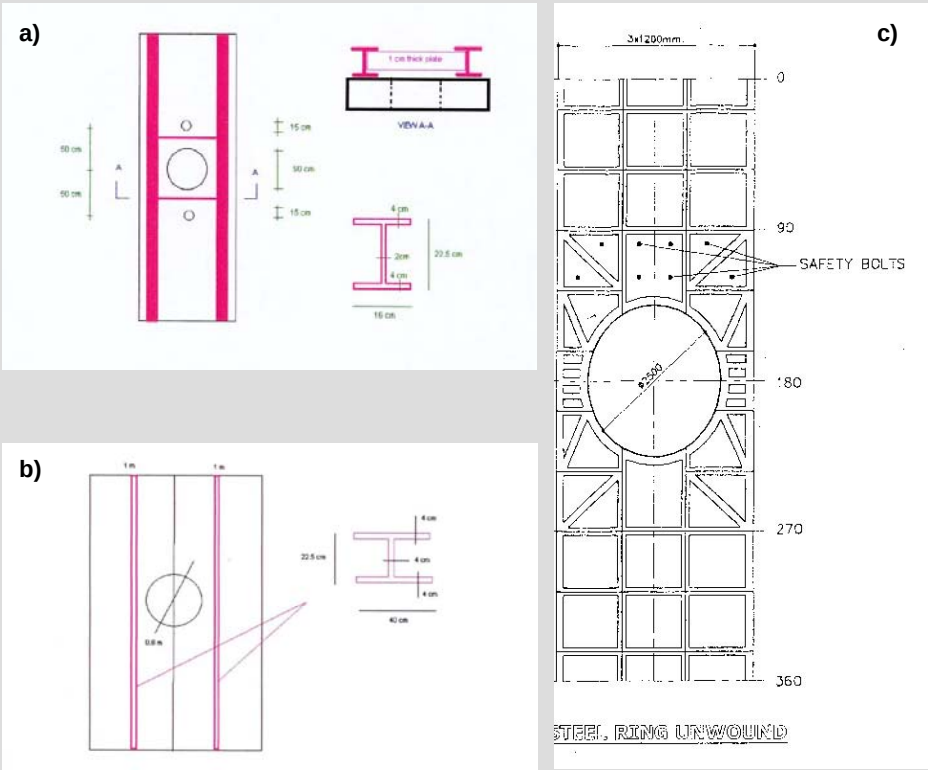

Figure 73: Reinforcement rings (BELGATOM). a) Reinforcement for a 500-mm opening surrounded by two small openings; b) Reinforcement for a 800-mm opening; c) Reinforcement for a 2500-mm opening.

10. Conclusions, evaluation, and recommendations

The provisional delivery of the connecting gallery on 07.06.2002 has been evidence of the success of the underground construction works. An industrial tunnelling technique has indeed been used, for the first time ever, to construct a gallery at 223 metres depth in a poorly-indurated clay. This technique has enabled the target excavation rate of 2 metres per 24 hours to be met, and the convergence to be limited. Moreover, the instantaneous convergence has been measured for the first time, and the CLIPEX instrumentation provided valuable hydromechanical information. Further characterisation and testing of the gallery lining and further characterisation of the excavation-disturbed zone, of the excavation-damaged zone, and of the healing and sealing phenomena are going on.

The total cost of the realisation of the connecting gallery, including the costs of the engineering office BELGATOM, the costs of the control office SECO, the costs of the main contractor SCM Tunnel, the costs of EURIDICE, and the various other costs, amounts to 5.7 millions EUR (economic conditions of August 2000).

The lessons of the construction of the connecting gallery will be very important for future excavation works, in particular for the construction of the future PRACLAY gallery and, possibly, for the construction of a repository for radioactive waste in the Boom Clay. Some of these lessons were collected during an official evaluation meeting held on 22.05.2002 between EURIDICE, SCM Tunnel, BELGATOM, and SECO. The most important ones are recorded below, in the chronological order of the project, including those regarding the measurement and research programmes, the infrastructure, and the safety aspects.

10.1. Adjudication procedure

The choice of the restricted call for tenders procedure confirmed itself as the most appropriate procedure, for the following three reasons.

- This procedure entailed a preselection of candidate contractors, which offered the advantage to rule out from the start contractors which were not qualified enough for this type of work.
- This procedure enabled EURIDICE to have a better grasp on the design of the construction works, which was advisable given their highly technological and innovative character. The design was taken care of by BELGATOM, in collaboration with EURIDICE. This collaboration thus associated experience in underground works with an in-depth knowledge of the Boom Clay.
- The analysis of the tenders showed clearly that an evaluation on the sole basis of the cost, as would have been the case with a standard adjudication procedure, would have been inadequate to retain the best tender: the contract has indeed not been adjudicated to the cheapest tenderer.

10.2. Technical specifications

Taking the time to write both the administrative and the technical aspects of the technical specifications with care has proved to be a valuable investment: there has hardly been any discussion afterwards regarding uncertainties, diverging interpretations, or gaps in the texts.

10.3. Preparatory works

The preparatory works carried out after the contract adjudication have all proved very useful: they have most likely saved EURIDICE major difficulties during the construction works. A similar, thoroughly thought out preparation remains advisable for future excavation works.

10.3.1. Auscultation programme

The auscultation programme was necessary because the clay massif had been heavily damaged by the excavation of the second shaft, as revealed by the difficulties encountered during the construction of the starting chambers. It has allowed these damages to be quantified. Such programme should be designed again in the future if the massif to be excavated has somehow been highly damaged. The results of the auscultation programme were confirmed later on by observations and measurements during the construction of the mounting chamber and of the connecting gallery.

10.3.2. Other preparatory works

After preparing the various aspects of the works in detail, SCM Tunnel turned them into plans and procedures that were then submitted for approval to EURIDICE. This detailed preparation has proved very useful during the works. It enabled not only some potential problems to be detected in advance, but it enabled also the problems detected during the works to be solved faster, thanks to the fact that the various parties involved were already very familiar with all aspects of the works.

Shield design The minor modifications brought to the initial shield design during the preparatory works, as a result of numerical simulations carried out by EURIDICE, proved to be appropriate: increasing the shield diameter by 20 mm facilitated the placement of the segments and the steering of the shield, and increasing the maximum oversize from 20 to 30 mm (on the radius) enabled an adequate contact length between shield and clay. Too short a contact length would indeed have entailed the risk that the shield would "float" in the gallery; too long a contact length, on the contrary, would have made steering of the shield more difficult and would have increased the risks that the shield would get trapped. The elasto-plastic models used in the simulations have thus been sufficient for determining the shield geometry.

Segment design Overall, the design of the segments was adequate. The dimensions of the lining were well chosen and the range of possible ring diameters was large enough. Despite the high tolerances imposed, spoiling occurred when concrete-concrete contact was allowed be-

tween the lining rings. High-density polyethylene plates were therefore inserted between the segments of adjacent rings. Furthermore, some of the segments did not meet the tolerances on their top side, namely on the side that was free during casting. Whenever this occurred, this side needed to be finished manually. Using metal moulds instead of concrete ones would probably enable higher tolerances to be reached.

Decision table Though the decision table has hardly been used, a similar analysis, namely an inventory of the possible difficulties in case of excessive or insufficient convergence of the clay, with the corresponding corrective actions, remains advisable for future excavation works.

Test assembly The test assembly and the trial carried out on surface prior to assembling the tunnelling machine in the mounting chamber have enabled a few shortcomings that would have been very difficult to correct in the underground to be detected and corrected on surface. Together with the detailed procedure prepared by SCM Tunnel for the underground assembly, it enabled the tunnelling machine to be assembled fast, correctly, and safely. This approach remains advisable in the future.

10.4. Construction works

The construction works have overall been successful. The accuracy of the numerical predictions of the displacements of the massif and of the pressures on the lining has allowed the design and the dimensions of the tunnelling machine and of the lining segments to be appropriately defined. The two major challenges have thus been met:

- optimising the overexcavation and the shield diameter with respect to the dimensions of the lining, so as to minimise the convergence while avoiding the shield getting trapped in the massif;
- reaching the target construction rate of 2 metres per 24 hours. (The excavation rate could be increased up to 10 metres a day with minor adaptations to the excavation technique and provided the access shaft was larger.)

Though the technical problems encountered during the whole construction process and related to the design aspects were only minor, there has been one major, unexpected problem: the extent of the detachment of clay blocks from the front and from the unsupported sidewalls. This has been both a safety issue and a construction issue.

10.4.1. Construction of the mounting chamber

Though it has followed the initial planning quite well, the construction of the mounting chamber has been hampered by the fracturation of the massif and the detachment of clay blocks. Besides the fractures caused by the excavation of the second shaft, extra damages resulted from suboptimal choices regarding the excavation sequence, the excavation tool, and the excavation regime. Those three aspects are easy to modify. The anchors used to reinforce the clay front did not perform as expected though, but there is not yet a consensus within EURIDICE as to whether they should still be used in the future.

Excavation sequence The best excavation sequence for the mounting chamber has been confirmed by experience to be the one described in the technical specifications, namely excavating the top half first, lining it with steel ribs and shotcrete, and then closing the ribs during the excavation of the bottom half of the chamber and shotcreting them.

Excavation tool Instead of excavating the front of the mounting chamber with a pick hammer, which created heavy vibrations in the front and favoured the detachment of clay blocks, it would be preferable to use a road header.

Excavation regime Instead of excavating the mounting chamber according to a discontinuous regime (interruption of the works during weekends), it would be preferable to work without interruptions (24 hours a day, 7 days a week). This would indeed reduce the tensions on the front and, hence, the convergence and the accompanying detachment of clay blocks.

Anchors The glass-fibre anchors that had been foreseen to reinforce the front and to retain falling clay blocks proved unable to retain the blocks sufficiently during the construction of the mounting chamber, because of the decompression of the massif and, hence, because of the loss of contact between the anchors and the clay. The secondary fracturation that appeared around the anchors reduced the size of the falling blocks, though. However, these anchors could present an interest in fracture characterisation programmes, such as auscultation programmes, to visualise the fracturation of the clay massif, as a result of the fractures being filled by the resin injected in the anchors.

10.4.2. Tunnelling machine

The design of the tunnelling machine and, in particular, the design and dimensioning of the shield, was overall adequate. This dimensioning proved to be very compatible with the nominal dimensions of the lining. And as regards the design, most of the improvements suggested hereafter are relatively easy to implement. They relate, however, to aspects that would substantially increase the cost of the machine and that can therefore only be envisaged for constructing galleries that are much longer than the connecting gallery. A major change, though, could be to abandon the road header to the benefit of a full-face tunnelling machine, in order to reduce the fall of clay blocks from the front and from the unsupported zone.

Using a *full-face* tunnelling machine instead of a road header would enable a uniform and simultaneous excavation of the front and could drastically reduce the detachment of clay blocks from it. If used in combination with "*fingers*" (steel laths or rods) at the upper rear of the shield (Figure 74), such machine could also largely prevent the fall of clay blocks at the back of the shield. The fingers would be parallel to the gallery axis and in contact with the roof of the gallery. Their length should be at least that of the unsupported zone. To prevent the fingers from being in the way during ring construction, the segments would have to be provided on their outer side with longitudinal grooves, deep and wide enough to allow for steering and progression of the shield. The presence of the fingers would thus not prevent the rings from being expanded against the clay massif. Should such a technique be used, then it would be necessary to investigate whether to inject the gaps between the segments and the clay afterwards.

Figure 74: A full-face tunnelling shield equipped with fingers (source unknown).

Integrating a full-face tunnelling machine in the design would imply to take the following aspects into consideration, though:

- a cutting head would still be necessary to obtain a smooth excavation profile;
- the shield would need to remain short;
- the construction in the mounting chamber would possibly be more difficult;
- there could be more convergence around the shield, as a result of the absence of decompression at the front.

Integrating a full-face tunnelling machine in the design would also present some disadvantages:

- impossibility to inspect the front, and in particular to observe the fractures and the status of the clay;
- impossibility to avoid pyrite and septaria concretions;
- limited access to the drilling tools;
- potential difficulties to evacuate the excavated clay without using water, which is the usual manner.

The main design improvements recommended as regards the tunnelling machine are the following:

- *Automating the setting of the oversize*, for instance through little jacks, to allow easier, faster, and safer modifications, without the need for workers to be near the front.
- *Automating the measurements of the position and orientation of the shield*, to save time and to be able to perform them more frequently and more accurately. The position of the shield could be determined by (semi-)automated topographic measurements of the position of two reference points fixed to it. *An automated level could* be used to determine the inclination of the shield. The proposed system allows a better, real-time control of the steering corrections, and avoids the presence of workers near the front.
- *Integrating an anti-roll system in the shield design*. The use of segments as counterweights has largely compensated for the roll of the shield, but is not an optimal solution. (The solution foreseen in the technical specifications, namely repositioning the jacks, has not been used, because it is a difficult and lengthy process.)
- *Reducing the overdimensioning of the thrust force of the jacks*. In practice, the total nominal force of the jacks (40 000 kN) has never been used. Limiting the total force pro-

vided by the jacks (for an otherwise comparable tunnelling machine) to 20 000 to 30000 kN should be sufficient.

- *Using an erector with more degrees of freedom* for the placement of the lining segments, among others to deal better with the large cavities in the roof and sidewalls.
- *Adapting the design of the shield to facilitate the handling of the keys and their insertion in the rings*.
- *Designing the tunnelling machine in such a way that at least the road header and the transport conveyor could be driven backwards far enough from the front* to enable repairs and maintenance works to be carried out in safe conditions.
- *If possible, designing the head of the road header to take into account the presence of large lumps of pyrite* at the considered depth in the massif. In addition, *foreseeing a spare head and designing the machine in such a way that the head could be easily replaced* in case it got damaged, for instance because of the presence of pyrite lumps. This would enable broken tooth-holders and broken teeth to be replaced on the surface and not close to the front.

10.4.3. Construction of the connecting gallery

The wedge-block technique has been preferred to a technique using bolted segments for the following two reasons: first, because it is fast to install and expanded against the clay massif, hence disturbing it less; second, because of economical reasons. It could be improved through a few simple modifications and remains a privileged option for future underground construction works in the Boom Clay, even though misalignments between the segments of up to 5 cm were reported, as a result of the presence of cavities in the excavated profile. (The bolted-segments technique would suppress the problem of the falling clay blocks in the unsupported zone, since the bolted segments would be assembled in the shield. This technique however enables the massif to converge more than the wedge-block technique does, since the lining is not expanded against the massif. The resulting disadvantage of having to inject the annular space between the lining and the massif should be placed in perspective, though: some sections of the connecting gallery had to be injected as well in order to fill the cavities induced by the fractures.)

The excavation parameters were very well under control during the construction of the connecting gallery, which limited the uncertainty on the interpretation of the measurements.

Steering of the shield The steering of the shield did not present any particular difficulties. The vertical corrections were much easier than the horizontal ones, though, since the radial convergence observed in the unsupported zone in the vertical plane was lower than in the horizontal plane. This observation, which is in contradiction with the fact that the largest undisturbed in situ stresses are vertical, can be explained by the presence of fracture planes, which enable movements of the clay blocks and redistribute the stresses. This phenomenon would probably be less present with a full-face tunnelling machine, since there would not be as many clay blocks detaching themselves from the front and unsupported zone. An additional option would

be to use an automated system enabling different oversizes, depending on the position on the cutting head.

Ring design The design of the rings could have been based on the use of a single key, which would have enabled the rings to be placed faster and more easily. Using two keys, however, enables a larger range of ring diameters, and should therefore remain the preferred design for future excavations.

Damage to segments The use of high-density polyethylene plates between adjacent rings is necessary in order not to damage the segments, certainly under high constraints.

Excavation sequence The excavation of the front with a road header should always start with the top half to limit the fracturation and the detachment of clay blocks. This has been done towards the end of the works and seemed to help.

10.5. Measurement and research programmes

The measurements and research programmes carried out before and during the construction of the mounting chamber and during and after the construction of the connecting gallery have led to a very comprehensive characterisation of the fracturation pattern and of the instantaneous hydromechanical response of the Boom Clay to an excavation using an industrial tunnelling technique.

The intensive characterisation of the *fracturation pattern* performed during the construction of the mounting chamber and of the connecting gallery was particularly important, since the fracturation has a direct impact on the hydromechanical response of the massif. It resulted in a description, in terms of orientation and shape, of the fractures around the gallery and in a better understanding of how these fractures are formed. All the fractures observed at the Mol site have been induced by excavation. They originated at some 6 metres ahead of the excavation front. Coring after completion of the gallery indicated that the fractures extend up to about 1 metre in the radial direction. Future core samplings and the construction of the PRACLAY gallery, perpendicularly to the connecting gallery, will provide more information. Natural, pre-existing fractures were not observed, though it is impossible to prove their absence. An important remaining issue is the impact fractures can have on the long-term performance of geological repositories. This impact will probably be limited by the healing and sealing mechanisms that have already been identified qualitatively in various ways. For instance, although fractures induced by the Test Drift 15 years ago were encountered at about 6 metres before the Test Drift front, signs of oxidation were only seen in the last metre of clay. This observation suggests that the fractures between 6 metres and 1 metre before the front had been sealed and were reactivated by the excavation of the connecting gallery.

Interestingly, the fracturation pattern observed in the cores taken during the auscultation programme was similar to the pattern observed during the excavation of the connecting gallery: the fractures at the front had a parabolic shape (Figure 75).

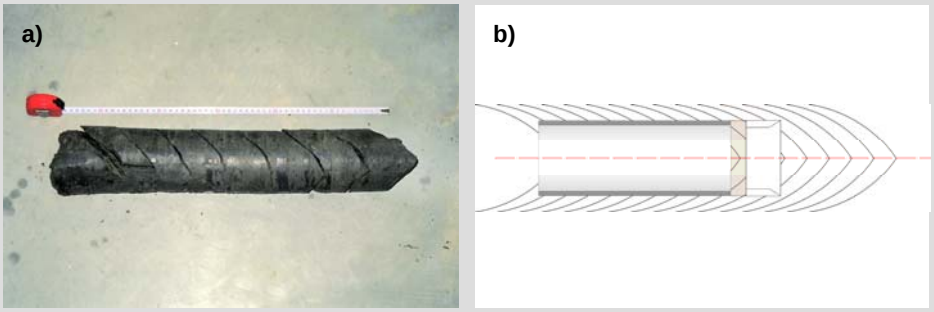

Figure 75: Similarities between the fracturation patterns observed (a) in the cores taken in the framework of the auscultation programme and (b) during excavation of the connecting gallery.

As regards the *CLIPEX programme*, the characterisation programme has allowed to choose the best parameter set and to establish hypotheses for the blind predictions of pore water pressures, displacements, and pressures on the lining. The monitoring of the hydromechanical behaviour of the clay has overall proved to be successful, too. It has evidenced that the behaviour of the Boom Clay massif is characterised by a strong hydromechanical coupling, already noticeable at an unexpectedly large distance from the excavation, and by a clear time dependency. It furthermore confirmed the blind predictions performed earlier in terms of both displacements and pressures on the lining, the modelling of the evolution of the pore water pressure deep inside the clay massif being still an unresolved issue.

- The measurement of the pore water pressure through piezometers has proved to be a very reliable, accurate, and mature technique. For the first time, the pore water pressure has been measured outside of the zone of influence of the HADES facility. The value measured at 223 metres depth was confirmed to be equivalent to the initial undisturbed pore water pressure, namely about 2.2 MPa. Furthermore, the measurements of the pore water pressure clearly reflected the influence of the fracturation, of the decompression of the massif, and even, in the vicinity of the excavation front, of the excavation phases (alternation excavation/lining). Finally, measurements have revealed that the hydraulically disturbed zone extends up to more than 60 metres from the excavation front, which is important for the proper understanding of the hydromechanical behaviour of the Boom Clay.
- Displacement measurements so far have led to a good knowledge of the axial displacements on the gallery axis and of the ground settlement at 5 metres above the gallery. Together with the measurements of the equilibrium pressures on the lining, they proved that the hydromechanical disturbances have been minimised and that the construction technique used allows a very good control of the excavation parameters.
- The pressure on the lining increased very rapidly in the first few days after construction, as a result of the reconsolidation of the Boom Clay caused by the drainage of the pore water and the creep of the massif. This too indicates that the hydromechanical disturbances have been minimised. The pressure has been stabilising progressively since.
- Total pressure measurements in small-diameter boreholes, however, are a pending issue. The total pressure sensors indicated indeed a total pressure that was almost identical to

the pore water pressure. This unexpected observation, due to artefacts associated with the installation procedure of the sensors, requires further investigation.

The total *radial convergence* of the Boom Clay was about 9 cm on the excavated radius (2.5 metres), which is considered acceptable in terms of hydromechanical disturbances. This total radial convergence is the sum of the measured instantaneous convergence of the Boom Clay, which was about 45 mm on the radius, and the radial convergence ahead of the excavation front, which was also about 45 mm on the radius according to the modelling results and as confirmed by the displacement sensors.

Finally, the *petrographic study* has shown that the Boom Clay only oxidises on the fracture walls: the only evidence of pyrite oxidation, under the form of newly formed minerals, is indeed on the fracture planes, microfractures, and discontinuities. Such oxidation effects are the visual print of the geomechanical disturbances induced by excavation. Assessing them is important in the framework of the migration studies and for performance assessments.

10.6. Infrastructure

Replacing the existing lift cage at the beginning of the works by one specially designed for the works has been very useful and should be considered for the construction of the PRACLAY gallery.

Clearly, the hoisting installation is the Achilles' heel in the whole construction process: a breakdown would entail the interruption of the excavation and lining activities, as a result of the impossibility both to evacuate the clay and to transport the lining segments. This could very well lead to the tunnelling machine getting trapped in the clay because of convergence. For a real repository, it would therefore be advisable to foresee a redundant hoisting installation, spare parts for the hoisting installations, and a power backup. Such backup should also be foreseen for the tunnelling machine.

Still in the context of the construction of a real repository, the construction rate would be expected to reach, and even exceed, 10 metres per 24 hours. The access shaft would then indeed have a much larger diameter and be equipped with a much more powerful hoisting installation, having a much higher transport capacity

10.7. Safety aspects

The safety measures taken during the works have overall proved to be satisfactory. Implementing the following recommendations would improve safety, though:

- taking measures to limit the detachment of clay blocks from the front and from the unsupported zone (Sections 10.4.1 and 10.4.2);
- improving the design of the equipment (erector, ratio of the diameter of the cable drums with respect to that of the cables, etc.);
- limiting the number of visitors during the works, since these people can create dangerous situations just by ignorance.

Annex 1 Further reading

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