

CLIPLEX
CLay Instrumentation Programme for the EXtension
of an underground research laboratory

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FINAL REPORT

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Foreword

As in all nuclear power generating countries, the management of high-level and long-lived radioactive wastes is an important environmental issue today. Deep geological disposal of these wastes in clay formation is found to be a suitable management option. An essential item for the long-term performance of these underground repositories is the proper evaluation of the Excavated Disturbed Zone (EDZ) around excavation.

The aim of the CLIPEX project was to assess and characterise the EDZ around a gallery similar in size to that of a disposal gallery for radioactive waste. The CLIPEX project was realised in the frame of the extension of the underground research facility (HADES) in the deep Boom Clay formation at Mol (Belgium). The extension consisted of a second shaft, and the excavation of a gallery connecting this shaft with the existing facility. The connecting gallery was realised with a tunnel machine allowing an excavation rate about three meters per day, and minimising perturbation of the host rock. A unique and original opportunity was then given, within the CLIPEX project, to monitor the hydro-mechanical parameters ahead of the face of the connecting gallery. An important part of the project was also devoted to blind predictions and their comparison with the in-situ measurements.

The CLIPEX project was co-ordinated by EIG EURIDICE. It was a joint project with ANDRA and ENRESA acting as main contractors and G3S, GEOCONTROL and UPM acting as associated contractors. The project was financially supported by EC, ANDRA, ENRESA and ONDRAF/NIRAS.

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Executive summary

One important step in the study of the feasibility of radioactive waste disposal in Boom Clay, is the demonstration that we can construct galleries by industrial technique keeping the disturbance of the host-rock at an acceptable level for the long term safety of the disposal. The use of a tunnel machine in plastic clays at great depth is really a challenge, since the convergence of the rock increases the risk of seizing the machine. The design and the construction of such a tunnel machine require a very good knowledge of the response of the host rock. The CLIPEX project has played a key role in the design of the tunnel machine and has allowed the full characterisation of the Excavated Disturbed Zone. The results of the CLIPEX project demonstrate that it is possible to construct the galleries by industrial technique at great depth in Boom Clay with a very good control of the disturbance of the host-rock around the gallery.

The CLIPEX project was realised in the frame of the extension of the underground research facility (HADES) in the deep Boom Clay formation at Mol (Belgium). The extension consisted in the realisation of a second shaft and a connecting gallery of 90 m length. The CLIPEX project aimed to provide the experimental data to characterise the short-term hydro-mechanical behaviour of the Boom Clay induced by the excavation of a gallery. As the connecting gallery was excavated from the second shaft to the existing facility, a unique and original opportunity was then given to monitor hydro-mechanical parameters ahead of the face of the advancing gallery. An important part of the project was also devoted to blind predictions and their comparison with the in-situ measurements.

The construction of the connecting gallery has been realised with a Tunnel Boring Machine (TBM) composed of a 2.3 m long shield, a road-header for the excavation of the rock and a bird-wing erector system for the installation of the lining. The shield was equipped with a cutting head to ensure a smooth excavated profile. The wedge block system was used for the lining, which is an expanded lining system. In order to reduce the plastic zone created by the tunnelling process, a fast advancing rate was achieved, the over-excavation was reduced to a minimum, and the lining was installed as soon as practically possible to minimise radial movement of the clay wall.

The CLIPEX project started with the characterisation of the host-rock in the zone to be excavated. This programme consisted in the realisation of pressuremeter, dilatometer and hydro-fracturing tests to measure the natural stresses, the permeabilities, the deformability and the strength of the rock mass. The results of this characterisation programme have allowed to choose the best parameter set and establish the hypothesis for the blind predictions.

The blind predictions of the hydro-mechanical behaviour of the clay massif during and after the construction of the connecting gallery were performed by five modelling teams: G3S, UPM, GEOCONTROL, SCK•CEN and ONDRAF/NIRAS using four different codes ANTHYC, ABACUS, FLAC and BOOMCLAY-code. The blind predictions consisted of two main steps:

- case studies to compare the predictions of the different codes for problems related with the excavation of underground structures. In those calculations, all the data were imposed: geometry, boundary conditions, constitutive model, geotechnical characteristics, and construction sequence;

- blind predictions of the hydro-mechanical behaviour of the clay mass during and after the construction of the connecting gallery. They quantified the evolution with time of pore water pressures, stresses and displacements in the medium as well as pressures on the gallery lining. Based on their experience, the modelling teams were free to use the data and the constitutive model they considered most appropriate.

The CLIPEX instrumentation programme aimed to assess the performance of the mechanised excavation techniques and the corresponding reduction of the plastic zone in the frame of a radioactive waste repository. For the first time in Boom Clay, it was possible to measure the hydro-mechanical response ahead of the face of the advancing gallery. The host-rock was therefore instrumented both in the zone to be excavated and around the gallery. The choice for the location and magnitude of the sensors was based on the results of the blind predictions. This led to the following considerations:

- pore water pressure and total stress measurements need to be performed from the axis of the connecting gallery (in the zone to be excavated) up to a radial extent of 12 m;
- displacement measurements need to be performed from the axis of the connecting gallery up to a radial extent of 7 m;
- sensors around the gallery need to be located in two planes (vertical and horizontal planes) in order to check for the possible stress anisotropy;
- maximum displacements of 12 cm along the gallery axis;
- a maximum radial convergence at tunnel circumference of 4 cm;

The instrumentation was installed from the Test-Drift (part of the HADES laboratory before its extension), from the second shaft and in the lining. The tunnel machine was also instrumented to measure the radial convergence profile on the wall between the excavated front and the last installed lining element.

Many lessons have been learned from the practical experience connected to the instrumentation programme and the related measurement performance. The design of the instrumentation programme based on modelling has proven to be very valuable. Measurement of pore water pressure through the piezometers and the displacements through extensometers and inclinometers have shown to be reliable and mature techniques. The measurement of total pressure in small diameter boreholes is still a pending issue.

The CLIPEX instrumentation programme has allowed us to obtain a very valuable data bank for the hydro-mechanical behaviour of Boom Clay. The pore water pressure and the displacements evolutions have been followed in three dimensions before and during the construction of a gallery. The sensor readings of the CLIPEX instrumentation installed from the front of the Test-Drift permitted to characterise the hydro-mechanical behaviour ahead of the excavated gallery. The instrumentation installed in the lining permitted to assess the hydro-mechanical behaviour of Boom Clay and interaction between the lining and the host rock. In addition, the convergence measured through the shield of the tunnel machine provided an accurate characterisation of the instantaneous behaviour of Boom Clay under decompression.

During the construction of the connecting gallery, systematic observation of the excavated wall was performed. The fracturation scheme has been fully characterised. This is a matter of

great significance, since the understanding of the fracturation has an important impact on the hydro-mechanical response of the host-rock.

The propagation of the fractures ahead of the excavated face was also observable. Fractures were opened ahead of the excavated front up to a distance of 4 m along the gallery axis.

It is the first time that pore water pressure could be measured outside the zone of influence of the HADES laboratory. This measurement allowed to confirm the initial undisturbed pore-water pressure about 2.2 MPa.

It was remarkable to observe effect of the different excavation phases from the pore water pressure and displacement measurements. This proves the sensitivity and the reliability of the measurements.

The in situ measurements indicated a very large extent of the Hydraulic Disturbed Zone. Indeed, variation of the pore water pressure were observed in the zone instrumented from the Test-Drift during the construction of the second shaft and from the beginning of the excavation of the connecting gallery. This means that pore water pressure reacts at distance greater than 60 m. This very surprising result is obviously of prime importance for the proper understanding of the hydro-mechanical behaviour of Boom Clay.

The numerical results of the different partners are generally in good agreement. Some discrepancies were pointed out considering the Cam-Clay model for the prediction of the axial displacement: the values obtained by SCK and GEOCONTROL are about twice as large as the value obtained by UPM and G3S. The fact that these discrepancies are only observed with the Cam Clay model points toward the conclusion that it is closely related to this model and its formulation/implementation. In this respect, a difference in integration rule for the Cam Clay model seems to be the most probable explanation.

The comparison between the prediction results, considering classical constitutive laws like modified Cam-Clay and Mohr-Coulomb model, and in-situ measurement showed a very good agreement for the displacements and the equilibrium pressure on the lining. The simulations can reproduce the tendency of the pore water pressure evolution but cannot explain the large extent of hydraulic disturbance. Of course the pore water pressure is much more sensitive to mechanical changes than the displacements. First tentative simulation with a more sophisticated constitutive law (Bounding surface elastoplastic law) has brought some hopes of improvement on the far-field behaviour, it however requires more work for the calibration of parameters.

The comparison of the blind predictions obtained by different partners showed that, when using a classical elastoplastic model, various aspects can still influence the numerical results: size of mesh, integration method of constitutive law, hydro-mechanical parameters, initial boundary conditions, etc. Based on the comparison, a 2D axisymmetrical model can produce quite challenging results with respects to a 3D model in considering the isotropic stress condition. The comparison of the numerical prediction with the in situ measurement indicated that improvements would be made in the future considering the following issues:

- A correct consideration of plasticity behaviour certainly improves the quality of prediction, especially the evolution of the pore water pressure;
- The consideration of time related effects through hydraulic diffusion process and viscosity of skeleton;

- The consideration of the fractured zone around the gallery, which can influence significantly the hydro-mechanical response.

The good prediction of the displacements and of the pressure on the lining has allowed to optimise the geometry of the tunnel machine and the design of the lining elements. Owing to this optimisation, only minor technical problems were encountered during excavation and an excavation rate of three meters per day was reached. The excavation rate can be easily increased up to 10 meters per day considering larger access shaft and minor adaptations to the excavation technique.

The results of the CLIPEX project showed that the technique used for the realisation of the gallery lining is acceptable both from an economical and technical point of view. The measurements of displacement and of equilibrium pressure on the lining proved that the hydro-mechanical perturbation has been minimised and that the technique allows a very good control of the excavation parameters. The measure of the pore water pressure and the observations demonstrate that opened fractures do not extend further than 0.5 meter in radial direction around the gallery. The observed total radial convergence, about 9 cm at the excavated wall, is considered acceptable in terms of hydro-mechanical disturbance. It is important to note that this convergence takes into account the convergence occurring ahead of the excavated front. The hydraulic variations, which can be measured from far away from the formation, have no impact on the long-term safety of a repository, since the permeability is not affected in this zone.

The CLIPEX project has therefore allowed a full characterisation of the instantaneous Hydro-Mechanical response around a gallery during its excavation, with a high level of confidence on the results. The measurements showed that the Hydro-mechanical disturbance is acceptable in respect of the long-term safety of a repository. The numerical simulations gave reliable blind predictions in terms of displacement and pressure on the lining. This simulation allowed the optimisation of the tunnelling machine. Future model developments are necessary to explain the variation of pore water pressure deep inside the clay formation.

The CLIPEX project was a great success and constituted an important step in the development and design of an underground nuclear waste repository. The CLIPEX project constituted also important issues and applications in the field of civil engineering. The results of the project have been disseminated in many international conferences.

List of abbreviations

ANDRA: Agence nationale pour la gestion des déchets radioactifs, France

CSL: Critical State Line

EDZ: Excavation Disturbed Zone

EIG EURIDICE : Economic Interest Grouping between NIRAS/ ONDRAF and SCK•CEN.
European Underground Research Infrastructure for Disposal of Nuclear Waste in Clay Environment

ENRESA : Empresa Nacional de Residuos Radioactivos SA, Madrid, Spain

G3S: Groupement pour l'étude des Structures Souterraines de Stockage, France

GEOCONTROL: GEOCONTROL S.A. Madrid, Spain

HDZ: Hydraulic Disturbed Zone

LCPC : l'Ecole Nationale des Ponts et Chaussées, Paris

MCC: Modified Cam Clay model

MC : Mohr – Coulomb model

NIRAS / ONDRAF : Nationale Instelling voor Radioactief Afval en Verrijkte Spleijstoffen /
Organisme Naional des Déchets Radioactifs et des matières Fissiles enrichies, Belgium

SBPT : The Self Boring Pressuremeter tests

SCK•CEN : Studie Centrum voor Kernenergie • Centre d'Etude de l'énergie Nucléaire

UPM: Universidad Politécnica de Madrid, Spain

1 Introduction

The R&D programme on geological disposal for high-level and long-lived waste (HLW) was initiated in Belgium at SCK•CEN in 1974. A deep tertiary clay formation, the "Boom Clay", present under the Mol-Dessel nuclear site, was selected as a candidate host formation for experimental purposes. An underground research facility, called HADES, was constructed and has allowed large scale integrated tests close to real conditions. The facility consists of one shaft and one horizontal gallery of 110 m length at 223 m depth.

The construction by the EIG EURIDICE (Economic Interest Grouping between NIRAS/ONDRAF and SCK•CEN) of a second shaft and of a 90 m long connecting gallery has started in June 1997 and was planned to be completed before year 2000. The CLIPEX project aimed to elaborate an instrumentation programme for this extension and to compare the experimental results with blind predictions.

The project started in January 1997 and covered initially a period of three years. It has been extended to the end of 2002 due to the important delay in the construction of the connecting gallery.

The CLIPEX instrumentation programme aims to assess the performances of the mechanised excavation techniques and the corresponding reduction of the plastic zone in the frame of a radioactive waste repository. For the first time in the Boom Clay formation, the displacements ahead of the face of the advancing tunnel have been measured that allowed to get the immediate convergence occurring during the excavation. A characterisation programme has also been performed to get more reliable data on the initial hydro-mechanical field conditions.

An important part of the project was also devoted to blind predictions of the hydro-mechanical behaviour of the clay massif during the construction of the connecting gallery. Five different modelling teams were involved: G3S, UPM, GEOCONTROL, SCK•CEN and ONDRAF/NIRAS.

The modelling programme of the CLIPEX project consists of:

- case studies to compare the predictions of the different codes for problems related with the excavation of underground structures. In those calculations, all the data were imposed: geometry, boundary conditions, constitutive model, geotechnical characteristics, and construction sequence;
- further calibrations of constitutive models and geotechnical properties for Boom Clay, from the existing laboratory and in situ data;
- blind prediction of the hydro-mechanical behaviour of the clay mass during the construction of the connecting gallery. They quantify the evolution with time of pore pressures, stresses and displacements in the medium as well as pressures on the gallery lining. Based on their experience, the modelling teams were free to use the data and the constitutive model they considered the most appropriate.

In this report, the results of the blind predictions are compared with the in-situ measurements. Discrepancies are discussed and a new tentative simulation with a more sophisticated constitutive law is given.

The lessons learned from this project are pointed out and a perspective research program is proposed at last part of the report.

2 The characterisation programme

In order to better characterise the clay layer to be excavated, an in-situ testing programme consisting in the measurements of natural stresses, permeabilities, deformability and strength of rock mass was planned.

This characterisation programme consists of Pressuremeter, Dilatometer, Self Boring Pressuremeter (SBPT) and Hydrofracturing tests from the front of the Test-Drift. A summary of the whole fieldwork is given in Table 2.1.

Table 2.1 : Summary of the whole characterisation fieldwork

Borehole	Test	Depth from the front of Test Drift (m)			
B-1	Pressuremeter	12,3	15,8	---	---
	Dilatometer	13,1	15,1	17,1	19,1
	Hydrofracture	12,5	14	15,5	17,5
B-2	Pressuremeter	12,5	14,6	17,4	19
	Dilatometer	12	14,2	---	---
	Hydrofracture	12	14	16,5	---
B-3	Pressuremeter	11,7	12,5	15,5	18,3
	Dilatometer	11,9	15,1	---	---
	Hydrofracture	12,5	14,5	16	17,5
B-4	SBPT	6 tests were performed between 20 and 30 m			

2.1 The dilatometer and pressuremeter tests

The results of these tests have allowed to determine the shear modulus G in pressuremeter and dilatometer tests (Figure 2.1). The average result ranges between 45 and 57 MPa for borehole B1, depending on the direction of the measurement. For borehole B2, the average result is between 41 and 53 MPa. Finally, for borehole B3, it is within a range of 35-48 MPa. Maximum module is usually measured on vertical direction. Shear moduli in unloading have also been calculated from dilatometer tests, with an average value of about 120-130 MPa. These moduli are very homogeneous, regardless of the borehole and depth of the test and of the measured direction. Yield pressure has also been calculated, and the average result is about 40-50 MPa.

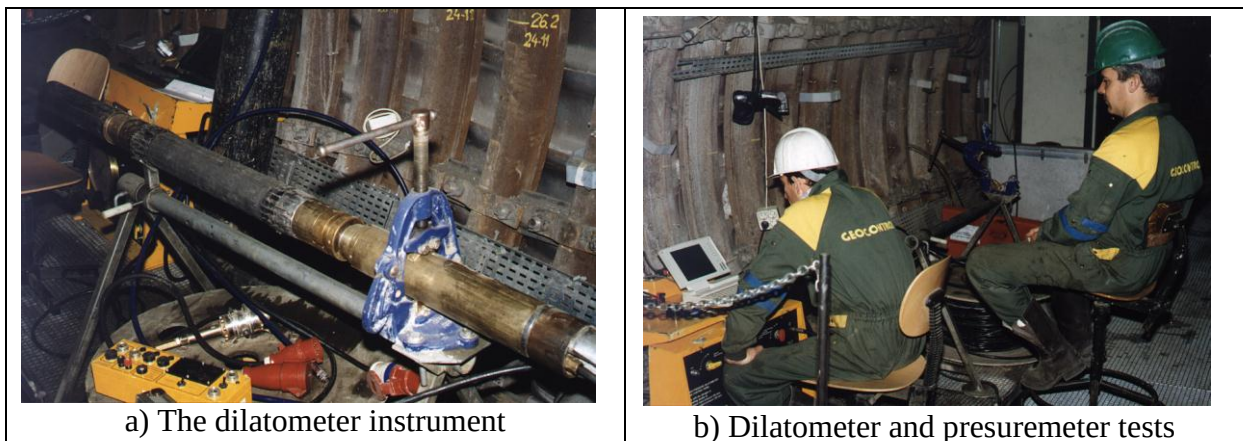


Figure 2.1 : Dilatometer and pressuremeter tests

2.2 The Hydrofracturing tests

Although none of the hydrofracturing tests (Figure 2.2) demonstrated the initiation of new fractures with clear breakdown events, the hydrofracturing tests yielded reliable "refrac - and - shut - in" pressure data. The impression packer tests conducted showed that in most tested sections axial, sub-vertical fractures with similar orientation were stimulated. Therefore, the stress analysis was carried out on the basis of the classical HUBBERT and WILLIS (1957) concept for horizontal principle stress S_h . Neglecting the effect of the ambient pore pressure, the results of the calculations can be summarised in Table 2.2.

Table 2.2 : The minimum horizontal stress (S_h) and the vertical stress (S_v)

Depth (m)	S_h (MPa)	S_v (Mpa)
225	2.55 ± 0.24	5.23 ± 0.54

The direction of the maximum horizontal stress S_h was determined as $N 175^\circ + 6^\circ$. For the estimation of its magnitude further hydrofrac tests in a vertical borehole are required.

The analysis of the short pressure pulse tests conducted prior to the hydrofrac tests yields a mean hydraulic conductivity of $(4.3 + 1.0) \cdot 10^{-10}$ m/s for the Boom Clay in the close vicinity of the test boreholes. The high conductivity together with the lack of fracture initiation events during the hydrofrac tests indicate that the rock in the vicinity of the boreholes was slightly damaged by the drilling process.



Figure 2.2 : The hydrofracturing test

2.3 The Self Boring Pressuremeter tests

The SBPT tests (Figure 2.3) have been realised successfully between 20th and 25th January. Six expansion tests were realised at depths between 20 and 30 m along a single horizontal borehole extending from the end face of the Test-Drift. The position of the borehole was 0.6m

under and 1.05 m to the left of the tunnel axis. The borehole was angled at approximately 10 degrees as viewed from above.

Prior knowledge of the site conditions indicated to choose a weak rock configuration with a six arm self-boring pressuremeter. Pore water transducers were also fitted.

The fieldwork was actually separated in two parts. The first part, in November 1998, ended prematurely when it was discovered that modifications to the self-boring drilling system (using air flush) were required. The second part, mentioned above, was completed after the necessary modifications.

All six tests produced meaningful results. The analyses suggest that the Boom Clay has a large non linear-elastic range, a shear modulus about 150 - 200 MPa, shear strength of about 2000 and 2300 kPa with in-situ stresses of around 3400 kPa at the location of the measurements. Vertical and horizontal stress anisotropy is also quite small. All the tests produced good pore water data and the tests can be shown to be undrained. Once reached, the maximum radial effective stress remained almost constant at 2000 kPa during the tests.



Figure 2.3 : Selfboring Pressuremeter instrument

2.4 Comparison of the results

Results from the Pressuremeter test (PT), the Dilatometer test (DT), the Hydrofracturing test (HT) and the Self-Boring-Pressuremeter test (SBPT) are compared in Table 2.3.

Table 2.3 : Comparison of the results of the characterisation programme

Mpa	PT	DT	HT	SBPT
Shear mod. (L)	42/69	66/72		
Shear mod. (U)		89/167		83/225
Shear strength				2/2.3
Yield pressure	4.5/6.7			
Vertical Stress			5.23	3 N.L.
Horizontal Stress			2.55	3.4 N.L./ 4.3 L.

The results of the dilatometer, the pressuremeter and the SBPT are in good agreement. Vertical and horizontal stresses determined respectively by the hydrofracturing tests and the SBPT are quite different. Results from SBPT look more consistent.

The results obtained from this characterisation programme were helpful for the modelling teams for the choice of the rheological laws of the ground and the determination of the parameters.

2.5 *Conclusions*

The programme of characterisation allowed us to get valuable information on the natural stresses, the permeabilities, the deformability and the strength of the host-rock in the zone to be excavated. In addition, it allowed to choose the best parameter set and establish the hypothesis for the blind predictions.

3 Modelling teams and corresponding numerical codes

The numerical codes used by the various modelling teams are given hereafter:

3.1 *SCK•CEN and GEOCONTROL: FLAC code*

The code FLAC is an explicit finite difference program and simulates the behavior of structures built of soil, rock or other materials. FLAC has found his wide fields of application related to the geotechnical problems in the field of mining, underground engineering and rock mechanics and research.

3.2 *G3S : CESAR and ANTHYC codes*

The code CESAR (developed by LCPC) was used for the preliminary modelling (Test cases). The final blind prediction was however realised with an internal code named ANTHYC.

CESAR is a finite element programme allowing computations on porous media with coupled effects.

ANTHYC allows 1, 2 or 3D modelling of mechanical equilibrium coupled with several transfer phenomena: thermals, hydraulics and/or chemistry. The code is dedicated to the study of porous media and rock mechanics, and can thus solve problems in the framework of Thermodynamics of Open Continuum Media in transient or steady state regimes.

Both anisotropy and non-linear material properties can be included when considering transfer phenomena. The code allows taking into account not only diffusive but also convective transport terms.

The mechanical resolution is quasi-static. The mechanical behaviour may be elastic, plastic and/or viscoplastic. Moreover, mechanical anisotropy and non-linear properties (stiffness, thermal expansion, ...) can be considered equally.

The code allows choosing between Finite Elements or Finite Volumes calculations. But both formulations are based on a Finite Element interpolation. When discretized, these formulations lead to a first order differential system with respect to time, which can be linear or non-linear. It is solved through a direct implicit/explicit integration method. In case of non-linear problems, the choice of a non-explicit resolution naturally leads to a non-linear problem resolution at each time step: a simple method of substitution is used to solve it.

3.3 *UPM : ABAQUS code*

ABAQUS is a suite of engineering simulation programs, based on the finite element method that can solve problems ranging from simple static analyses to the most challenging non-linear simulations. Designed as a general-purpose simulation tool, ABAQUS can be used to study more than just structural (stress/displacement) problems. It can simulate problems in such diverse areas as heat transfer, mass diffusion, thermal management of electrical components (coupled pore fluid-stress analyses), and piezoelectric analysis.

3.4 *NIRAS/ONDRAF: BOOMCLAY-code*

The BOOMCLAY-code is a 1-D finite element code where a rheological model specific to Boom clay is implemented.

4 The preliminary modelling (Test cases)

In order to compare the predictions by different codes, two test cases studies related with the excavation of underground structures have been modelled by different modelling teams: G3S, UPM, GEOCONTROL, SCK•CEN and ONDRAF/NIRAS. In those calculations, all the data were specified: geometry (except the mesh), boundary conditions, constitutive model, geotechnical characteristics, and even construction sequence.

4.1 First test case (see appendix 1: Data for Test case 1)

The first test case consists of a uniaxial (1D) excavation problem to be solved using the Mohr-Coulomb and the Modified Cam-Clay model. Some of the modelling teams have performed complementary calculations with other data sets.

Figure 4.1 and Figure 4.2 show the most significant results and the comparison between partners. We can see that the results are in good agreement between the partners. Some interesting results to be pointed out are:

- at a radial distance of 6 m (= plastic zone extent), the tangential stress (σ_t) reaches its maximum, and the water pressure keeps its initial value (2,25 MPa).
- the radial displacement at $r = 2$ m is 7 cm, and at 6 m it is 2 cm.

This first test case aimed at comparing the different codes rather than the constitutive models.

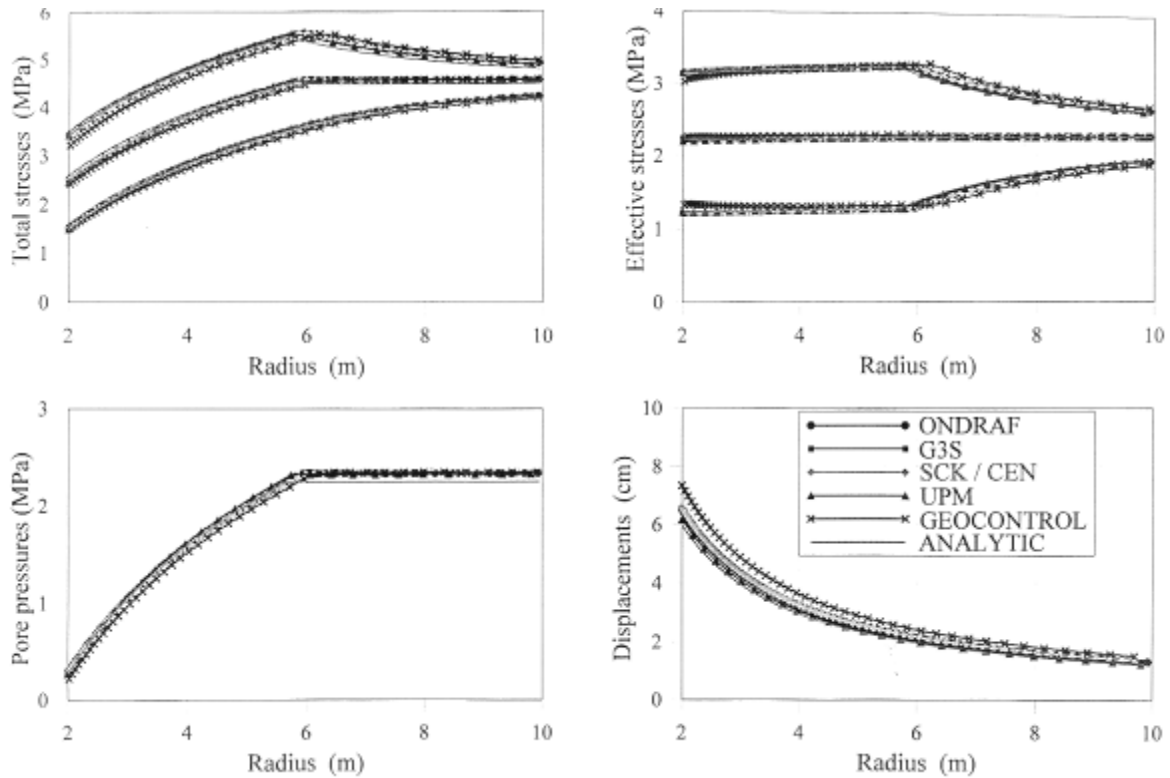


Figure 4.1 : Results of test case 1 for a Mohr-Coulomb model

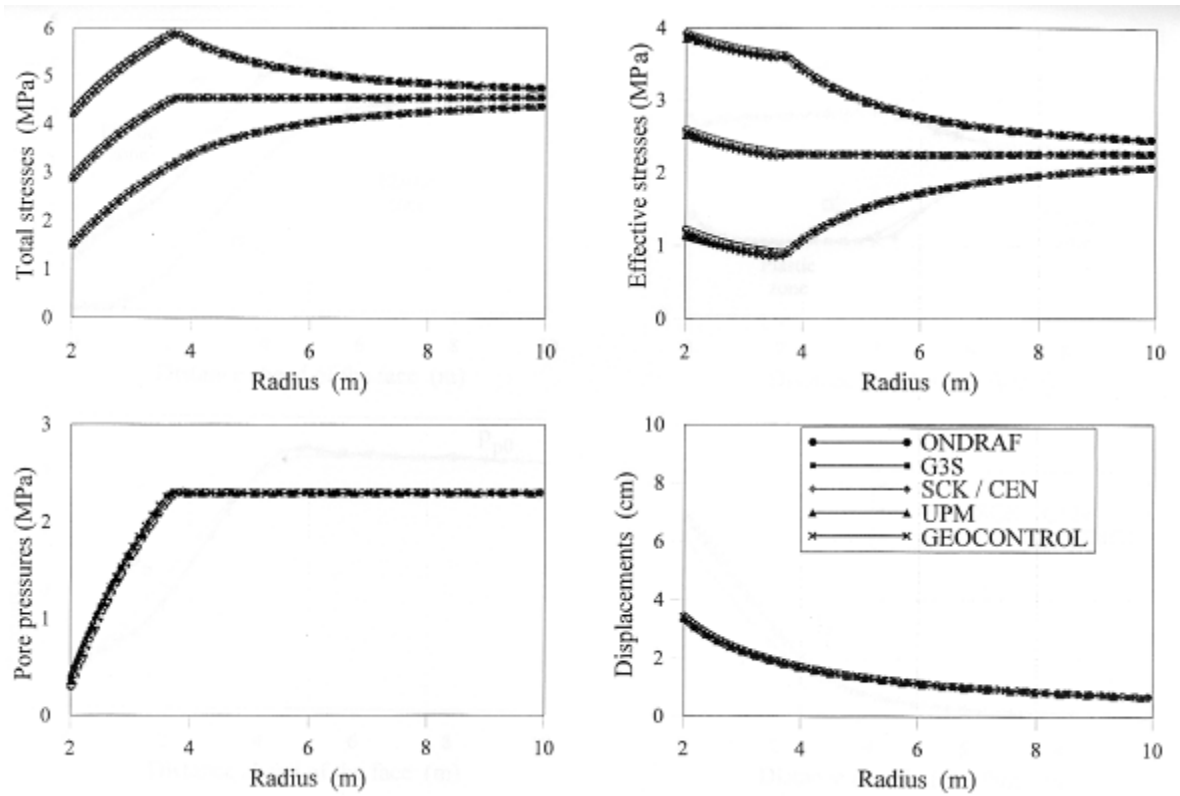


Figure 4.2 : Results of test case 1 for a Cam-Clay model

4.2 Second test case (see appendix 2: Data for Test case 2)

The second test case is much closer to the in situ reality that would be encountered during the construction of the connection gallery. It accounts amongst others for the decompression of the clay mass ahead of the excavation face, for the support laying distance and for the sequential tunnelling process.

The geometry and the boundary conditions of this test case are given in Figure 4.3. Since the supporting system of the connecting gallery was not decided yet at that time, a lining consisting of cast iron segments (internal radius 1.75 m, segments thickness 16 cm) has been assumed for the modelling.

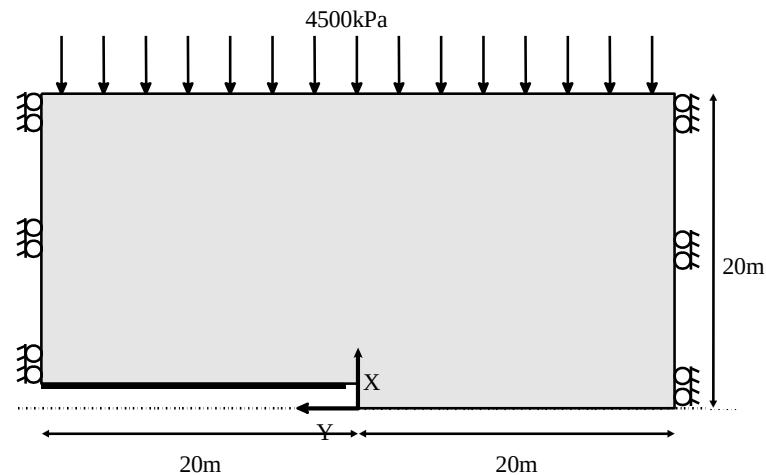


Figure 4.3 : Geometry and boundary conditions of the 2-D axisymmetric test case.

According to the working sequence, the following assumptions were made in the calculations (see Figure 4.4): the entire cross section of the gallery is excavated in a single step by semi-mechanical means (backhoe of roadheader type of machinery); the rounds are 0.5 m long and after each round a 0.5 m wide ring is installed; the distance between the front and the segments varies within 0.25 and 0.75 metre; the tunnel advance rate is about 2 m/day.

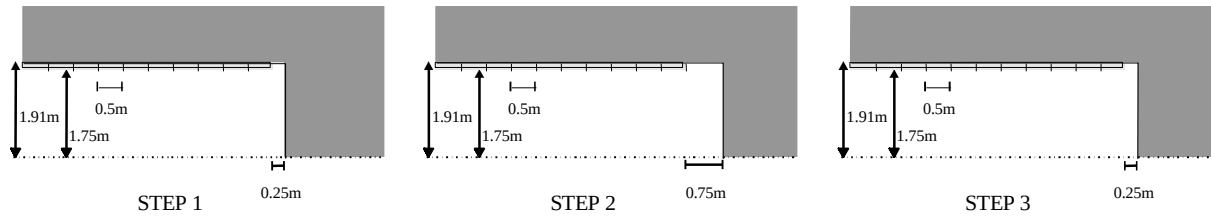


Figure 4.4 : Schematic view of the sequential tunnelling and lining process

Calculations were performed considering two different constitutive models:

- a linear-elastic perfectly-plastic model with a Mohr-Coulomb failure criterion using the following geomechanical characteristics.

Table 4.1 : Mohr-Coulomb parameters

Young's modulus	E	300 MPa
Poisson's ratio	ν	0.125
Friction angle	ϕ_c	18°
Cohesion	C	0.3 MPa
Dilation angle	ψ	0°

- a modified Cam-clay model using the following geomechanical characteristics

Table 4.2 : Cam-clay parameters

Slope of the normal consolidation line	λ	0.13
Slope of the elastic swelling line	κ	0.02
Critical state friction angle	ϕ_c	21°
Preconsolidation pressure	P_c	6 MPa
Poisson's ratio	ν	0.2
Specific volume at 1 kPa on the NCL	N	2.75

The presence of some gap between the outer face of the lining and the excavation wall has a significant influence on the equilibrium state around the gallery, and more particularly on the magnitude of the hydro-mechanical disturbances. To appraise the importance of this factor, two calculations were carried out (for both constitutive models): one without any over excavation gap and the other with an over gap of about 5 cm.

Since the CLIPEX instrumentation programme was mainly planned to assess the hydro-mechanical disturbances arising during the excavation of the connecting gallery, this case study consequently merely focused on the gallery construction and didn't evaluate the hydro-mechanical changes arising from the pore water pressure dissipation after excavation. Owing to the very low permeability of the Boom Clay formation, a 2 m/day of advancing rate can be assumed as very fast and accordingly the response of the medium to the excavation can be idealised as undrained.

Figure 4.5 and Figure 4.6 show the comparison of the axial displacements ahead of the face and of the radial displacement ten meters behind the front. Typical results are axial displacements ahead of the face up to 10 cm (non gap) and up to 15 cm (with gap), stresses up to 5,5 MPa, a plastic zone extending radially from about 4 m (no gap) up to 7 m (with gap), a radial convergence near the lining of 4 cm (no gap) to 9 cm (with gap).

We can see that considering the Mohr Coulomb model the results are in good agreement between the partners.

Considering the Cam-Clay model, the results are in agreement between SCK•CEN, GEOCONTROL and G3S for the cross section 10m behind the front but important discrepancies are obtained for axial displacements at the tunnel face: the values obtained by SCK•CEN and GEOCONTROL are about twice larger than the values obtained by UPM and G3S.

Several assumptions can be put forward for explaining such difference:

- the code run by SCK•CEN and GEOCONTROL uses a large strain formulation whereas the codes run by UPM-G3S uses a small strain formulation,
- the mesh size around the gallery is different between the modelling teams,
- the discretisation type used (mixed-type in the code run by SCK•CEN and GEOCONTROL),
- or the rule used for the integration of the Cam clay constitutive relation (Forward Euler scheme used in the code run by SCK•CEN and GEOCONTROL).

The influence of the small compared to the large strain formulation has been investigated at SCK•CEN and its influence is negligible in the present case. The respective influence of the three remaining assumptions (mesh size, discretisation type and integration rule) is difficult to evaluate in our working programme. However, the fact that this discrepancy is only observed with the Cam clay model points toward the conclusion that it is closely related to this model and its formulation/implementation. In this respect, a difference in integration rule for the Cam clay model seems the most probable explanation (for its influence, see e.g. Gens and Potts, 1988).

Additionally, SCK•CEN has realised a parametrical study, which shows that the initial pore water pressure and the pore water bulk modulus have a large influence on the results. The pore water bulk modulus varies according to the amount of gas dissolved in the water, but is very difficult to determine accurately.

4.3 Conclusions

The preliminary modelling in the test case 1 showed a good agreement between partners. Concerning the Test case 2, the modelling results showed that considering the Mohr-Coulomb model, the results are in good agreement between partners. The important discrepancies are obtained for axial displacements when considering Cam-clay model.

Through these preliminary modelling, some interesting first results of the hydro-mechanical disturbances induced by the excavation of the connecting gallery have been obtained such as the plastic zone extent, radial displacements range, pressure exerted on the lining, etc., which furnished a important basis for the design of an efficient instrumentation configuration.

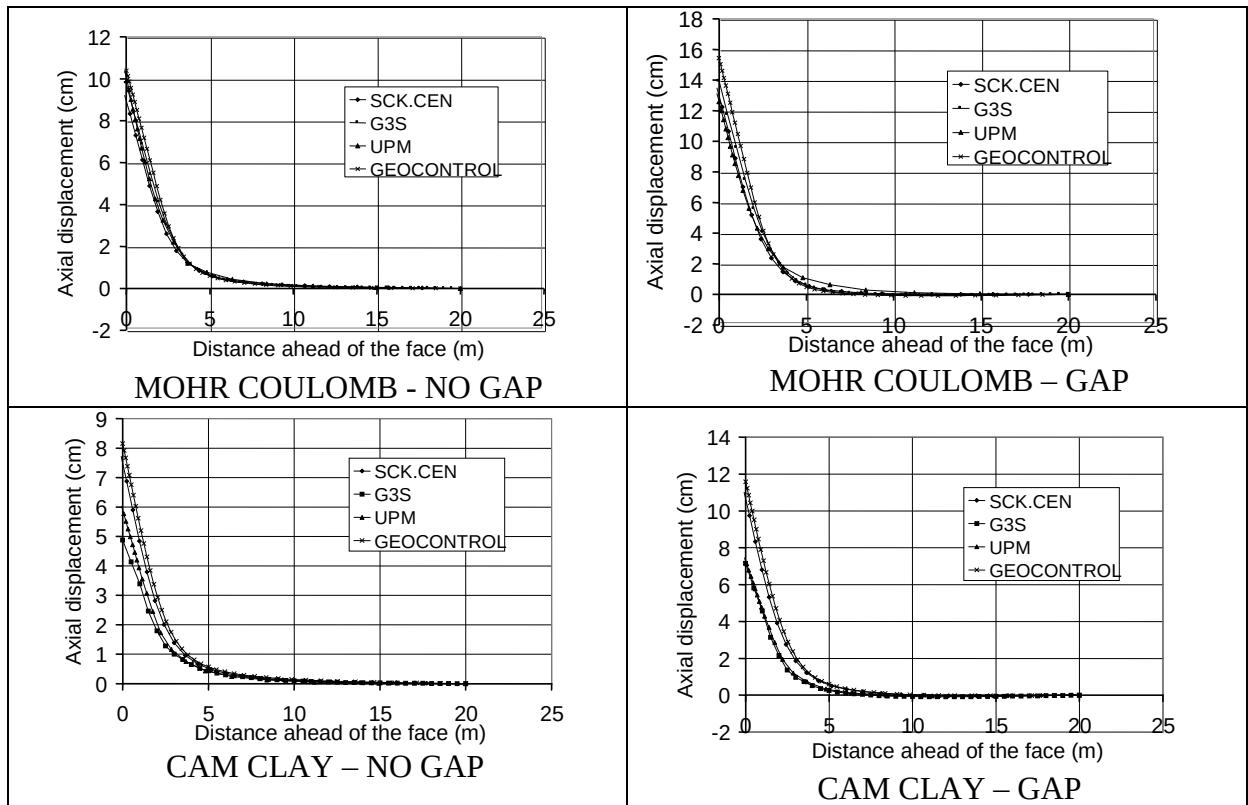


Figure 4.5 : Axial displacement ahead of the face

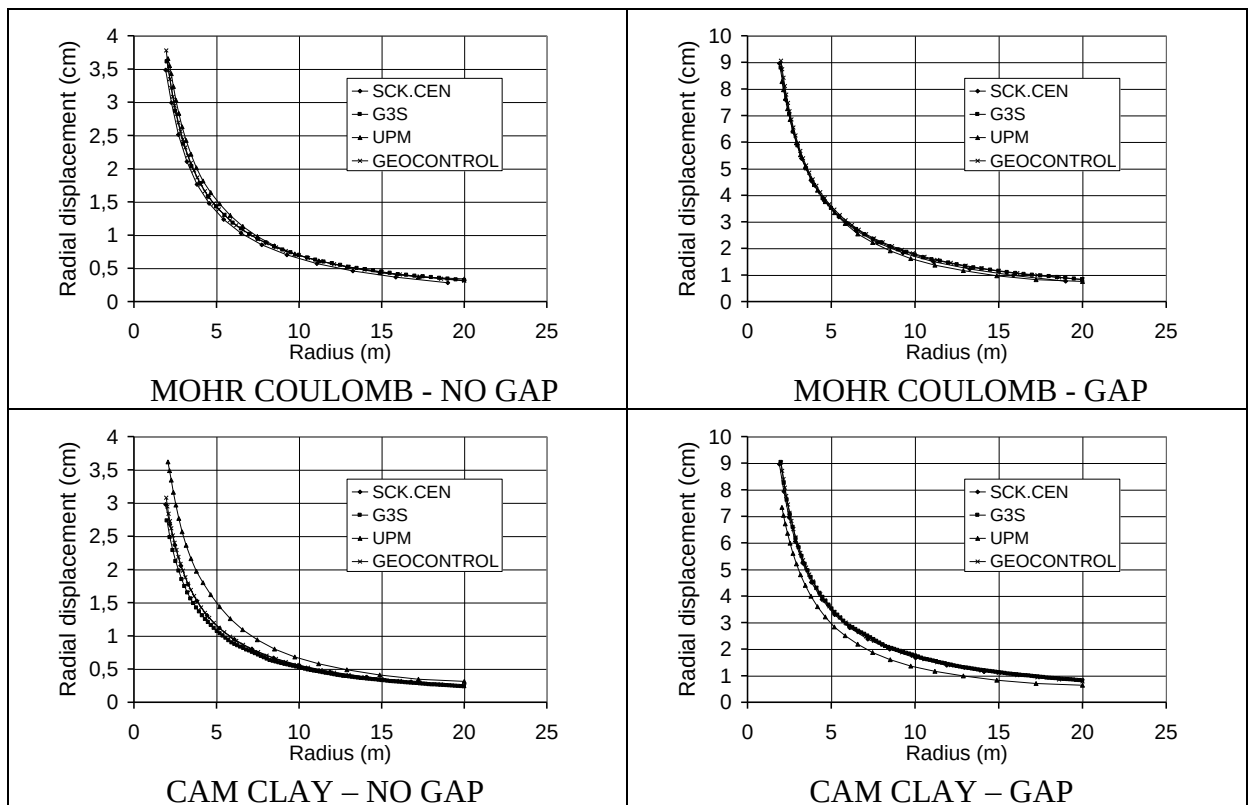


Figure 4.6 : Radial displacement 10 m behind the front

5 The instrumentation programme

The CLIPEX instrumentation programme aims to provide experimental data to assess the short term hydro-mechanical behaviour of the Boom Clay induced by a gallery excavation. As the connecting gallery is excavated from the second shaft, an unique and original opportunity has been given to monitor, from the existing facility (Test Drift), the evolution of hydro-mechanical parameters ahead of the face of an advancing gallery (see Figure 5.1). Although this problem has been extensively studied on a theoretical point of view, only few experimental data exist, especially regarding the quantification of the initial convergence u_{ri0} (Panet, 1995).

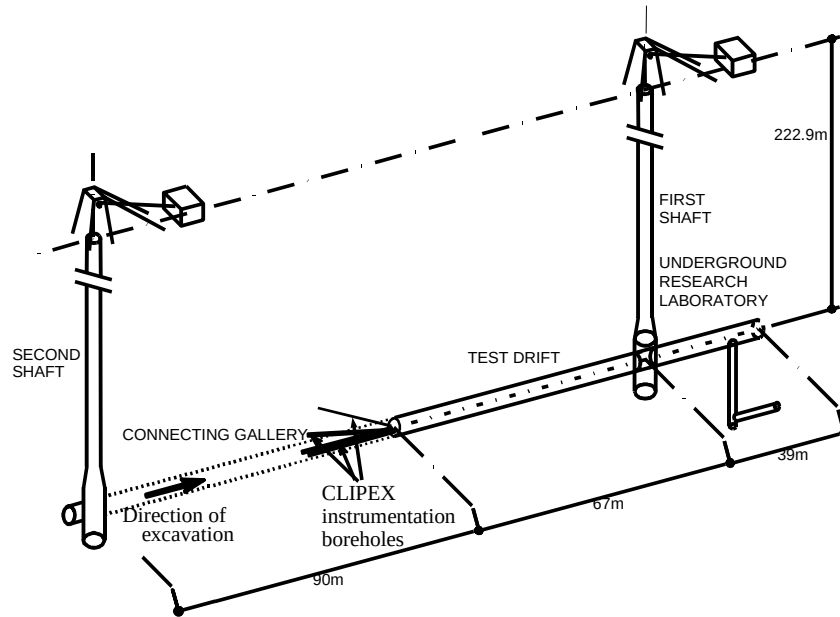


Figure 5.1 : The extension of the underground laboratory

The obtained experimental results should allow us to assess the performances of mechanised excavation techniques and the corresponding reduction of the plastic zone in the context of a HLW repository. They have to give field data that should lead to a better knowledge of the Boom Clay behaviour and consequently to validate and improve the numerical modelling of the excavation sequences. The measurements around the gallery have to give more representative data on the initial hydro-mechanical field conditions for designing, modelling and interpreting future experiments.

5.1 Choice of the location and of the magnitude of sensors

The numerical simulations of test case 2 (see §4.2) delivered us with the first results to assess the hydro-mechanical disturbances induced by the excavation of the connecting gallery. With these results, we designed the instrumentation programme:

- by defining optimal locations for the instruments,
- by assessing the expected magnitude of the measurements.

The results of test case 2 have led to the following conclusions (see Figure 5.2):

- the radial plastic zone extent lies between 4 m (no gap) and 7 m (with gap)

- axial displacements ahead of the face of the gallery up to 9 cm (no gap) and 12 cm (with gap)
- a total stress up to 5.5 MPa
- an initial radial convergence at tunnel circumference between 2.5 cm (no gap) and 4 cm (with gap)
- a pressure on the lining between 1 MPa (with gap) and 2 MPa (no gap)

Numerical simulations have also been performed considering a more sophisticated constitutive model (the two-surface “bubble” model developed by Al Tabbaa and Wood, 1989). This model is actually an extension of the modified Cam-clay that incorporates a bounding surface and a yield surface. The evolution of both surfaces is described through an isotropic hardening law, and the yield surface also evolves through a kinematic hardening rule. At this stage, only a 1-D axisymmetric problem has been studied (Labieuse 1997). The results are presented in Figure 5.3. We observe that the profiles of stresses are smoother, the hydraulic disturbance extends much further in the clay mass and the convergence is more important.

These preliminary numerical predictions have lead to the following choice for the location and magnitude of the sensors:

- pore water pressure and total stress measurements will be performed from the axis of the connecting gallery (in the zone to be excavated) up to a radial extent of 12 m;
- displacement measurements will be performed from the axis of the connecting gallery up to a radial extent of 7 m;
- sensors around the gallery will be located in two planes (vertical and horizontal planes) in order to check for a possible stress anisotropy;
- maximum displacements of 12 cm are expected along the gallery axis;
- a maximum radial convergence at tunnel circumference of 4 cm is expected;

These specifications led to four instrumentation zones from the Test Drift front; in each zone, pore water pressure, total stress and displacement measurements were applied. In addition, two boreholes from the second shaft for pore water pressure and displacement measurements respectively, and monitoring on and in the gallery lining of the connecting gallery would give complimentary data (see Figure 5.4).

The CLIPEX instrumentation programme therefore covers three parts: the first (and most extensive) part consists of eight instrumented boreholes drilled from the Test Drift front, the second part consists of two instrumented boreholes drilled from the second shaft, and the third part consists of the instrumented segments from the gallery lining.

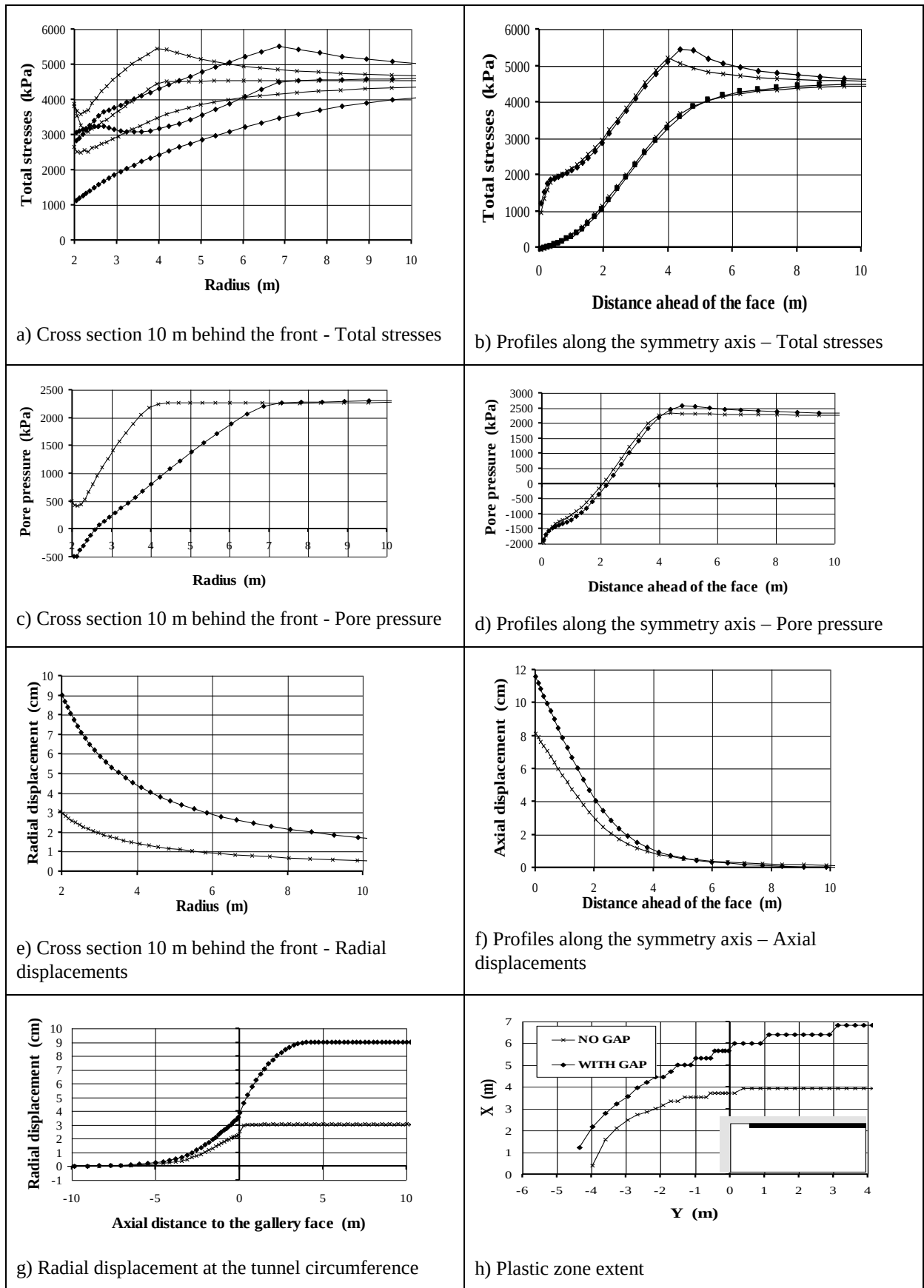


Figure 5.2: Results obtained from numerical simulations based on the 2-D test-case model

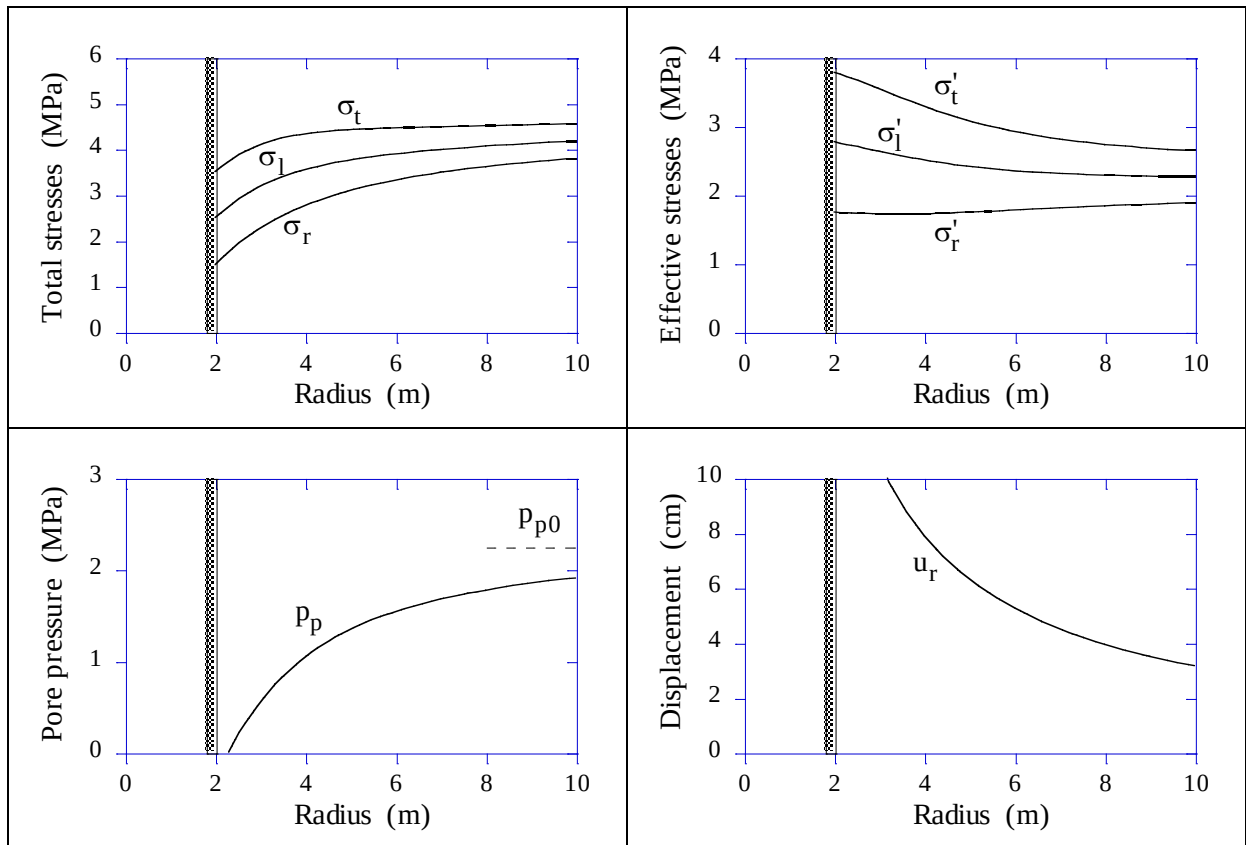


Figure 5.3: Results obtained from numerical simulations based on a 1-D test-case model with the two-surface “bubble” model, from (Labieuse, 1997)

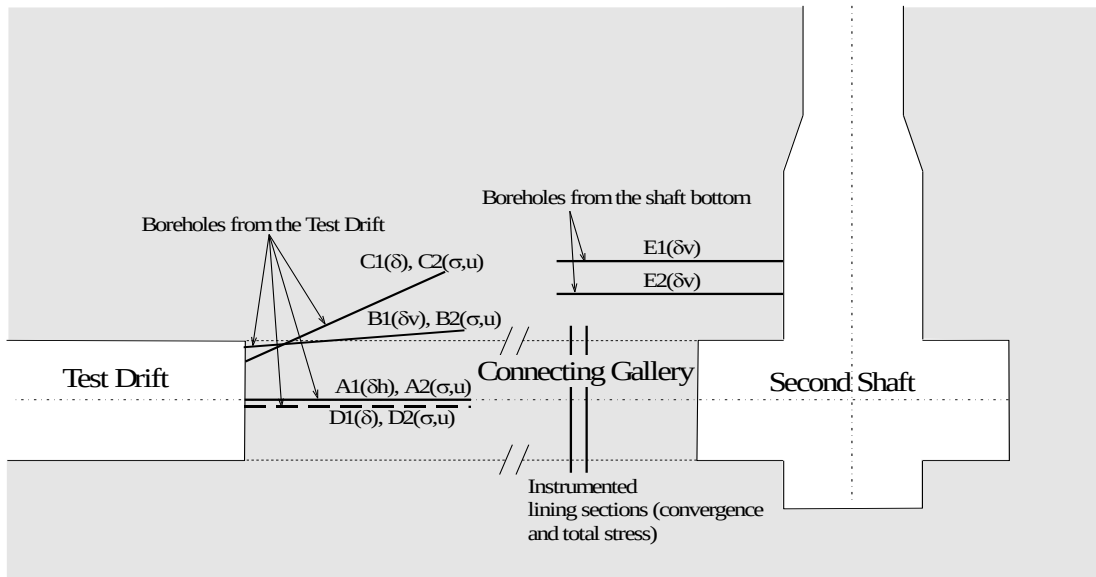


Figure 5.4: CLIPEX instrumentation program (not on scale)

5.2 Instrumentation installed from the Test Drift front

From the Test Drift front, four zones to be instrumented were identified:

- zone A in and near the axis of the connecting gallery;
- zone B slightly inclined upward, to make observations near the future gallery lining;

- zone C with a larger upward inclination, above zone B;
- zone D in the horizontal plane of, and diverging away from, the connecting gallery.

In each zone, one borehole for displacement measurements, and another borehole for pore water and total measurements has been foreseen, resulting in four boreholes for displacement measurements (A1 to D1), and four boreholes for pore water and total stress measurements (A2 to D2). The location of these boreholes is shown in Figure 5.5.

Borehole A1 is located near the axis of the connecting gallery; the extensometer installed in this borehole will monitor the axial displacements along the borehole.

Borehole B1 is located in the vertical plane above the gallery, and is only slightly inclined (some 4° upward). An inclinometer is located in this borehole to monitor the vertical movements (radial movement towards the gallery).

Borehole C1 is in the same plane as B1, but is more inclined (11° upward). It also contains an inclinometer.

Borehole D1 is located in the horizontal plane of the connecting gallery; it also measures the radial displacement towards the gallery, by means of a deflectometer.

Boreholes A2 to D2 are paired to boreholes A1 to D1, and all have a similar design, consisting of 6 filters (8 for C2) to collect pore water, measured by pressure transmitters in the gallery, complemented with some sections with total stress cells.

An extensive description of this instrumentation is given in the topical report covering Work Package 2 (EIG PRACLAY, 1999).

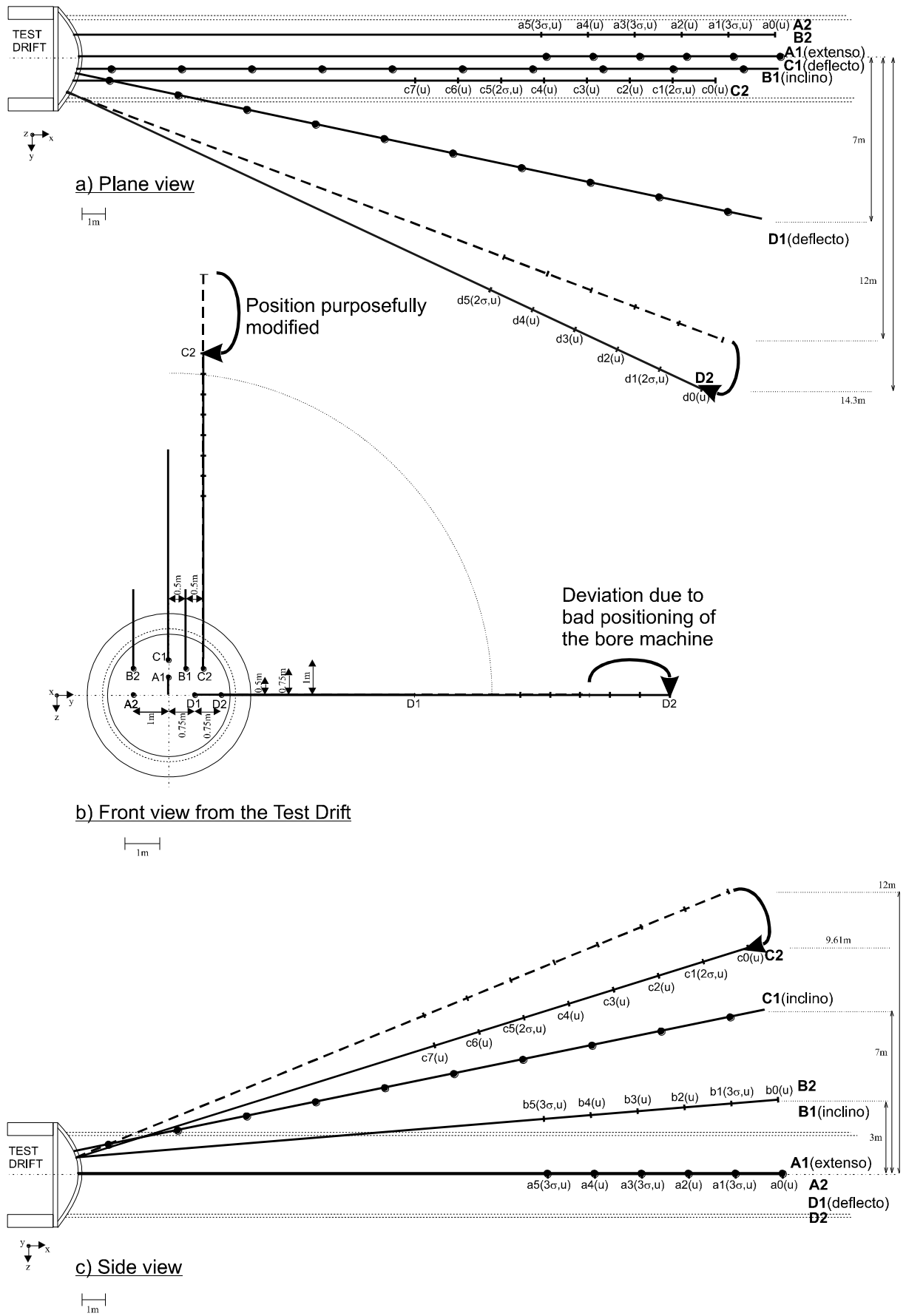


Figure 5.5: Position of the instrumented boreholes

5.2.1 Displacement measurements

5.2.1.1 Extensometer : A1

5.2.1.1.1 Description

The extensometer selected for borehole A1 consists of six individual anchors, connected to displacement transducers located at the Test Drift Front. The whole set-up is installed in a borehole of 30 m deep and 150 mm diameter.

Each anchor consists basically of an inflatable geotextile packer. During installation, the packers are inflated with a water-cement mixture to create a solid bond between anchor and host rock. Steel rods (14 mm diameter), inside a protective PE tubing (20 mm outer diameter, 2 mm thick), transfer the displacement of each anchor to the potentiometer displacement transducers. The rods are kept in place by centring discs. Also the grout tubing is connected to each anchor.

At the Test Drift Front, six potentiometer displacement transducers are located on a mounting plate with protective cover. The transducers have a measuring range of 200 mm. Because we expect an elongation (due to both the continuous decompression of the Test Drift Front and the excavation of the connecting gallery, we divided the measuring range in a compression range of only 20 mm and an extension range of 180 mm.

5.2.1.1.2 Installation

Borehole A1 was drilled at a diameter of 150 mm up to 30 m, followed by installation of the extensometer, consisting of the six packer anchors (installed between 20 and 30 m depth, as shown in Figure 5.6), connecting rods with protective tubing, and grouting tubes. To guide the rods along the length of the borehole, centring disks were placed at regular intervals between the anchors and the borehole mouth. After installation, the packer anchors were grouted to create the bond with the host rock. All these works were performed on 11 May 1998.



Figure 5.6: Installation of an extensometer anchor packer

After insertion of the extensometer equipment, we filled the remaining space of the borehole to limit convergence of the surrounding clay. The normal filling procedure consists of pumping a bentonite/cement mixture from the borehole mouth, while the air is vented through an air purge tube installed with the extensometer. Probably due to some obstruction caused by

the centring disks and the anchors, only the first 15 m of the extensometer borehole have been filled completely. We did not expect that this would cause any major problem since the clay will have sufficient time to creep (although in a less predictable way) around the extensometer. It should be noted that the manufacturer (Boart-Longyear) did not propose any way to fill the borehole as it was, according to this company, not essential to the correct functioning of the instrument.

5.2.1.1.3 Data acquisition

All displacement transducers were read out by Campbell dataloggers. The measurement values were sent to the supervising PC through a dedicated data acquisition programme. The measured values were then sent to the central database.

5.2.1.1.4 Measurement performance

After installation, we noticed a movement of all anchors of about 1 mm/month. This was due to the movement of the Test Drift front, in which the measurement head with the displacement transducers was located. This displacement was in line with the movement as determined by optical surveying.

A few months after installation, several transducers started to show erroneous readings. After removing the protective cap of the measurement head (Figure 5.7), we noticed that several transducers were damaged due to differential movement of the shotcrete shell on which the measurement head was installed. There were also clear signs of corrosion.

After repair of the extensometer head (1 Feb 2000), five out of six transducers showed again the initial behaviour (movement of Test Drift Front) – up to the actual excavation of the connection gallery. Only the anchor at 24 m (sensor EA4) seemed to be blocked and hardly moved.

During the actual excavation, the displacements of the anchors were lower than expected. Moreover, all five sensors showed a similar extension, questioning therefore the independent behaviour of each anchor.



Figure 5.7: Damaged displacement transducers of the extensometer

5.2.1.2 Inclinator: B1

5.2.1.2.1 Description

The inclinometer set-up consists of a cased borehole of 30 m long. The casing is made up of 3-m long ABS-segments (OD 70 mm, ID 60 mm, with groove), to be glued together (Figure 5.8).

Inside this casing, a fixed chain is installed. The chain is made up of ten 3-m long beams; each beam is equipped with an uni-axial tilt-sensor (electro-level) with accompanying temperature sensor. Each tilt sensor has a measuring range of 50 mm/m up/down (this corresponds with $\pm 3^\circ$). Because of the initial borehole inclinations, we specified an offset of 4.3° for the sensors.



Figure 5.8: Inclinator casing and sensor

5.2.1.2.2 Installation

On 14 May 1998, borehole B1 was drilled at a diameter of 150 mm up to 30 m (a larger diameter gives the drillmaster more control over the direction of the borehole due to a stiffer chain of drill rods). The inclinometer ABS casing was then assembled and inserted into the borehole. To keep the borehole centralised in the borehole, centralisers were attached to the casing. The remaining space between the casing and the borehole wall was filled with a cement-bentonite mixture. Before installing the inclinometer chain (28 May 1998), a topographical survey was carried out within the casing to obtain the initial deviation, which can then be correlated to the initial reading of the sensors. The topographical survey was also carried out for all the other boreholes and is given in the topical report of WP2.

5.2.1.2.3 Measurement performance

Stable measurements, with only minor variations, were obtained after the installation. The casing showed however a serious leak – several hundreds of ml of muddy water flowing out of the casing. During the excavation, meaningful measurement data were obtained, but the interpretation has proven to be cumbersome due to the lack of a clear reference point (both borehole ends have moved during the excavation works).

5.2.1.3 Inclinator : C1

5.2.1.3.1 Description

The inclinometer set-up is similar to B1, except for the sensor offset, which is 11.0° .

5.2.1.3.2 Installation

Borehole C1 was drilled on 14 May 1998 following the same procedure applied for borehole B1. However, the filling of the borehole was not complete due to the plugging of the air purge tube (only the first 15 m or so are filled). Since the Boom Clay will have enough time to creep around the casing, this was not considered as a major problem. The inclinometer chain was installed two weeks later, on 28 May 1998.

5.2.1.3.3 Measurement performance

A few weeks after the installation measurements, several sensors showed large variations, some of them going out of range. As no significant movements were expected, we think that the incomplete grouting of the borehole might have caused irregular creeping of the clay around the casing, thereby causing large local deformations. One thermistor also showed a broken signal cable. An effort to remove the inclinometer chain out of the casing proved unsuccessful, further indicating the deformation of the inclinometer casing.

The casing also showed some leakage – but far less significant than the B1 casing.

During the excavation, some meaningful measurement data were obtained. Only a partial interpretation is possible due to the fact that some sensors give out of range values.

5.2.1.4 Deflectometer : D1

5.2.1.4.1 Description

The deflectometer set-up (borehole D1) consists of a cased borehole of 30 m long. The casing consists of 3 m long PVC-segments (OD 114.3 mm, ID 101.6 mm), to be screwed together.

Inside this casing, a fixed chain is installed. The chain is made up of nine 3-m long beams and two end beams of 1.5 m each. Between each pair of beams, a biaxial sensor (potentiometer) measures the angle between the beams in the vertical and in the horizontal direction. Each sensor has a range of $\pm 3^\circ$.

5.2.1.4.2 Installation

After drilling the borehole at a diameter of 150 mm up to 30 m, the PVC casing was installed, all on 7 May 1998. Due to the small annular space between casing and borehole, no filling was performed.

The deflectometer chain was installed on 29 May 1998.

5.2.1.4.3 Measurement performance

At the beginning of July 1998, we noticed that the deflectometer chain had been pushed out of the casing. When retrieving the chain, we noticed some clay on the deep end, indicating that the end cap of the casing had been broken. A plug was developed, tested and installed. The remaining part of the deflectometer chain was re-installed, with shorter arms at the end to accommodate for the shorter length (some 25 m), and stable measurements were obtained.

During the excavation, meaningful measurement data were obtained, but the interpretation has proven to be cumbersome due to the lack of a clear reference point (both borehole ends have moved during the excavation works).

5.2.2 Pressure measurements

5.2.2.1 Description

Four boreholes have been realised from the Test Drift to install piezometers to measure the pore water pressure; the piezometers were complemented with total pressure transducers, so that effective stresses could be derived.

The pore water is collected through stainless steel filter cartridges integrated into the borehole casing (Figure 5.9). The characteristics of these filters are given in Table 5.1.

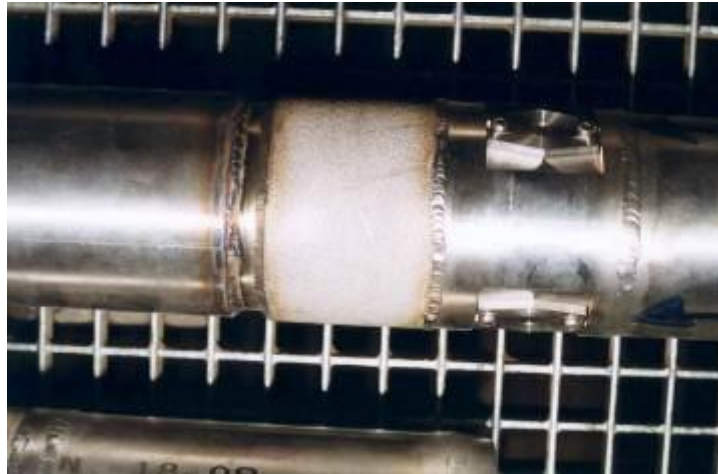


Figure 5.9: Filter and total pressure sensor integrated in the borehole casing

Table 5.1 : Characteristics of the stainless steel filters

Manufacturer	Sinter Metals Krebsöge Filters
Material and quality	AISI 316 L , SIKA-R 5/S
Filtergrade	5 μ absolute
Dimensions	ID 65 mm, OD 71 mm, filter length 50 mm

Through a twin-tube system (4/2 mm stainless steel tubes), the water is brought to the pressure transmitters at the Test Drift Front, where one of each tube pair is connected to a pressure transmitter DRUCK PTX1400. The maximum pressure is 25 bar (absolute range). Boreholes A2, B2 and D2 contain six filters at depths between 20 and 30 m, borehole C2 contains eight filters at depths between 16 and 30 m.

This system also allows to take samples of the water and to measure other hydraulic parameters (permeability etc.).

Each borehole for pressure measurements further contains miniature pressure sensors, whose diaphragm is mounted almost flush with the outer surface of the casing in the configuration as shown in Figure 5.9.

The pressure sensor has been supplied by Kulite (model HKM-155C-375M-55BARSG), with a typical sensitivity of 3 mV/V/MPa. Borehole A2 contains 3 sections with 3 pressure sensors

each, borehole B2 contains 2 sections with 3 pressure sensors each, while C2 and D2 each contain 2 sections with 2 pressure sensors in each section.

5.2.2.2 Installation

To combine the constraints of a small oversize for the borehole (to limit the perturbation of the stress state in order to obtain more representative pressure measurements) with the necessity of using large drill rods for a better control of the boring process, the following procedure has been applied for the four boreholes A2, B2, C2 and D2:

- drilling a 15 m deep borehole at a diameter of 150 mm;
- preliminary casing of the borehole with a steel tube;
- further drilling up to 30 m at 80 mm diameter, to limit the borehole convergence;
- optical survey of the borehole;
- installation of the instrumented casing (15 m) with extension casing (15 m); the extension casing is located within the preliminary casing of the first 15 m.
- grouting of the annular space between both casings when the pressure on all total pressure cells has built up (this indicates that contact between the instrumented casing and clay host rock had been established). At the borehole mouth, a hermetic welding between both casings prevents drainage through the grouting material.

The installation dates of the different boreholes were:

A2: 4 – 5 May 1998

B2: 12 – 13 May 1998

C2: 18 – 19 May 1998

D2: 5 – 6 May 1998

5.2.2.3 Data acquisition

All piezometers and total pressure cells were connected to Yokogawa DA100 modules. These were read out by the PC through a dedicated data acquisition programme. The measured values were then sent to the central database.

5.2.2.4 Measurement performance

After water is leaving all filters and the total pressure starts to increase, each filter is flushed through the twin-tube system. All piezometers then showed initially an increase in pressure, the highest pressure being reached at WD1, the deepest filter in borehole D2. Then, several piezometers, especially B2, have shown a pressure drop, apparently due to the construction activities of the second shaft.

The total pressure cells hardly measure any pressure above the pore water pressure, and therefore no effective pressure can be derived. To investigate this behaviour, several tests have been carried out. They consisted in draining the pore water pressure and monitoring the total pressure. The result showed however, that the pressure drop on the total pressure sensor, located a few cm from the filter, is smaller than the pore water pressure. This is probably due to the draining effect, creating a pressure gradient even over this short distance. The measurement performance of this type of sensors has however been demonstrated in other tests. The

RESEAL test showed that sensors in contact with buffer material clearly showed the swelling pressure, and, similar to our experience, the sensors in contact with the host rock did not register any significant pressure above the water pressure. One conclusion is that the installation procedure is an important factor when attempting to measure the total stress.

Several tests were also carried out to determine the permeability; no significant influence of the nearby Test Drift has been noticed.

During excavation, the water pressure evolution could be followed very well, including the excavation steps. The pressure increase due to the approaching front was somewhat higher than anticipated. For A2, two piezometers were replaced to cover a larger measuring range. The pressure peak at filter WD6 was partially above the measuring range of the sensor. The piezometers at A2 and B2 further show a sharp decrease, followed by suction and finally atmospheric pressure, which is reached somewhat earlier due to the fracturing of the clay host rock.

Both C2 and D2 have been reconnected after the tunnelling machine had cut the casing tubes. The D2 filters have reached high values again (except for one filter), while the C2 filters remain at a lower value.

5.3 Instrumentation installed from the second shaft

Parallel to the connecting gallery, two boreholes have been drilled: one for the installation of an inclinometer (E1), 5 m above the axis of the connecting gallery, and the other one, 6.5 m above the gallery axis, for piezometer E2 for pore water pressure measurements. The position of these boreholes is shown in Figure 5.10. In addition, also the position of other instruments is shown: the short, vertical piezometer F2 parallel with the shaft, and four sections with total pressure cells on the shaft lining (AA, BB, SS and NN).

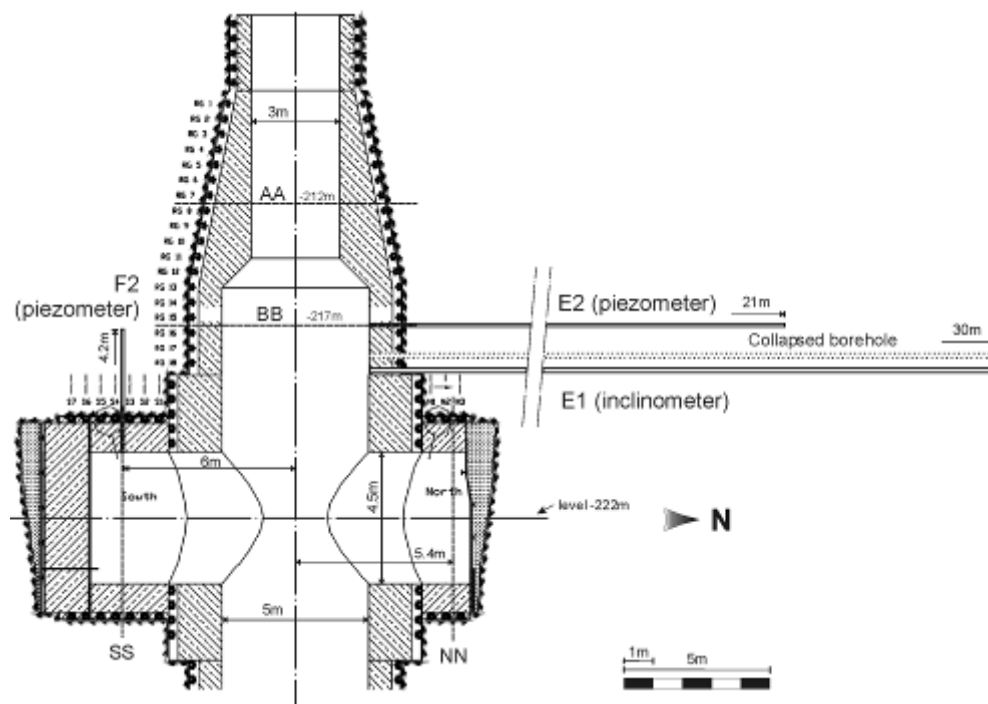


Figure 5.10: Instrumentation on and around the second shaft

5.3.1 Inclinator above the connecting gallery (E1)

5.3.1.1 Description

This device consists of a chain of 15 segments, each 2 m long, and installed inside a grooved casing. The inclinometer has been delivered by Interfels GmbH (D). The inclination of each segment, positioned inside the casing with a spring-loaded wheel mechanism is measured by means of an (uniaxial) electrolevel sensor, with integrated electronics to deliver a output signal between -500 and $+500$ mV, corresponding with an inclination between -3° and $+3^\circ$ with respect to the horizontal. The integration of all inclinations allows the calculation of the vertical displacements along the borehole.

5.3.1.2 Installation

At a radial distance of 5.5 m from the axis of the (future) connection gallery, a 150 mm diameter borehole has been drilled up to a distance of 30 m from the inner shaft lining; for this purpose, an access hole had been provided when casting the concrete shaft lining (Figure 5.11). After performing the initial optical survey, complemented with the borehole camera inspection, the borehole was cased with ABS grooved lining. The drilling started on 30 Oct 2000, and finished with the casing on the next day (31 Oct 2000).



Figure 5.11: Original position of inclinometer E1, one meter under piezometer E2; optical and borehole camera survey (right).

Due to the other drilling activities, the inclinometer chain could not be installed immediately.

When performing the second optical survey before installing the chain, it became clear that the casing had been deformed and collapsed. The inclinometer chain could therefore not be installed.

We decided to drill another borehole at 0.5 m under the first borehole. This required drilling through the shaft lining and reinforcement steel. Upon completion of the borehole on 1 March 2001, a steel liner (diameter, thickness) has been installed as a primary protection of the inclinometer casing. When probing the casing with an inclinometer casing section (7 June 2001), the tolerances on the steel tube proved to be too narrow, and the tube did not fit in. The small diameter inclinometer casing was finally installed after the inclinometer chain was adapted to fit into this casing on 12 June 2001.

5.3.1.3 Data acquisition

The sensors (IE1 at the deep borehole end to IE15 at the borehole mouth) are connected to a CR10X datalogger from Campbell through an AM416 multiplexer. Readings are taken every 2 hours and stored on the logger. The measurement data are downloaded on a regular basis for analysis in an Excel file.

5.3.1.4 Measurement performance

The inclinometer has provided us with quite stable readings. Both during the excavation of the mounting chamber and the start of the actual excavation, it provided us with very meaningful data.

5.3.2 Multi-piezometer above the connecting gallery (E2)

5.3.2.1 Description

This multi-piezometer has a total length of 21 m and contains 7 filters, spaced 3 m apart from each other. Through a twin tube system (4/2 mm stainless steel tubes), each filter is connected to a pressure transmitter located in the shaft. The pressure transmitters are supplied by Druck (model PTX1400) and have a range of 25 bar (relative).

5.3.2.2 Installation

At 6.5 m above the future gallery axis, a 103 mm diameter borehole has been drilled up to a distance of 21 m from the inner shaft lining; for this purpose, an access hole had been provided when casting the concrete shaft lining. The borehole was cased with the stainless steel instrumented casing (73 mm OD), which was welded together during the installation. Drilling and installation were performed on 26 October 2000.

After we noticed water flowing out from the deep filters, we connected the pressure transmitters to the filters, after saturating the twin tube system.

5.3.2.3 Data acquisition

The pressure transmitters (WE1 to WE7) are connected to a HP34970A datalogger, where the 4-20 mA signal is read through a 120 Ω shunt on the datalogger. Readings are taken every hour and stored on the logger. The measurement data are downloaded on a regular basis for analysis.

5.3.2.4 Measurement performance

The piezometer showed, after the initial pressure increase after the installation, a clear response (pressure decrease) when redrilling the E1 borehole. Also the excavation of the

mounting chamber, and the excavation of the connecting gallery were very clear registered in the pore water pressure readings.

5.3.3 Short vertical piezometer (F2)

This multi-piezometer has a total length of 4.2 m and contains 7 filters. Through a twin tube system (4/2 mm stainless steel tubes), each filter is connected to a pressure transmitter located in the shaft. The pressure transmitters are manufactured by Druck (model PTX1400) and have a range of 25 bar (absolute).

5.3.3.1 Installation

From the south starting chamber, a vertical, upward 83 mm diameter borehole was drilled, 4.2 m deep, at a radial distance of 6m from the axis of the second shaft. The borehole was cased with the stainless steel instrumented casing (73 mm OD), which was welded together during the installation. Drilling and installation were performed in one day (15 Nov 2000).

At the end of January 2001, we connected the pressure transmitters to the filters, after saturating the twin tube system.

5.3.3.2 Data acquisition

The pressure transmitters (WF1 to WF7) are connected to a HP 34970A datalogger. Readings are taken every hour and stored on the logger. The measurement data are, together with the data from the E2 piezometer, downloaded on a regular basis for analysis.

5.3.3.3 Measurement performance

Ups to now, the pressure sensors have observed no pressure, apart from the hydrostatic column between the filter and the sensor. This is most probably due to the fact that the zone surrounding the second shaft is still affected by the excavation and related disturbance, and that the pressure build-up therefore will be a very slow process.

5.3.4 Pressure measurements on the lining of the shaft and starting chambers

5.3.4.1 Description

Two sections on the shaft lining, and one section on each starting chamber, have been instrumented with total pressure cells. In addition, each section also contains a piezometer. The position of each section is shown on Figure 5.12.

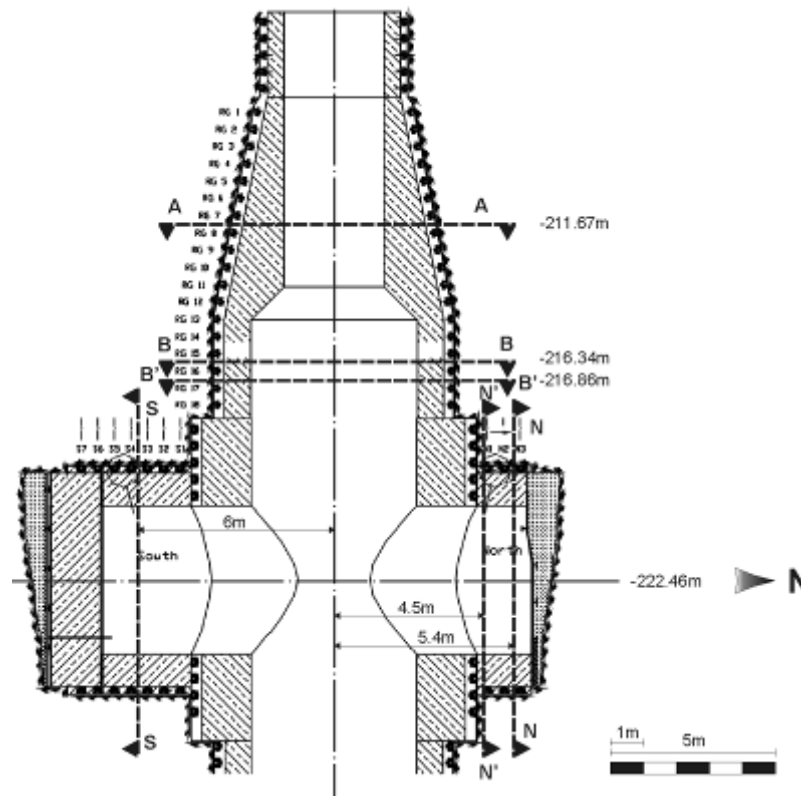


Figure 5.12 : Position of the different sections that are equipped with total pressure cells

The total pressure cells are manufactured by Sinco (Boart-Longyear Group), and consist of a flat jack, connected to a vibrating wire transducer with a range of 3.45 MPa (500 psi). Each section is also instrumented with one piezometer, consisting of a filter (5 μ pore), attached to a filter room, to which a twin tube (2 x 4 mm OD tubing) is attached for connection with the pressure transmitters.

5.3.4.2 Installation

The sensors are positioned between the circular steel ribs, in direct contact with the excavated clay (after local removal of the steel net). After a preliminary mounting of the sensors (Figure 5.13), the concrete lining was cast in place.



Figure 5.13: Preparation of the clay interface (left) ; installed sensor (sensor B3), before casting concrete (right)

5.3.4.3 Data acquisition

The sensors are read out manually on a regular basis.

5.3.4.4 Measurement performance

After installation, the total pressure increased very slowly. The excavation of the mounting chamber, as well as the excavation of the gallery can be clearly observed through the pressure increases.

The piezometers on the shaft lining show no significant pressure increase, as the zone around the shaft lining is still affected by the excavation of the shaft (cf. also piezometer F2).

5.4 *Instrumentation installed in the gallery lining*

5.4.1 Design

To monitor the stress build up in the gallery segment lining as a result of the installation procedure (wedge block) and the pressure exerted by the clay host rock, the segments of three rings have been instrumented with embedded vibrating wire strain gauges.

Based on the design of the individual segments and of the segment ring, two measurement configurations were put forward: one with six (type A) and one with twelve gauges (type B). Each configuration contains both intrados (10 cm from the inner wall) and extrados (10 cm from the outside) gauges, distributed along three levels, and always in the circumferential directions. Each segment ring consists of 4 segments with 12 gauges, and 6 segments with 6 gauges. The key segments have not been instrumented.

After contacting several potential suppliers, we decided to cooperate with Gage Technique International Ltd. (UK), as this company was the only one to give practical support in the installation (embedment) of the gauges in the segments. Also their reference list convinced us further of their competence in this field. The strain gauge (model TES/5.5/T) is based on the vibrating wire principle, and is further equipped with a thermistor. To embed the gauges in the concrete segments, we decided to mount the gauges on a light steel (6 mm rebar steel) cage, serving as a support. This would be installed in the mould prior to concreting. The cage also allows a better protection of the cabling whilst pouring the concrete. The final design and manufacturing of these cages was done by the segment production company, Buchan (UK).

5.4.2 Production of the instrumented segments

The segments were produced in October and November 2001.

A solution for the feedthrough of the gauge cables that was compatible with the manufacturing of the segments was proposed by Gage Technique. It consisted of a epoxy resin shell (fibre reinforced), connected with the cage through steel bars with some spring action so that this shell was pushed against the inside wall of the mould. The position of each cable feedthrough identifies the individual gauge location. The position of the shell itself took into account the location of the gauges, the manipulation holes, and the concrete pouring. The 12 gauge segments have the shell above (relative to the mould position when casting concrete) in the middle of the inside wall, whilst the 6 gauge segments have it at mid-height at the right side. Figure 5.14 shows some details of an instrumented 6 gauge segment cage.

The instrumented cage was positioned in the mould by plastic distance holders (disc-shaped, 100 mm radius). The feedthrough shell was protected by a plastic sheet to prevent the concrete from entering this area. Casting of concrete was performed in several phases, alternated with vibrating the concrete. Figure 5.15 shows the instrumented cage just before and during casting. Later on, guiders were added to the cages to guide the vibrators and to prevent them from touching the gauges.

Upon unmoulding the next day, the feedthrough shell is embedded in the concrete segment as planned as shown in Figure 5.16. No sensor failed during production of the segments.

Transport of the segments to Mol took place in December 2001.



Figure 5.14 : Instrumented cage (left); detail of feedthrough shell (right)



Figure 5.15 : Cage (12 gauge segment) just before and during concrete casting



Figure 5.16 : View on the first two segments after unmoulding; detail (right) of the feedthrough shell after the protective sheet has been removed

5.4.3 Installation

Taking into account the construction works and the expected location of the future PRACLAY- gallery, we decided to put the instrumented segments at ring locations 15, 30 and 50. Two spare segments were placed in ring 75. The configuration of each ring is given in appendix 3.

The installation dates were:

ring 15: 2002-02-12

ring 30: 2002-02-18

ring 50: 2002-02-25

ring 75: 2002-03-05

Prior to installation, a zero reading was performed on all segments. After installation, readings were taken when the field works made this feasible.

To ease the manual read-out, a connector box with a multipole connector was installed on each segment after the segments had been installed.

As the read-out frequency can be reduced to once a month, and to save on extra cabling and programming work, as well as on extra investments, we decided not to install an automatic data-acquisition system.

5.5 Conclusion

Overall, the instrumentation programme has been implemented successfully. The initial modelling exercise allowed us to design an efficient sensor configuration. Most instruments have been installed with success; the problems with the plastic casing could be resolved by in situ reparation (D1) or by redrilling (E1); the only sensors lost deal with C1 and with some total pressure sensors on the piezometers. The care taken when embedding the sensitive strain gauges in the concrete segments also resulted in a complete success, with no sensor lost during casting of the concrete, despite the harsh manufacturing conditions. The same applies to the instrumentation around the second shaft: all instruments are still functioning flawlessly.

A good interaction between the modelling and instrumentation is a critical success factor in both the successful application of the instrumentation, and in the full valorisation of its measurements results.

6 The blind predictions (see appendix 4 : definition for blind prediction)

The blind prediction exercise was based on the real excavation parameters (excavation rate and sequence, excavated and lining radius) and on different rheological models. Each modelling team has made his own choice to represent the hydromechanical behavior of Boom Clay, based on his experience and on the results of in-situ tests. Each participant had to provide the evolutions of the hydromechanical responses (total stresses σ , displacements u and pore water pressure u_w fields to be recorded by the CLIPEX instrumentation installed in the front of Test Drift) as a function of the excavation front position.

A detail model description (mesh, constitutive laws, initial and boundary conditions and the numerical excavation procedures, etc.) adopted by each will be presented at first. Then the blind prediction results (displacements, total pressure, pore water pressure) will be detailed, compared and analysed.

Detail data considered for the blind prediction can be found in appendix 4.

6.1 Models description

As each modelling team was free to treat the problem with his own numerical tool, the strategy adopted by each for the blind prediction differs one from another. Resumptive description of them facilitates therefore the comparison of the prediction results.

6.1.1 SCK•CEN

6.1.1.1 Hypotheses

The following main hypotheses have been adopted:

- Axisymmetric conditions.
- Plane strain conditions.
- Isotropic stress state.
- Homogeneous initial stress and pore pressure state: variation with depth was neglected in the modelling zone.
- Non influence of the 2nd shaft.
- Perfect contact between the clay and the lining: the contact behaviour between them was not modelled.
- Excavation was supposed under undrained condition.

6.1.1.2 Geometry

The modelling zone is concentrated on the region around the connecting gallery and the head of Test Drift. The dimension of the grid is chosen as large enough compared to the gallery radius R_0 (≈ 2.45 m) as indicated in Figure 6.1a, where the reference axes are also represented.

The grid is composed of 42 zones height (x direction), graded by a factor of 1.15, and 153 zones in the axial direction of gallery (y direction) (Figure 6.1b). The density of mesh is increased when approaching the CLIPEX instrumentation.

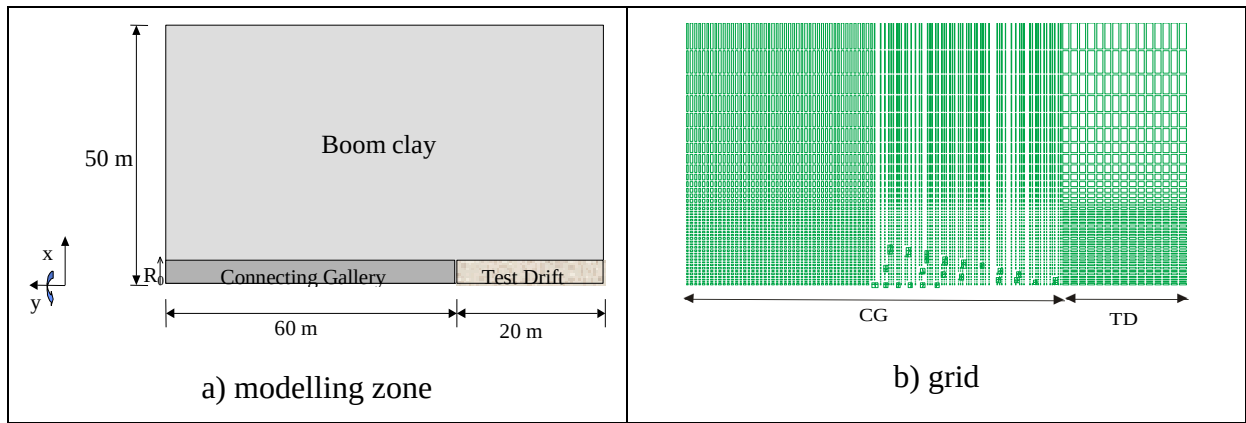


Figure 6.1 : Geometry of the modelling zone and the grid used

6.1.1.3 Modelling procedure

The modelling was performed in the framework of the soil mechanics for saturated porous media taking into account the hydro-mechanical coupling.

According to the construction sequence of URL (Underground Research Laboratory) at SCK•CEN, the modelling procedure was divided into three major stages:

Stage 1: Test Drift excavation (only last 20 m is considered)

Since most of target sensors were instrumented in the front of the Test Drift (see Figure 5.4), it was felt necessary to take into account the complete unloading history of clay massif around these instruments, consequently, the Test Drift excavation has been modelled.

The modelling of Test Drift excavation was accomplished by a simplified way commonly used with FLAC: i.e. by nulling zones to be excavated in one step and relaxing the tunnel boundary pressure linearly to zero in order to get more static solution. The Test Drift lining procedure was not modelled and simply replaced by applying a limit pressure on the tunnel boundary.

The problem was considered as undrained due to the weak hydraulic conductivity of Boom clay.

Stage 2 : 14 year's drainage around the Test Drift (from 1987 to 2001)

The long-term hydro-mechanical behavior during 1987 to 2001 (during 14 years, after Test Drift construction and before connecting gallery excavation) was modelled by considering the drainage of the Test Drift.

Stage 3 : Connecting gallery excavation

The connecting gallery was excavated under the protection of a shield. The construction of the connecting gallery was modelled following its real constructing sequence: excavation and then installation of the lining. These two steps progressed in turn successively with the advancing front. The excavation was modelled by nulling an entire cross section to be excavated in a single step in accompanying a linear relaxation of the boundary pressure. The length of the rounds excavated varies so following the irregular mesh size (from 0.33 m to 0.652 m). The elements representing the lining were activated about 3.58 m (length of the

shield + uncovered span) behind the advancing working face. For the same reason, the width of lining varies from 0.33 m to 0.652 m. The uncovered span between the rear of shield and lining segment is 1.2m in maximum (Figure 6.2).

The excavation was supposed under the undrained condition taking into account the quickness of excavation (more than 2 m/day). The shield protection was numerically modelled by blocking the convergence. The lining was represented using classical elements with elastic model.

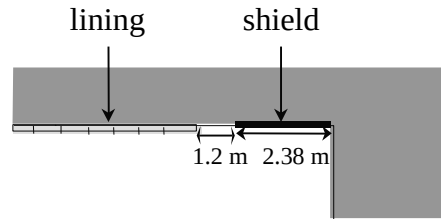


Figure 6.2 : Modelling sequence of connecting gallery: excavation – lining installation

6.1.1.4 Initial and boundary conditions

The clay formation around the gallery is considered as homogeneous and isotropic. The initial stresses are supposed to be hydrostatic ($\sigma_0 = \sigma_h = \sigma_v = \rho \cdot g \cdot z$, ρ is the material specific mass, g the gravity acceleration and z the depth). In addition, the modelling zone is supposed to be sufficiently deep so that the variation of the stress and pore pressure with depth is neglected. Consequently, the initial stresses and pore pressure inputted in the model correspond to the values at depth of gallery axis (Table 6.1).

Table 6.1 : Initial stress and pore fluid pressure in Boom clay around gallery

Total stress	$\sigma_0 = \sigma_h = \sigma_v$	4.5 MPa
Pore pressure	u_w	2.25 MPa
Effective stress	σ'_0	2.25 MPa

The boundary conditions are sketched in Figure 6.3 where a normal pressure corresponding to the overburden of the interested zone is applied on the upper boundary.

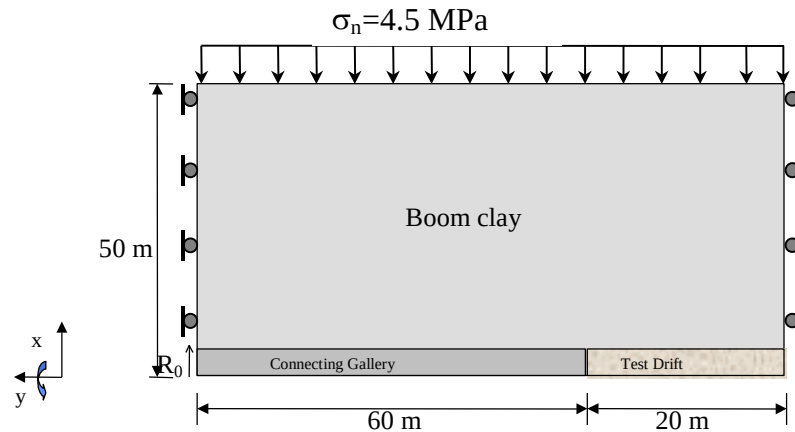


Figure 6.3 : Boundary conditions

The hydraulic boundary conditions were imposed in such way:

Stage 1: excavation of Test Drift: impermeable boundary, non water flow was allowed.

Stage 2: drainage of Test Drift: The Test Drift wall is considered permeable, the pore water pressure along the Test Drift wall was imposed to be zero.

Stage 3: excavation of connecting gallery: all boundaries were impermeable.

6.1.2 GEOCONTROL

Due to the large size of the problem, GEOCONTROL has modelled two zones separately as shown in Figure 6.4.

Zone 1 corresponds to the joint between the Test Drift and the connecting gallery,

Zone 2 represents the second shaft and the connecting gallery.

The zone 1 is equivalent to the zone modelled by the other teams. Consequently, it's the model for zone 1 that is resumed here for the sake of comparison. Part of the results of the model for zone 2 will be referred when comparing the measurements from 2nd shaft.

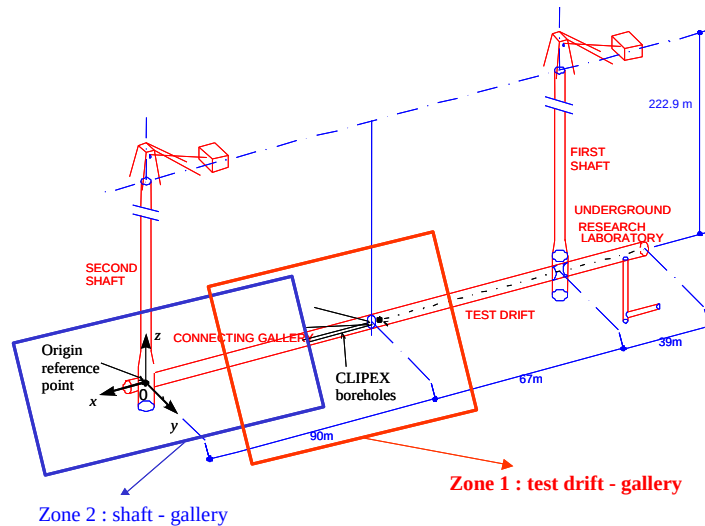


Figure 6.4 : Location of the models developed by GEOCONTROL

6.1.2.1 Hypotheses

The following main hypotheses of the model for zone 1 have been adopted:

- Three Dimensional condition, symmetry in a vertical plane along the gallery axis has been considered.
- Isotropic stress state.
- Initial in-situ stress and pore pressure depending on the depth.
- Non influence of the 2nd shaft.
- Imperfect contact between the clay and the lining: the contact between them is modelled using the interface elements.
- Excavation was supposed under undrained condition.

6.1.2.2 Geometry

The Cartesian frame (0, x, y, z) used in the model is represented in Figure 6.4, i.e. the z-axis is vertically upward oriented and the x-axis is oriented following the axis of the connecting gallery. The gallery axis is located at a depth of 222.9 m from the ground surface. The origin of the model is located in the shaft axis at the same depth of the gallery axis. Symmetry in a vertical plane along the gallery axis has been considered.

The model is 90 m long in the x direction (30 m corresponds to the Test Drift and 60 m to the connecting gallery), 43 m in the y direction and 95.8 m high, with 104448 elements and 112041 grid points. Figure 6.5 shows a general view of the model developed.

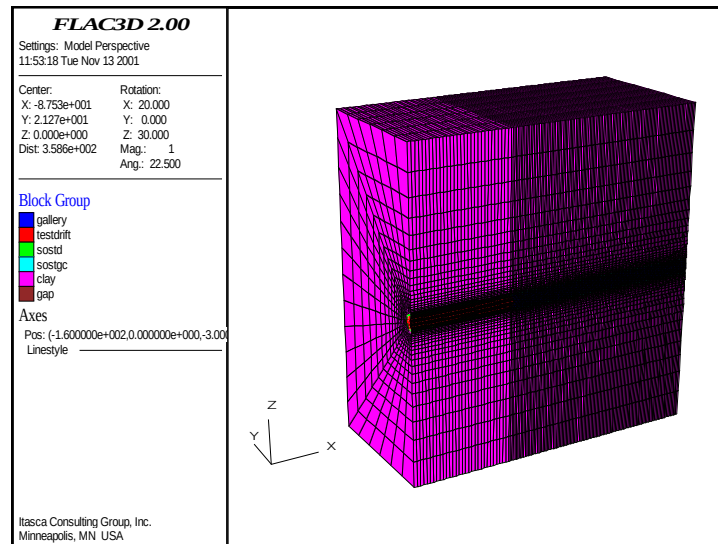


Figure 6.5 : General view of the model

The excavation radius of the Test Drift is 2.55 m and the lining external radius is 2.40 m, so there is a gap of 15 cm between the ground and the Test Drift lining. The thickness of the lining is 0.60 m. Figure 6.6 shows a view of the Test Drift.

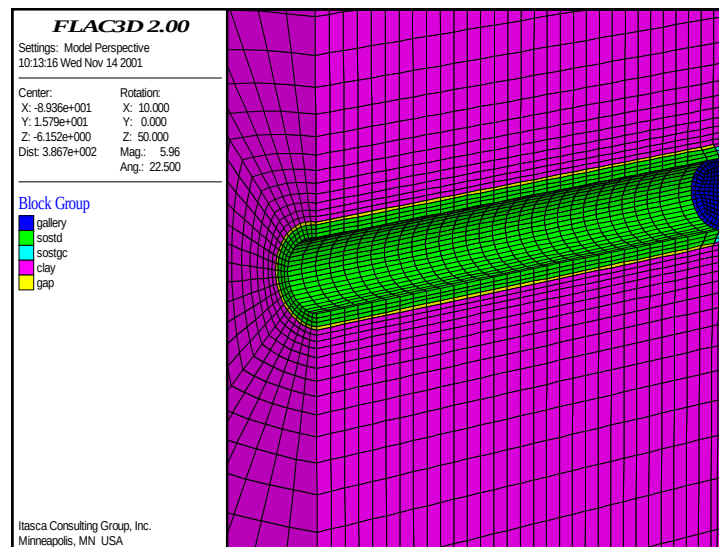


Figure 6.6 : Geometry of the Test Drift.

The connecting gallery has been modelled using an excavation radius of 2.405 m being the thickness of the lining rings of 0.40 m. The geometry of the connecting gallery is shown in Figure 6.7.

Figure 6.8 shows a view of the joint between the Test Drift and the connecting gallery.

The shield used for the excavation of the connecting gallery has been also modelled with elements. The external radius of the shield is 2.405 m and the contact between the soil and the shield has been modelled using an interface. Figure 6.9 shows the simulation of the excavation process of the connecting gallery with the shield.

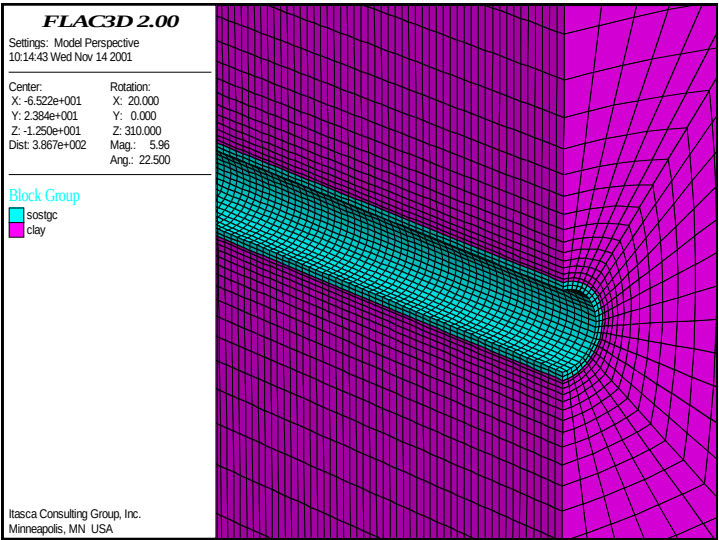


Figure 6.7 : Geometry of the connecting gallery.

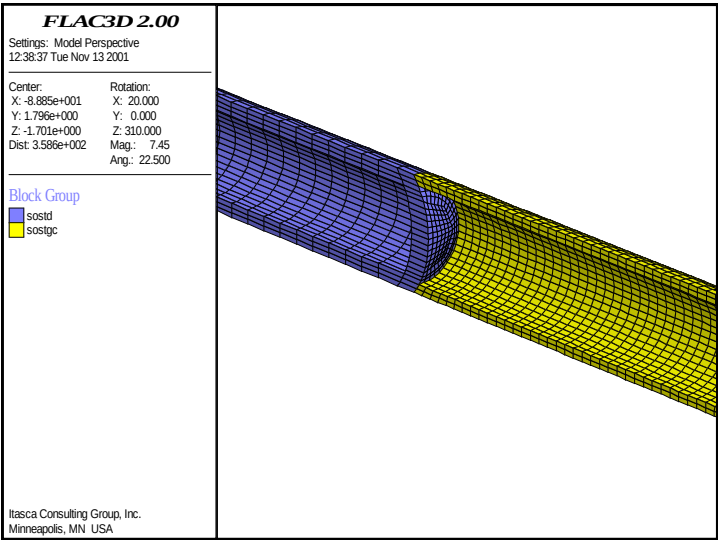


Figure 6.8 : Geometry of the joint (Test Drift-Connecting gallery)

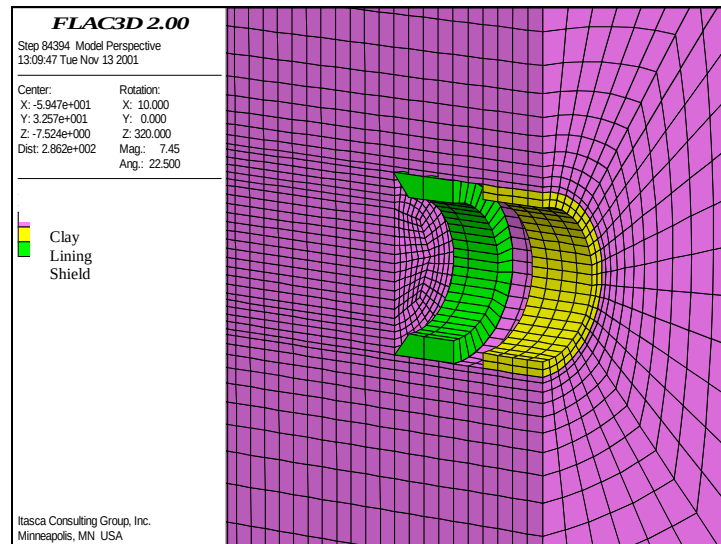


Figure 6.9 : Geometry of the shield.

6.1.2.3 Modelling procedure

The excavation of both test drift and connecting gallery has been modelled following as realistically as possible the excavation procedure.

The Test Drift has been excavated manually, so the excavation sequence followed in the modelization was:

- Excavation of a drive length of 1 m (which means that one element is deleted).
- Activation of the lining segment (1 m long and 0.60 m thick) 1 m behind the front, which means an unsupported span of 1 m. Due to the gap existing between the soil and the lining, the grid deforms (large strain mode) and comes into contact with the lining some meters behind the front. This contact between the clay and the lining is modelled using an interface. Figure 6.10 shows the excavation sequence of the Test Drift.

As for the connecting gallery, the excavation sequence followed was:

- Excavation of a drive length of 1 m (which means that two elements are deleted) under the protection of the shield, which is 2 m long.
- Activation of the lining segment (1 m long and 0.40 m thick) 3 m behind the front, which means an unsupported span of 1 m. The lining segments are placed directly in contact with the soil. The contact between the shield and the ground is modelled using an interface. The excavation sequence developed is shown in Figure 6.11.

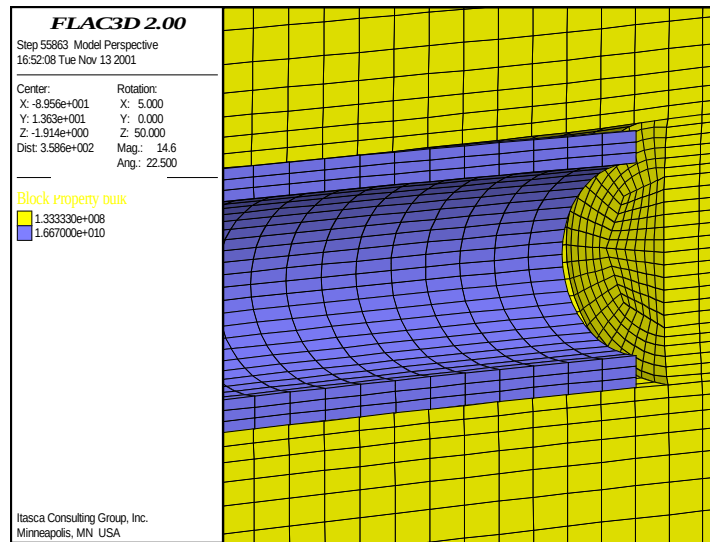


Figure 6.10 : Excavation sequence of the Test Drift.

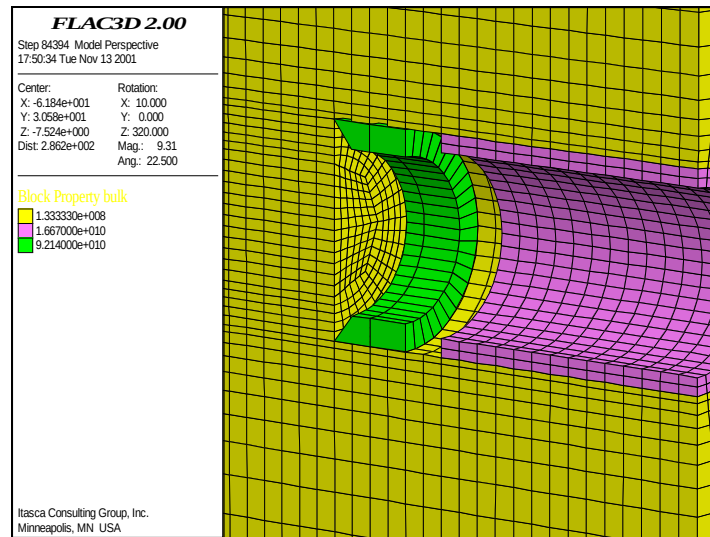


Figure 6.11 : Excavation sequence of the connecting gallery.

6.1.2.4 Initial and boundary conditions

As boundary conditions, displacements in the normal direction of the vertical planes have been fixed. In the bottom boundary of the model displacements in the z direction have been fixed. Figure 6.12 shows a sketch of the model with its fixed conditions.

The model have been initialised to the in situ stress field before excavations according to the following:

$$\begin{aligned}\sigma_{zz} &= \rho g z, & \text{vertical stress } (\cong 4.5 \text{ MPa at the gallery axis depth}) \\ \sigma_{xx} &= 1.0 \sigma_{zz}, & \text{horizontal stress in the direction parallel to the tunnel axis} \\ \sigma_{yy} &= 1.0 \sigma_{zz}, & \text{horizontal stress in the direction normal to the tunnel axis}\end{aligned}$$

The pore water pressure has been initialised according to $u_w = \gamma_w z$.

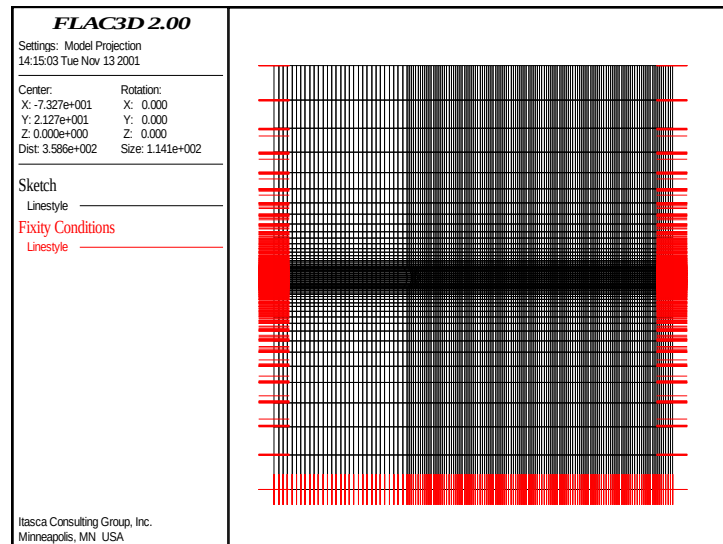


Figure 6.12 : Fixed conditions

In the upper boundary of the model a vertical stress corresponding to the weight of the upper ground has been applied (Figure 6.13).

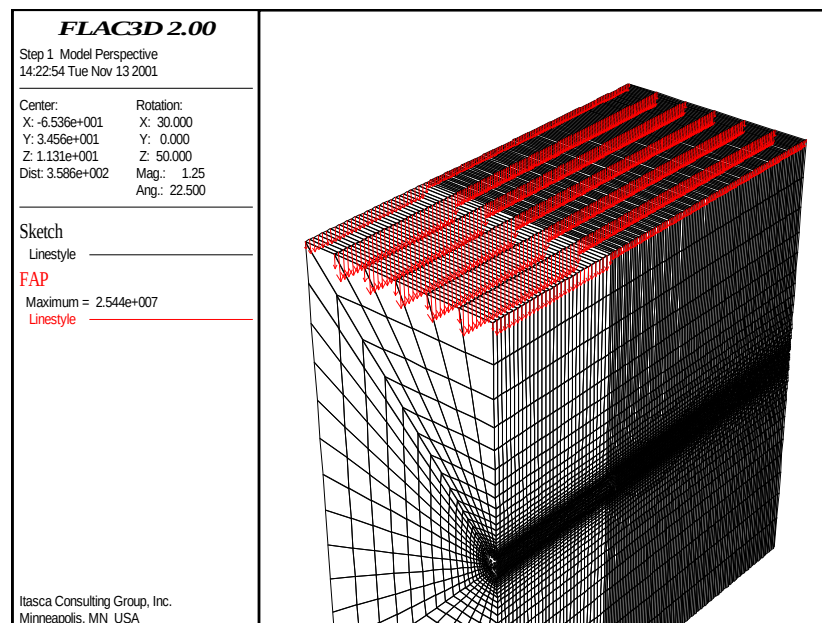


Figure 6.13 : Boundary conditions

6.1.3 UPM

6.1.3.1 Hypotheses

Same hypotheses as those of SCK•CEN have been adopted (see § 6.1.1.1).

6.1.3.2 Geometry

The total length of the mesh is 120 m, of which 90 m corresponds to the connecting gallery and 30 m to the test drift (Figure 6.14). The width of the mesh is of 48 m, approximately 12 times the diameter of the tunnel.

A grid with 33600 nodes and 4200 elements with variable step between nodes in different zones of the mesh has been used. The mesh in the zones where it is anticipated that intense gradients of stress and deformation exist is then refined with the purpose of obtaining better results in the modelling. Consequently, the grid comprises:

For the clay, there are 3600 elements, 28800 nodes in the connecting gallery zone, 400 elements and 3200 nodes in the test-drift zone. 200 elements and 1600 nodes are attributed to the steel (lining).

The element for the clay is the type CAX8RP, which is a hydromechanical coupled axisymmetric continuous finite element being of 8 nodes with reduced integration (5 points of integration) and, three degrees of freedom that are the radial displacements, axial and the pore pressure. In the simulation of the steel of the support, mechanical elements of the type CAX8R have been used, similar to the previous ones but that they do not allow the calculation of the pore pressure.

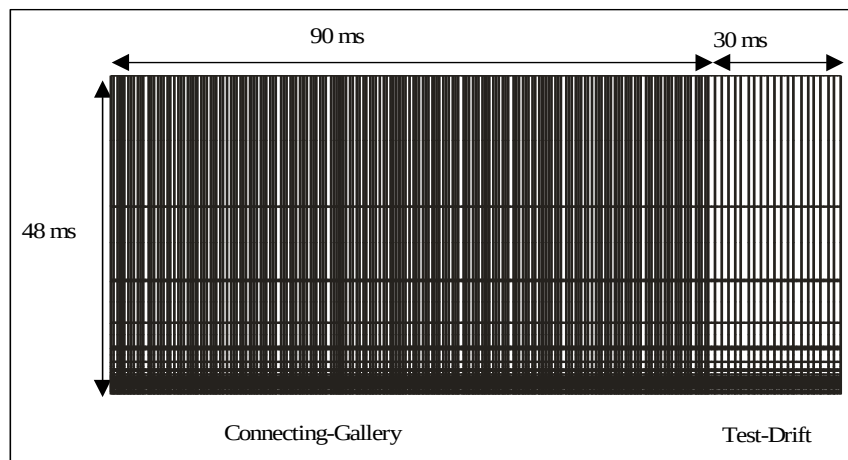


Figure 6.14 : Geometry of the model.

6.1.3.3 Modelling procedure

As we can see in Figure 6.15, the simulation process was started with excavating the last thirty meters of the test drift, trying to simulate in certain way the changes occurred in the soil by the previous excavation. The simulation of the test drift excavation was made in a simplified way using a coarse grid. The following excavation sequence of the total gallery was considered:

1. Each complete section of the tunnel was excavated in only one step by means of semi-mechanic methods. That is to say, in the same step, two clay slices are removed and two liner slices are added.
2. The length of the excavation slices was the following: 1 m length in the test drift and 0.5 m length in the connecting gallery.
3. During each excavation step the distance between the front of advance and the support was 0.5 m.
4. The rate of advance of the connecting gallery excavation is 1m/day.

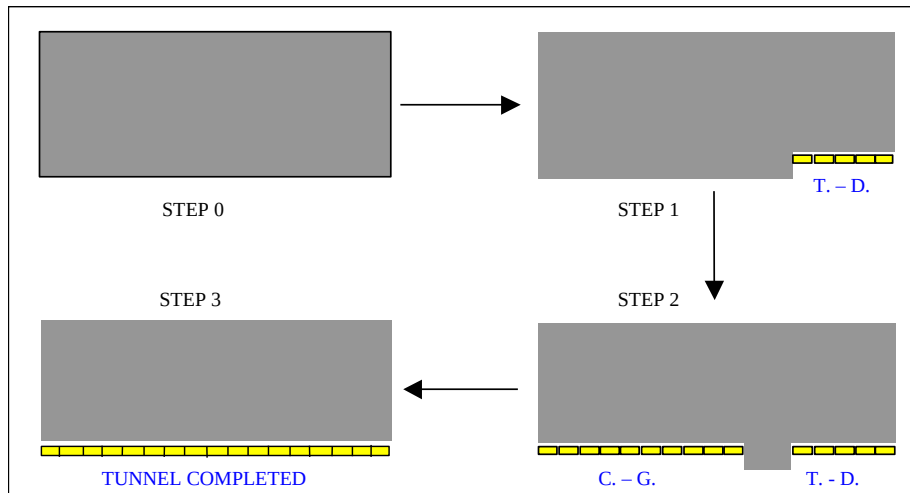


Figure 6.15 : Excavation procedure in three steps (T-D: Test Drift, C-G: connecting gallery)

6.1.3.4 Initial and boundary conditions

The same initial conditions as in the model of SCK•CEN have been chosen (see § 6.1.1.4). Concerning the boundary conditions, no-flow and null axial displacements in the lateral contour and null radial displacements in the gallery axis were imposed.

6.1.4 G3S

6.1.4.1 Hypotheses

Same hypotheses as made by SCK•CEN were adopted.

6.1.4.2 Geometry

The size of modelling zone was chosen based on several preliminary hydraulic steady state regime calculations in searching a relevant distance of the outer hydraulic Dirichlet boundary (constant pore pressure) to fit better the measurements in the piezometers installed at front of Test Drift. The geometry obtained thus extends 400 m in the direction of the gallery axis (which is the axisymmetric axis), and 100 m in the radial direction as shown in Figure 6.16a. The mesh is sketched in Figure 6.16b.

6.1.4.3 Modelling procedure

In order to get relevant stress and pore pressure fields already disturbed due to the existing galleries and shafts, it was decided to simulate the problem in transient regime accounting for:

1. the excavation of the TD (~1987),
2. the excavation of the starting chamber (~1998),
3. the excavation of the connecting gallery (~ end of 2001, as it was first planned).

The excavation was realised numerically by deactivating the stiffness of the elements representing the ground being excavated (Figure 6.16c, d).

The effect of the lining was simulated by stopping the convergence at a likely value of 5 cm (on the basis of the previous blind prediction exercise).

6.1.4.4 Initial and boundary conditions

The initial state putted in the model was the same as in the SCK•CEN's model (Figure 6.16a).

Concerning the boundary conditions, a null radial displacement and a null water flow were imposed on the axis. The other boundaries, far from the excavations, were assumed not to be affected by the excavation (Figure 6.16a).

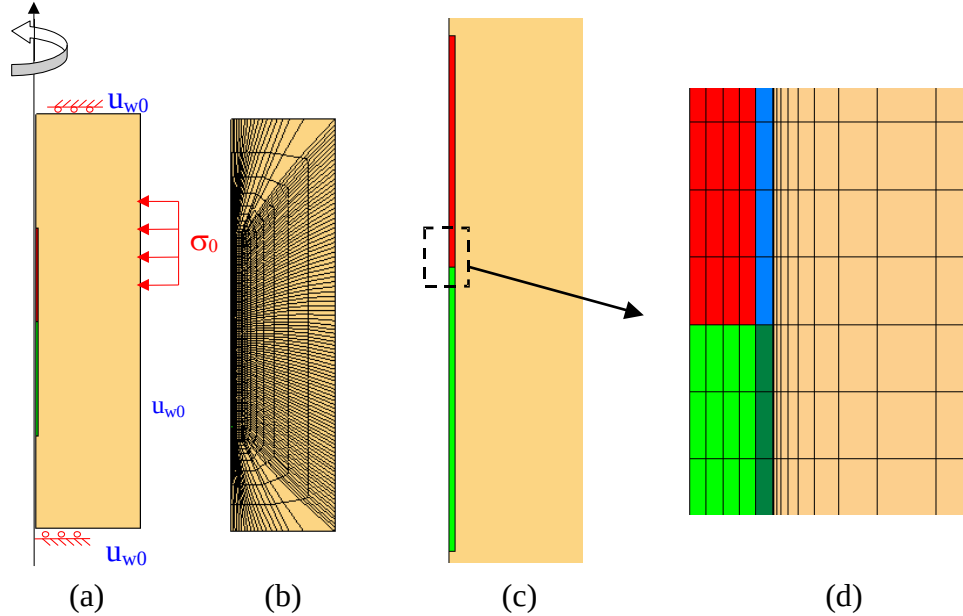


Figure 6.16 : Geometry, initial and boundary conditions – Zones to be excavated.

6.2 Mechanical constitutive laws used for Boom clay and corresponding parameters

6.2.1 SCK•CEN and GEOCONTROL

6.2.1.1 Constitutive laws : Modified Cam-Clay model and Mohr –Coulomb model in FLAC

The SCK•CEN has used respectively two classical elasto-plastic constitutive laws to represent the behaviour of the Boom clay under excavation: modified Cam-clay model and Mohr-Coulomb model. GEOCONTROL used then only the Mohr-Coulomb model in his simulation.

The Modified Cam-Clay model (M-C-C) is written in the frame of the critical state concept in which both the volumetric yielding and frictional shear yielding behaviour can be taken into account. The influence of the volume change of material on the bulk property can be considered via a non-linear elasticity formulation (the bulk modulus K depends on both effective pressure and the specific volume). The yielding surface in a p' - q stress space is shown in Figure 6.17a where p' represents the effective pressure, q the second invariant of the stress deviator. The slope of the Critical State Line (CSL) M is related to the friction angle ϕ .

The Mohr-Coulomb model (M-C) is a conventional model to present the shear failure in soils. The representation of failure criterion in a p' - q plane is sketched in Figure 6.17b, where m depends on the friction angle ϕ . The resistance on extension is taken into account through the cohesion c .

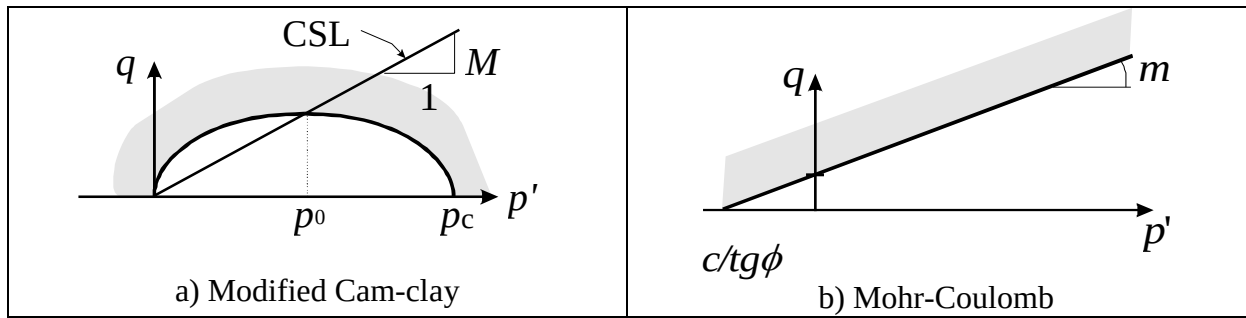


Figure 6.17 : The yield surfaces of two classical constitutive laws

6.2.1.2 Corresponding parameters used for the simulations

A considerable effort has been expended formerly to determine the parameter scope of the Boom clay for these two constitutive laws in different projects (Bernier et al. 1997, Bernichon, 1998, Barnichon et al. 2000, Horseman et al. 1987).

Parameters for modified Cam-clay model

The 'best' set of values usually considered at SCK•CEN to represent the Boom clay behavior around 225 m depth is listed in Table 6.2.

The two values 0.64 and 0.81 for the slope M of the CSL correspond respectively to the triaxial extension value and compression one. As the modified Cam-clay model in the FLAC code is implemented without taking into account the influence of the Lode angle (so not the intermediate stress σ_2 influence), two predictions results with $M = 0.64$ and 0.81 have been compared firstly by SCK•CEN in order to assess the influence of this parameter. Based on this comparison, the decision was made to make the prediction with their average value ($M = 0.72$).

Table 6.2 : Cam-clay parameters representative of Boom clay

Slope of the normal consolidation line (NCL)	λ	0.13
Slope of the elastic swelling line (SWL)	κ	0.02
Slope of the critical state line (CSL)	M	0.64-0.81
Preconsolidation pressure	p_c	6 MPa
Poisson's ratio	ν	0.2
Specific volume at 1 kPa on the NCL	N	2.75

Parameters for Mohr-Coulomb model

The parameters required for Mohr-Coulomb model are reported in Table 6.3. They are usually considered as the 'best' set of values at SCK•CEN to represent the Boom clay behavior around 225 m depth. This set of parameters was applied by GEOCONTROL too.

Table 6.3 : Representative Mohr-Coulomb parameters for Boom clay

Young's modulus	E'	300 MPa
Poisson's ratio	ν'	0.125
Friction angle	ϕ'	18°
Cohesion	c'	0.3 MPa
Dilation angle	ψ	0°

6.2.2 UPM

6.2.2.1 Constitutive law used : Modified Cam-clay model in ABAQUS

Theoretically, the modified Cam-clay model is implemented in ABAQUS taking into account two special aspects:

- In a plane p - q , the shape of the yield surface on the “wet” side may have a different curvature from the elliptic arc on the “dry” side via a constant β (see Figure 6.18).
- the intermediate stress effect can be taken into account via a coefficient K (Figure 6.19) so that the different ultimate failure state in compression and extension of soil can be reflected by the yield surface in a deviatoric plan.

It's however necessary to note that this version of modified cam-clay model falls into a classical modified cam-clay model like the version in FLAC when $K=1$ and $\beta=1$. It's the case adopted in UPM. So, UPM has used the same law as SCK•CEN in point of fact: classical modified Cam-clay model.

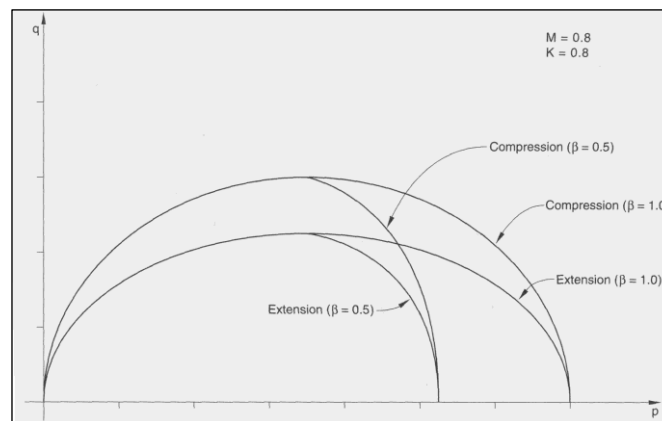


Figure 6.18 : Cam-clay yield surface in the p - q plane

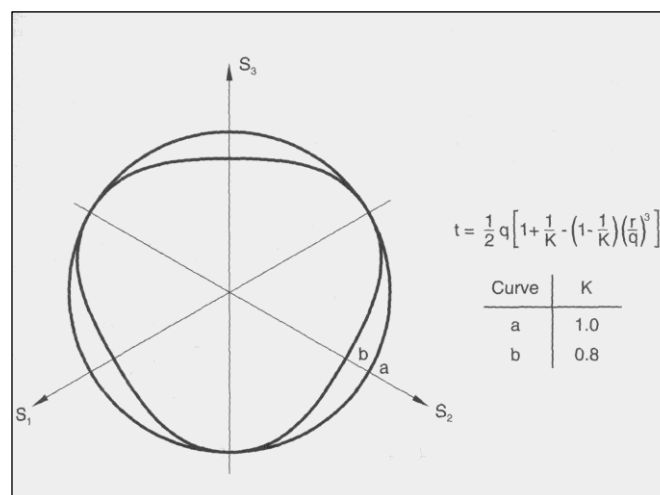


Figure 6.19 : Cam-clay surfaces in the deviatoric plane

6.2.2.2 Corresponding parameters used for the simulations

The set of parameters used by UPM is the same as listed in Table 6.2, but with a critical state line $M = 0.814$.

6.2.3 G3S

6.2.3.1 Constitutive law used : Cam-clay model in ANTHYC

The Cam-clay model was chosen also by G3S due to its simplicity and economic CPU. However the non-linear elasticity is not taken into account.

6.2.3.2 Corresponding parameters used for the simulations

The corresponding parameters used in the model are given in Table 6.4 below.

Table 6.4 : Cam clay model parameters used by G3S

○	Slope of the normal consolidation line $\lambda \in [0.05 ; 0.25]$
○	Slope of the elastic swelling line: LINEAR
▪	Young Modulus $E = 332 \text{ Mpa}$
▪	Poisson ratio $\nu = 0.2$
○	Preconsolidation pressure $P_c \in [4.5 \text{ MPa} ; 6 \text{ MPa}]$
○	Critical state friction angle $\phi_c = 12^\circ$

It's worth to draw attention on the value of the critical state friction angle: 12° instead of 21° . The objective of this important modification is to increase the EDZ (Excavation Disturbed Zone) and thus increase the hydraulic perturbation according to the recorded pore pressure profile ahead of the Test Drift before the connecting gallery excavation.

6.3 Hydraulic properties

All partners have simulated the problem in the frame of hydro-mechanical coupling behaviour for saturated soil.

The main hydraulic parameters required for the simulations are given in Table 6.5. It's noted that the big difference lays on the permeability value.

A non-linear permeability law has been used by G3S: the permeability depends on the plastic dilatancy $tr \varepsilon^p$. In consideration of the increase of the permeability in the EDZ, the hydraulic perturbation due to the T.D. excavation seems to be better modelled. In fact, a series of parametric studies has been conducted by G3S to investigate the influence of the permeability on the pore water pressure profiles measured from the CLIPEX piezometers (G3S, 2002).

Table 6.5 : Hydraulic properties related to Boom clay around depth of 225 m

	Saturated permeability $k_w \text{ (m/s)}$	Bulk modulus $\lambda \text{ (GPa)}$	Porosity N
SCK	$4 \cdot 10^{-12}$	2	0.39
GEOCONTROL	$4 \cdot 10^{-13}$	2	0.39
UPM	$1 \cdot 10^{-14}$	2	0.39
G3S	$k_w = k_{w0} \cdot (1 + \gamma tr \varepsilon^p)$	2	0.39
	$k_{w0} \approx 4 \cdot 10^{-12} \text{ m/s} , \gamma = 10^4$		

6.4 Lining properties used by different partners

The linings were modelled using the mechanical elements by most of partners and depicted by an elastic constitutive law with the following parameters (Table 6.6):

Table 6.6 : Lining properties

		Young's modulus E (GPa)	Poisson's ratio ν	Thickness a (cm)	External Radius R (m)	Stiffness K_{sn} (GPa)	Gap b (cm)
C.G.	SCK•CEN	30	0.3	40	≈ 2.4	≈ 5.5	2
	GEOCONTROL	25	0.25	60	2.405	6.65	?
	UPM	75	0.3	16	1.91	6.9	9
	G3S	Not modelled explicitly					5
T.D.	SCK•CEN	Not modelled explicitly					0
	GEOCONTROL	25	0.25	40	2.4	4.44	15
	UPM	75	0.3	16	1.91	6.9	9
	G3S	Not modelled explicitly					5

6.5 Parametric studies

In order to study the influence of different factors on the numerical results, SCK•CEN, GEOCONTROL and G3S have effectuated several parametric studies before doing blind predictions (GEOCONTROL, 2002, SCK•CEN, 2002, G3S, 2002). Concretely, following studies have been realised:

1. Influence of mechanical constitutive law (SCK•CEN and G3S)

As mentioned before, SCK•CEN has realized two predictions using modified cam-clay model and Mohr-coulomb model respectively. Globally, all results stay qualitatively same one another. The Mohr-coulomb model gives more displacements.

G3S has also conducted several modelisations in varying the mechanical constitutive laws: In addition to Cam-clay and Mohr-Coulomb models, a viscoplastic one named Rousset's model (Rousset, 1988) was used also. The simulation results showed that the Rousset's model could give larger pore water pressure perturbation around excavation of Test Drift. This implies that the viscosity plays certainly an important role on the HDZ generation (Hydraulic Disturbed Zone). Further research works are however needed to confirm this aspect.

2. Influence of the slope of Critical State Line (CSL) (SCK•CEN and G3S)

Two predictions with the values of the slope of critical state line (CSL) corresponding to the triaxial compression and extension state respectively have been carried out by SCK•CEN. Generally, the lower M is, the bigger the displacements are, the larger the generated plastic zone extends.

As mentioned in § 6.2.3.2, G3S has decreased significantly the frictional angle (and so the slope of CSL) to extend the EDZ and consequently, more hydraulic perturbation (more pore pressure drop after Test Drift excavation in the piezometers). The influence of the plasticity on the HDZ was put in evidence numerically.

3. Influence of null pore pressure limit condition (GEOCONTROL)

In case of null pore pressure limit condition, the main difference obtained in the results of the simulation is that the pore pressure does not reach so negative values as in the case of no pore pressure limit condition. The displacement is not affected so much by this condition.

4. Water flow effect (GEOCONTROL)

Letting the water flow during the simulation brings more dropping down of the pore pressure near the excavation face and lesser displacements.

5. Influence of permeability variation in function of plastic state (GEOCONTROL and G3S)

The results obtained by GEOCONTROL showed that increasing the permeability in the plastic zone induces higher displacements in the direction of the excavation and quite different distribution of pore pressure.

G3S's results showed that the non-linearity of the permeability in the EDZ can effectively modify the hydraulic perturbation pattern, but its effect seems less than the viscosity/plasticity effects.

6.6 *Blind prediction results : comparison between partners*

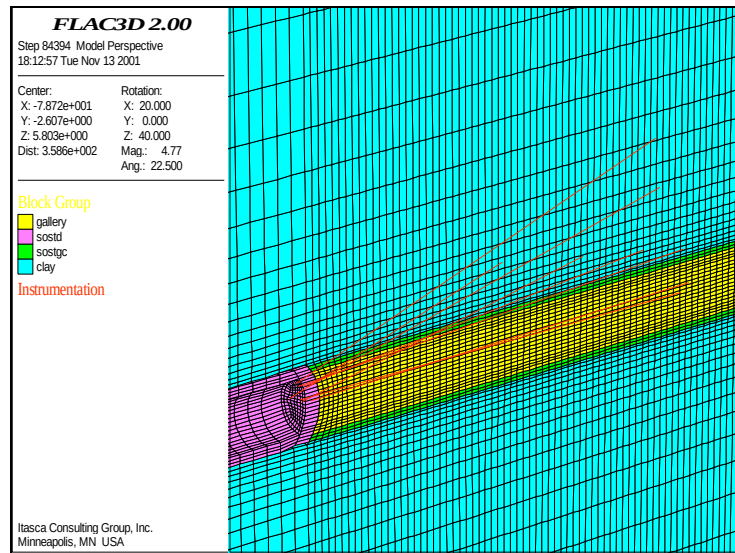
For the sake of conciseness, only the most important aspects through the comparison will be given hereafter and analysed. The whole prediction results obtained by each partner can be referred to their CLIPEX project reports (GEOCONTROL, 2002, SCK•CEN, 2002, G3S, 2002, UPM, 2002).

Table 6.7 gives at first a short sight on the main considerations for the prediction by each partner, which may be helpful to discern the difference between partners.

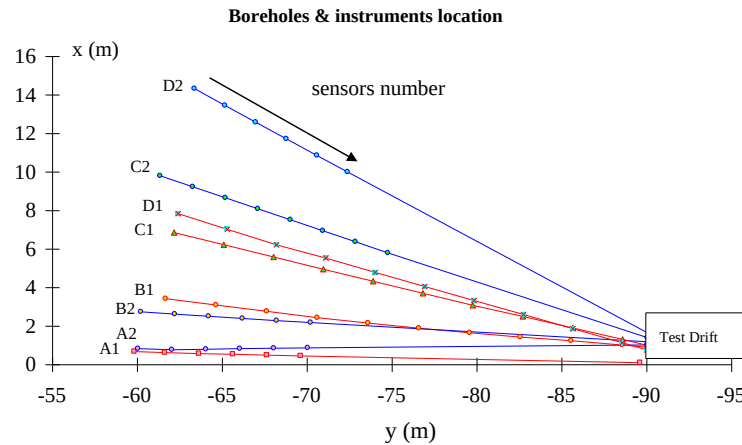
Table 6.7 : Main considerations of the prediction

	SCK•CEN	Geocontrol	UPM	G3S
Model	2D –axi-sym.	3D	2D –axi-sym.	2D –axi-sym.
Laws used	M-C-C; M-C	M-C	M-C-C	C-C
Initial conditions (σ_h, σ_v, u_w)	Homogeneous	Linear with depth	Homogeneous	Homogeneous
Interface clay-lining	Non	Yes	Non	Non
Gap (over cut)	2 cm	?	9 cm	5 cm
Drainage T.D. (1987-2001)	Yes	Non	Non	Yes

It's worth to note that the representation of the position of CLIPEX boreholes is somewhat different following the co-ordinate system on which the simulation is realised. In a 3D model (GEOCONTROL), the position of all sensors is represented as in the reality (Figure 6.20a). In the case of axi-symmetric model (SCK•CEN, UPM, G3S), it depends only on the co-ordinates x (radial position) and y (axial position) (Figure 6.20b). To facilitate the comparison, the names of all sensors in each borehole are recalled in Table 6.8.



a) in a 3D model



b) in an axisymmetrical system

Figure 6.20 : Location of the CLIPEX instrumentation boreholes

Table 6.8 : The names of boreholes and associated sensors

Boreholes	A2		B2		C2		D2		A1	B1	C1	D1
Sensors	WAx	KAx	WBx	KBx	WCx	KCx	WDx	KDx	EAx	IBx	ICx	Idx
Measures	u_w	σ	u_w	σ	u_w	σ	u_w	σ	u	u	u	u

6.6.1 Pore pressures predictions

The predicted pore pressures of the 4 piezometers in function of the advanced excavation front position of the connecting gallery are given in Figure 6.21 and Figure 6.22.

Generally speaking, the evolutions of pore pressure predicted by each partner are very comparable:

- The pore water pressure increases slightly (0.2 to 2.5 bar) before dropping down with the advanced front. The drop of pore water pressure is certainly due to the volumetric

dilatation of the soil skeleton following the excavation (decompression), and the slight increase may be induced by the undrained responses before the dilatation.

- Due to the high hydromechanical coupling formulation, the high values of suction (negative pore pressure) are obtained numerically in both A2 and B2 borehole (Figure 6.21a, b).

Through comparison, we can draw attention on the following points:

- The results of SCK•CEN and those of GEOCONTROL are most closed to each other comparing with those of G3S and of UPM.
- For the boreholes installed nearly on the axis of connecting gallery (A2 and B2), the 2D axisymmetrical model made by SCK•CEN gives almost the same results as those of the 3D model (GEOCONTROL) (Figure 6.21a, b,c,d).
- The pore water pressures obtained by all sensors before connecting gallery excavation differ not very much from initial in situ ones except the results of G3S (Figure 6.22j). It implies that, with the normal values of parameters as used by SCK•CEN, UPM and GEOCONTROL, the excavation of TD and 14 year's drainage (realised only by SCK•CEN) did not perturb too much the pore water pressure at those positions where the sensors are. The slight difference existing in the results of GEOCONTROL comes mainly from the non-homogeneous initial pressure inputted by the 3D model (1 bar corresponds to about 10 meters of water head). This numerical aspect can be visualised more clearly with the help of Figure 6.23 to Figure 6.25. We observe that the Test Drift excavation induces only slight pore pressure perturbation in a very limited zone (2-5m in radial direction, Figure 6.23, Figure 6.25). The long term drainage of Test Drift extends the perturbation zone to about 20 m in radial direction (Figure 6.24, Figure 6.25). Even though, it doesn't reach the location of the sensors.

This is however not true for the results of G3S (Figure 6.22j). In reality, G3S has adjusted significantly the mechanical parameter (internal frictional angle) to fit better the pore pressure profiles measured by CLIPEX before excavation of connecting gallery. This significant modification may be discussible, since it may "smother " the real mechanism of soil hydro-mechanical behaviour. Additional laboratory tests to determine the evolution of this material characteristic in function of the stress – strain paths would be however helpful to find an apprehensible model to improve the numerical simulations.

- An axi-symmetrical model predicts nearly same evolution as a 3D model for the sensors far away from the axis of gallery (WCx and WDx). The curves obtained by all are more or less parallel (Figure 6.21e, f and Figure 6.22g, h). However it seems that the 3D model gives bigger rise and fall for the near face sensors as shown by the sensor WC8: SCK•CEN gives about 6 bars of drop, G3S nearly 8 bars and GEOCONTROL about 10 bar (Figure 6.21f). It's worth to mention that the G3S's results didn't give more pore pressure fluctuation during the connecting gallery excavation in spite of the lower value of the frictional angle used. His non-linear law of permeability didn't seem to contribute more hydraulic perturbation neither.

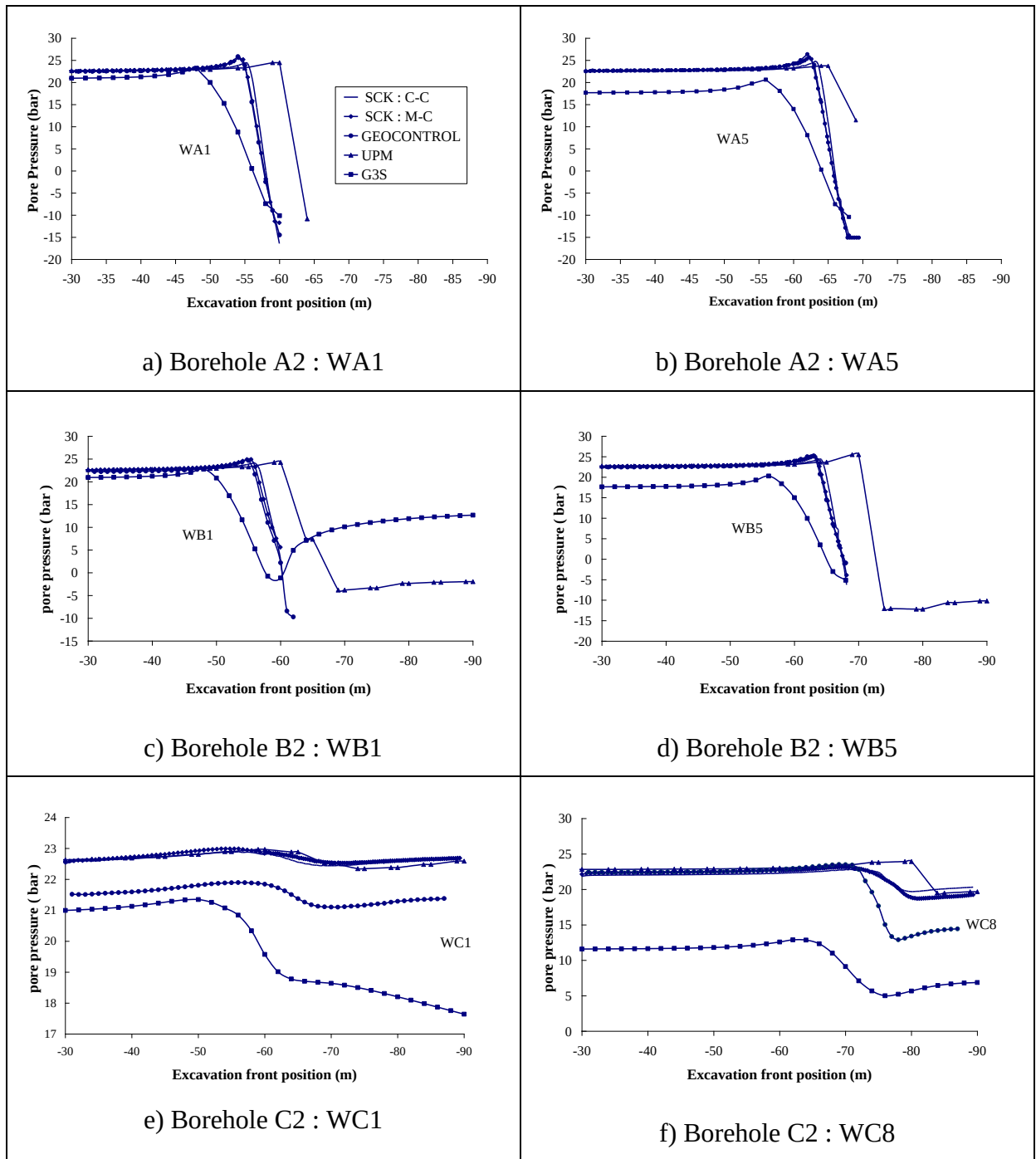


Figure 6.21 : Pore water pressure variation during excavation of connecting gallery

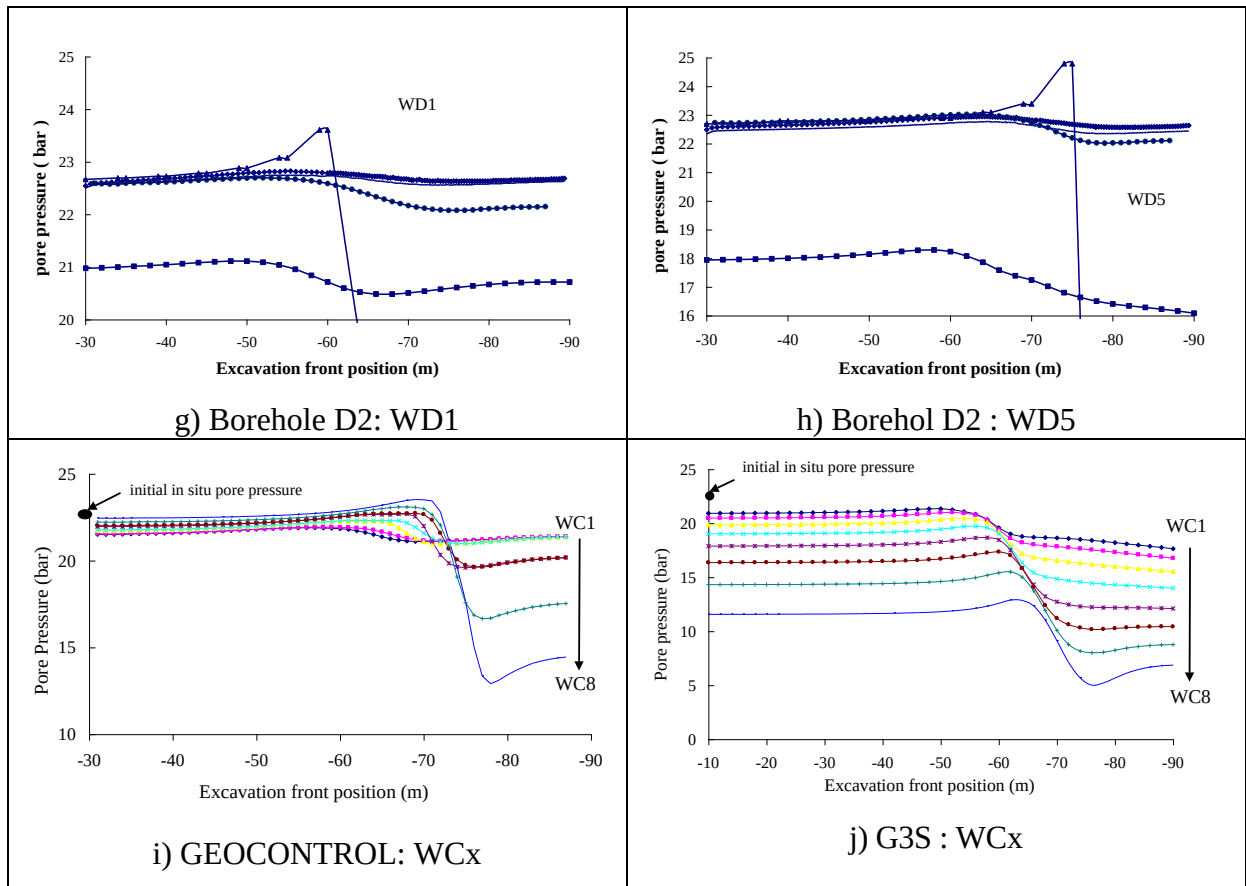


Figure 6.22 : Pore water pressure variation during excavation of connecting gallery

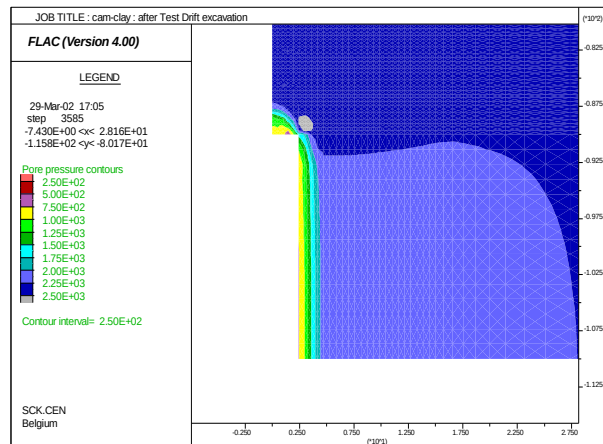


Figure 6.23 : Pore water pressure distribution after excavation of TD

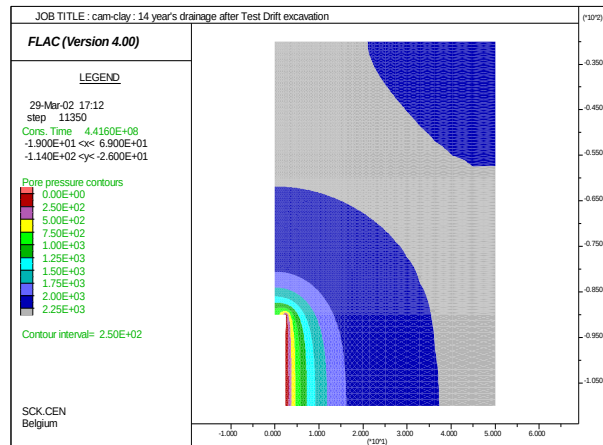


Figure 6.24 : Pore water pressure distribution after 14 year's drainage of TD

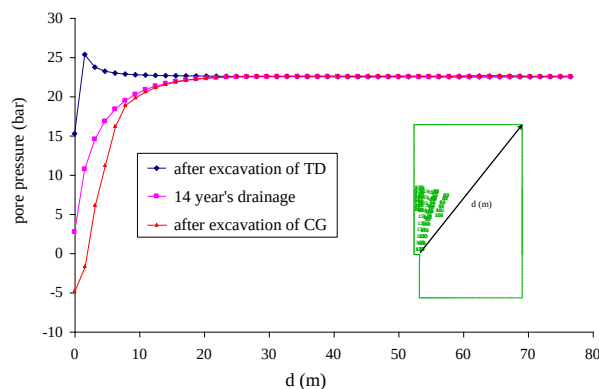


Figure 6.25 : Pore water pressure profile along A-A'
(obtained from SCK's model with the modified cam clay model).

6.6.2 Displacements predictions

To facilitate the comparison, the results are compared amongst two following groups:

- The couple UPM – SCK•CEN cam-clay:

The results of UPM are compared to that of SCK•CEN obtained by modified cam-clay, since both have used a 2D axi-symmetrical model and Modified Cam-Clay law with nearly same values of parameters.

- The triple GEOCONTROL – SCK•CEN Mohr-Coulomb – G3S:

The prediction results of GEOCONTROL are faced to those of SCK•CEN (Mohr-Coulomb) and of G3S. To do so, the radial displacements obtained by later ones are decomposed into two components taking into account the local cylindrical co-ordinates of each sensor: horizontal ones and vertical ones.

6.6.2.1 Comparison between UPM and SCK•CEN with Modified Cam-Clay

Part of predicted results are compared in Figure 6.26, Figure 6.27 and Figure 6.28.

Even though both teams use the same model and the same constitutive law, the dispersion seems important. The reason for this dispersion is very hard to discern. The big difference on the lining properties and on the overcut (gap) may be the main reason. Further analysis is anyway necessary. It's worth however to note that part of the UPM's prediction shows more or less same as that of SCK•CEN (borehole A1, C1). The evolution of the displacement predicted by two teams stay comparable on the whole.

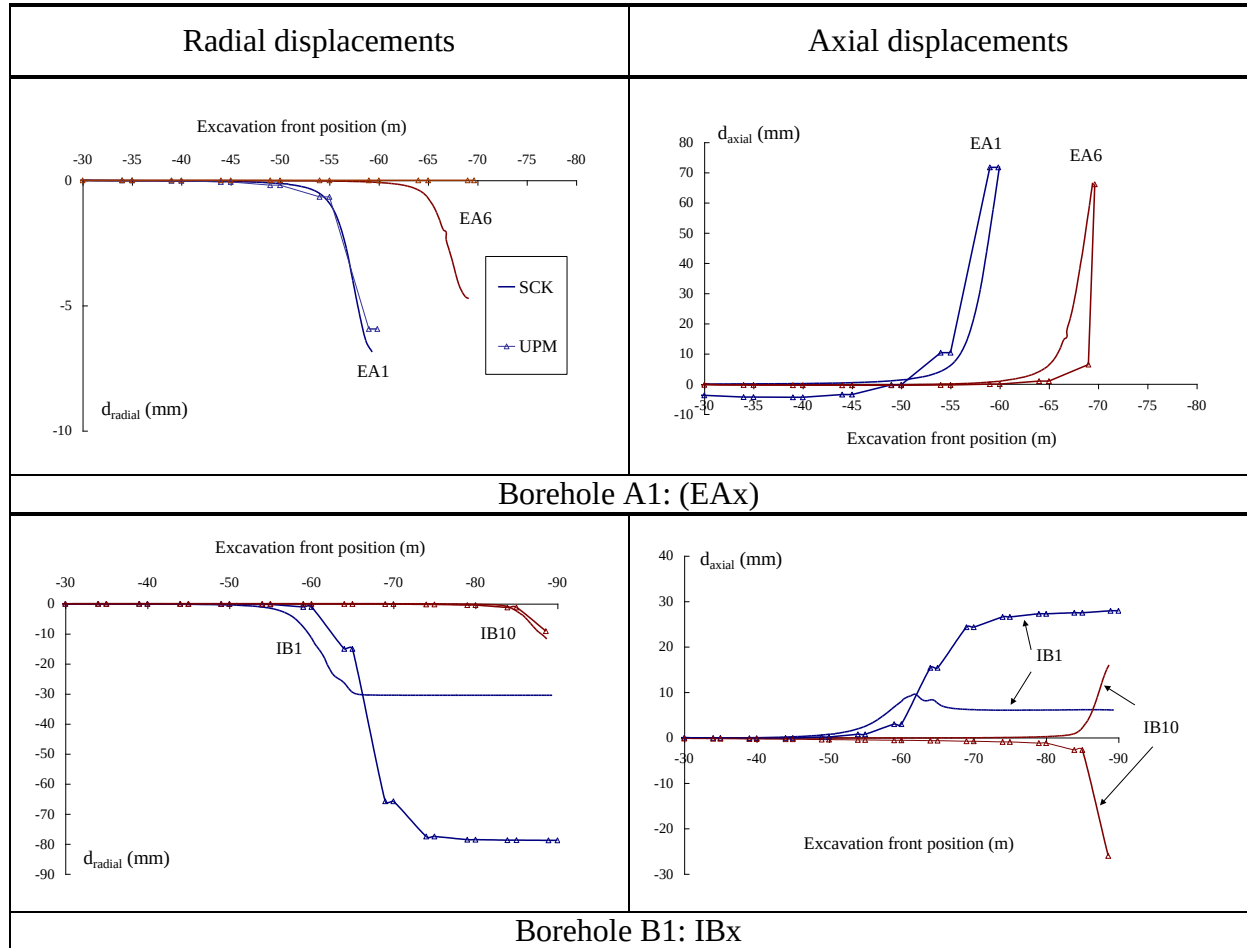


Figure 6.26: Comparison between UPM and SCK•CEN cam-clay (A1 and B1)

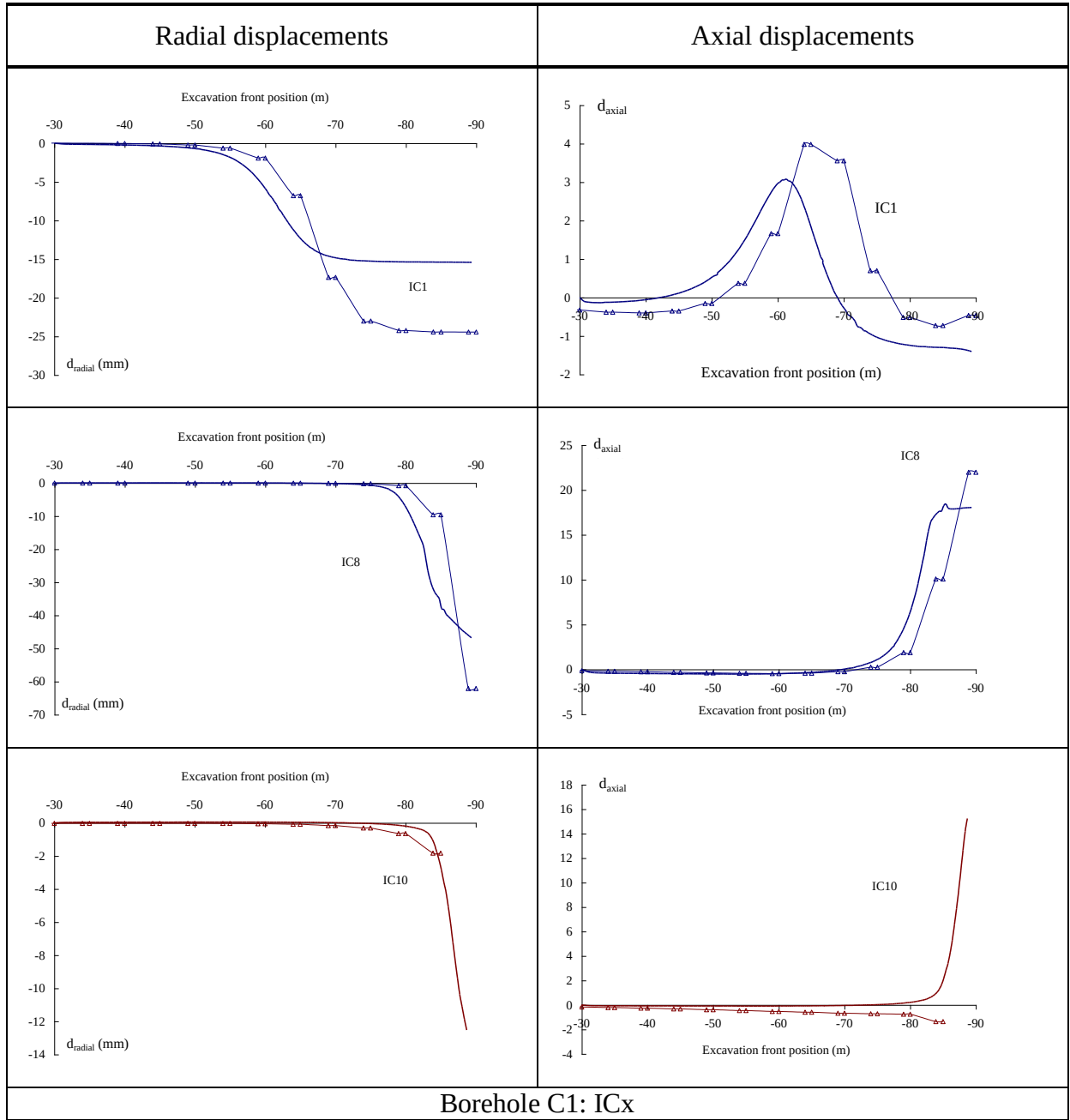


Figure 6.27: Comparison between UPM and SCK•CEN cam-clay (C1)

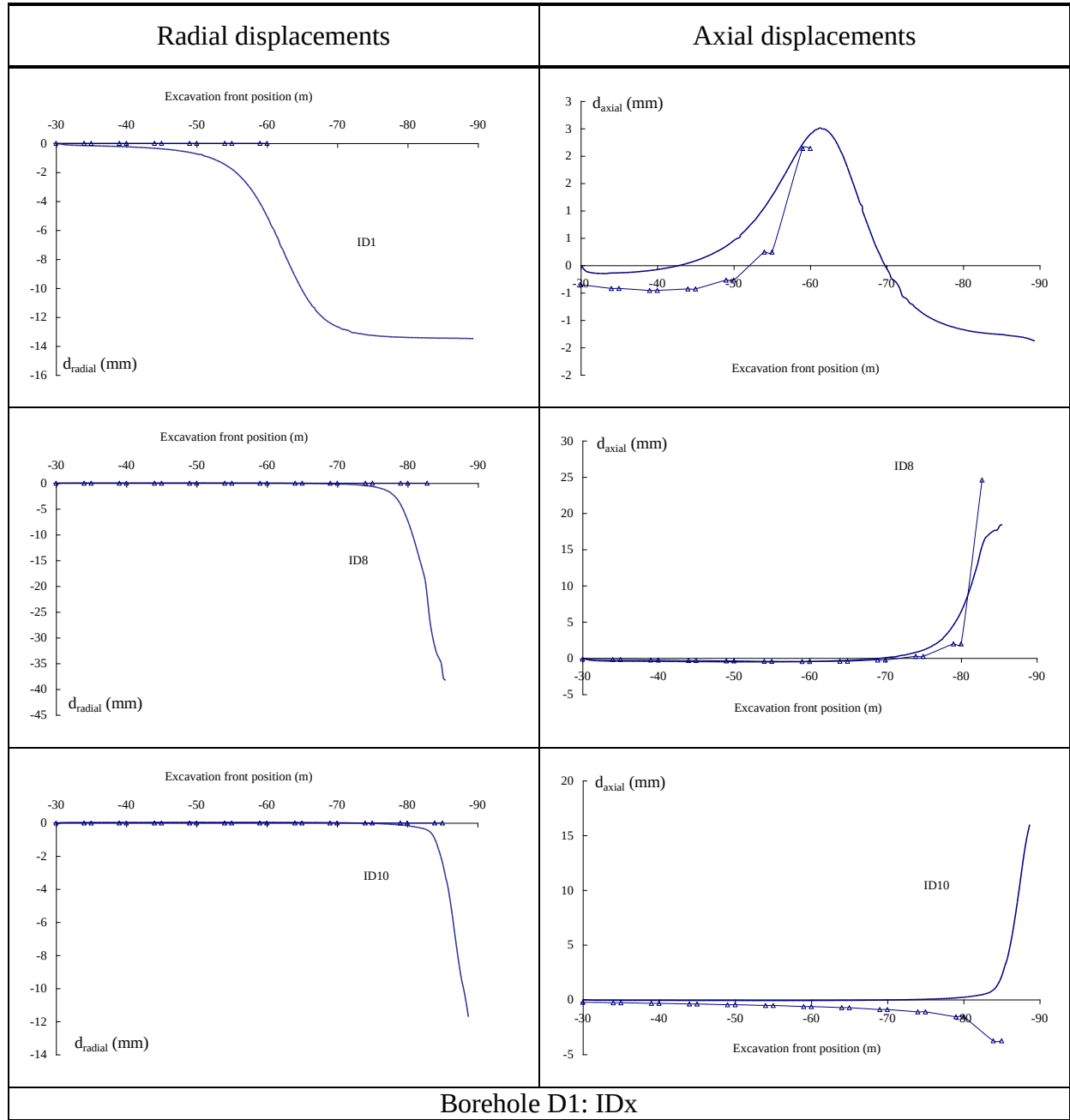


Figure 6.28: Comparison between UPM and SCK•CEN cam-clay (D1)

6.6.2.2 Comparison amongst GEOCONTROL, SCK•CEN (Mohr-Coulomb) and G3S

Parts of the predicted displacements obtained by cited three teams at different borehole are drawn in Figure 6.29, Figure 6.30 and Figure 6.31.

In general, the numerical predictions of displacements give following information:

- The maximum axial displacements take place on the axe of the connecting gallery, as shown by the borehole A1.

- The accumulate displacements (radial in the axi-symmetrical system, horizontal/vertical in the 3D model) seems to tend to be stable when the working front goes past the corresponding sensors (especially for boreholes B1 and C1) (see Figure 6.31).
- It's worth to note that the borehole A1 and part of B1 were destroyed by the coming gallery, as a result, the displacements records are not complete.

In comparison, the following remarks can be made:

- Generally, three teams predict quite similar **evolution** in function of advancing work face for the borehole A1.
- The results obtained by SCK•CEN match better to those of GEOCONTROL. However there exists often a big contrast between G3S and other two teams, especially for the axial displacements (IB1, IC10 and ID10).
- Comparing the results between SCK•CEN and GEOCONTROL, it seems that the 2D axisymmetrical model predicts somewhat less displacements than a 3D model (except the sensor ID8 and ID10).
- G3S predicts fewer displacements for the distant sensors (IC1, ID1), but more displacements for the sensors close to the gallery axis (IC10, ID10).
- For the borehole C1, which is inclined up in a vertical plane, the 2D axisymmetrical model obtains logically a vertical displacement closer to the 3D model. For the same logic, the 2D axisymmetrical model obtains a horizontal displacement closer to that of the 3D model for the borehole installed almost in a horizontal plane (borehole D1).

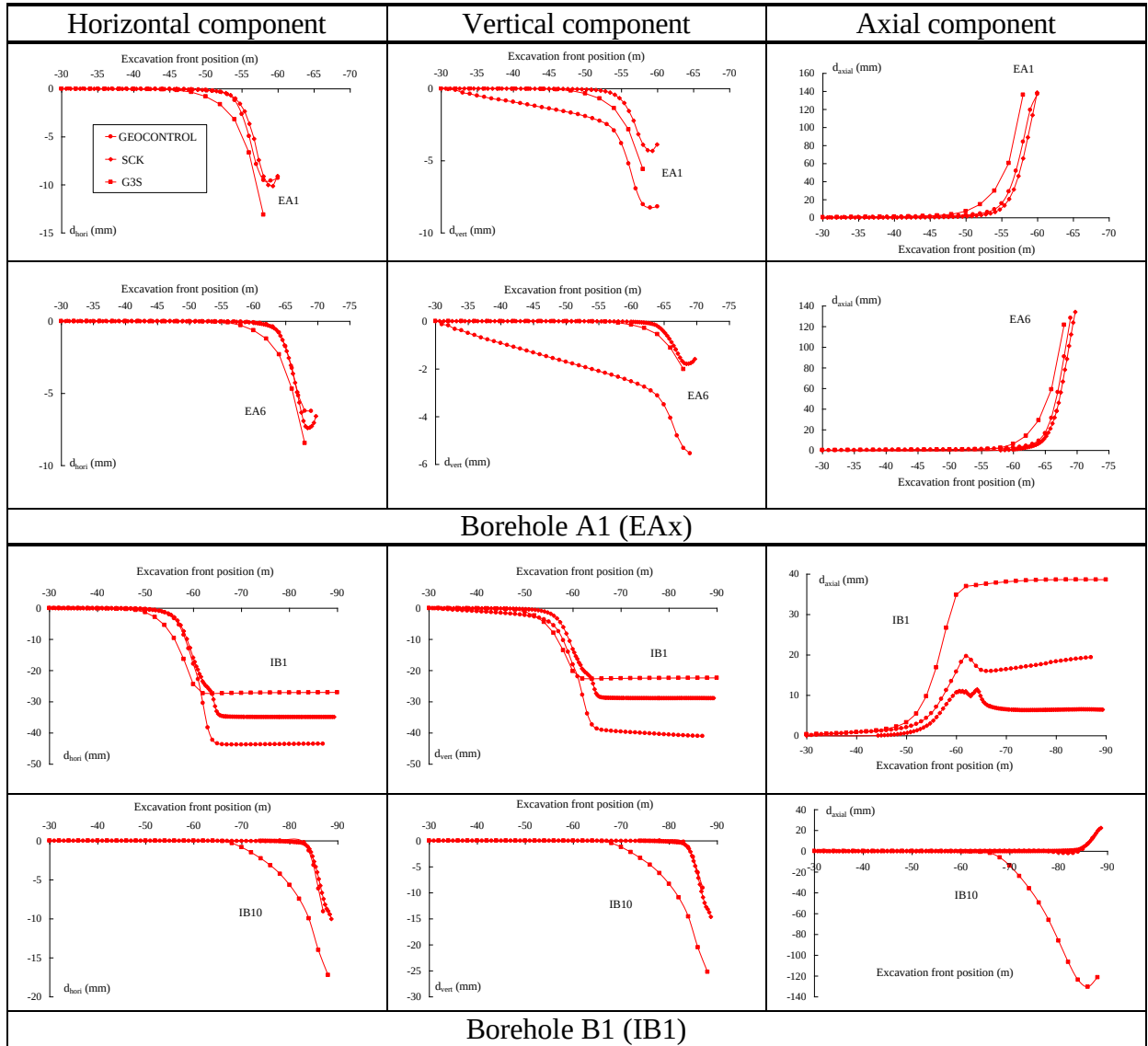


Figure 6.29: Comparison between GEOCONTROL, SCK•CEN and G3S (A1 and B1)

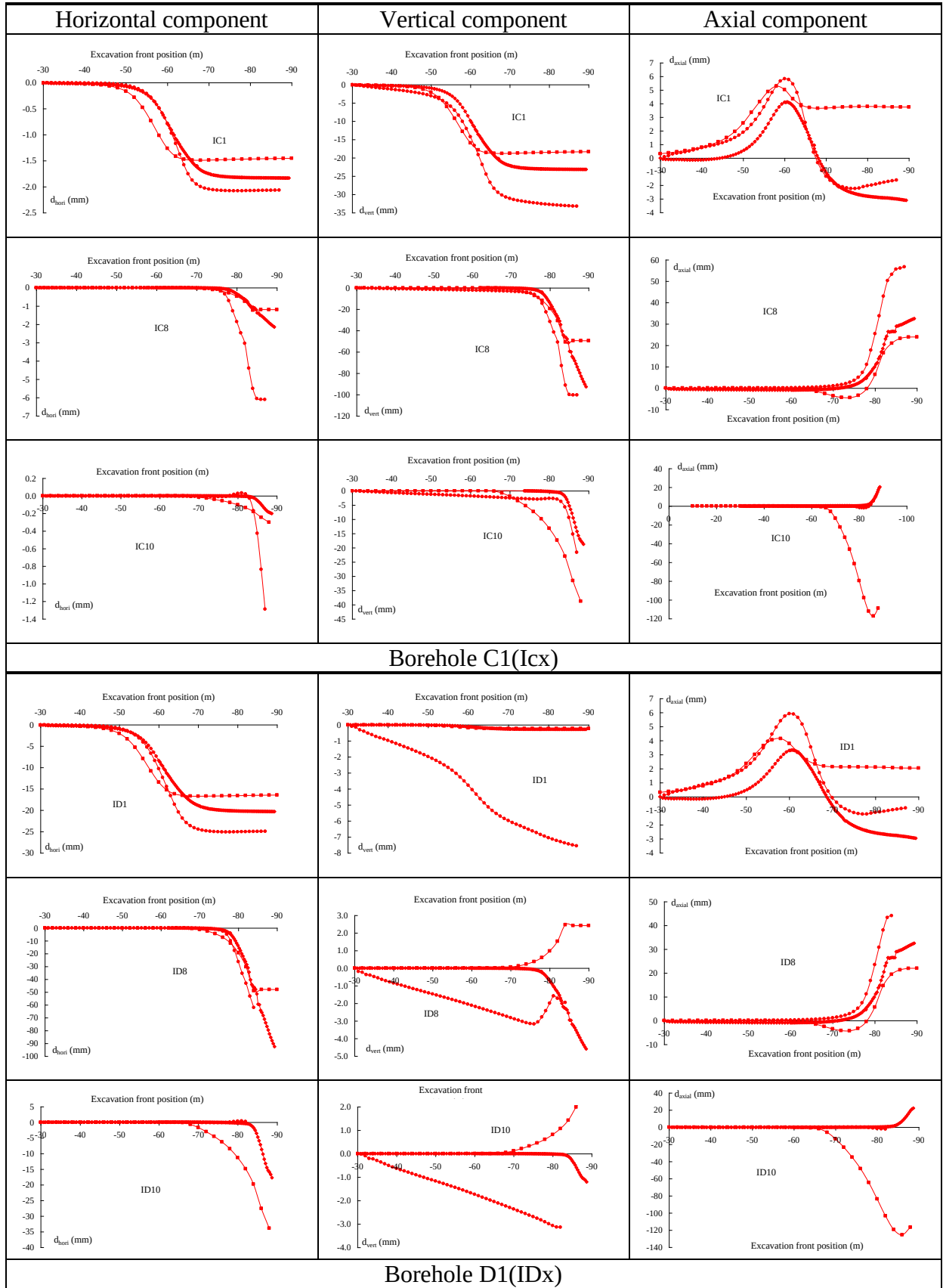


Figure 6.30: Comparison between GEOCONTROL, SCK•CEN and G3S (C1 and D1)

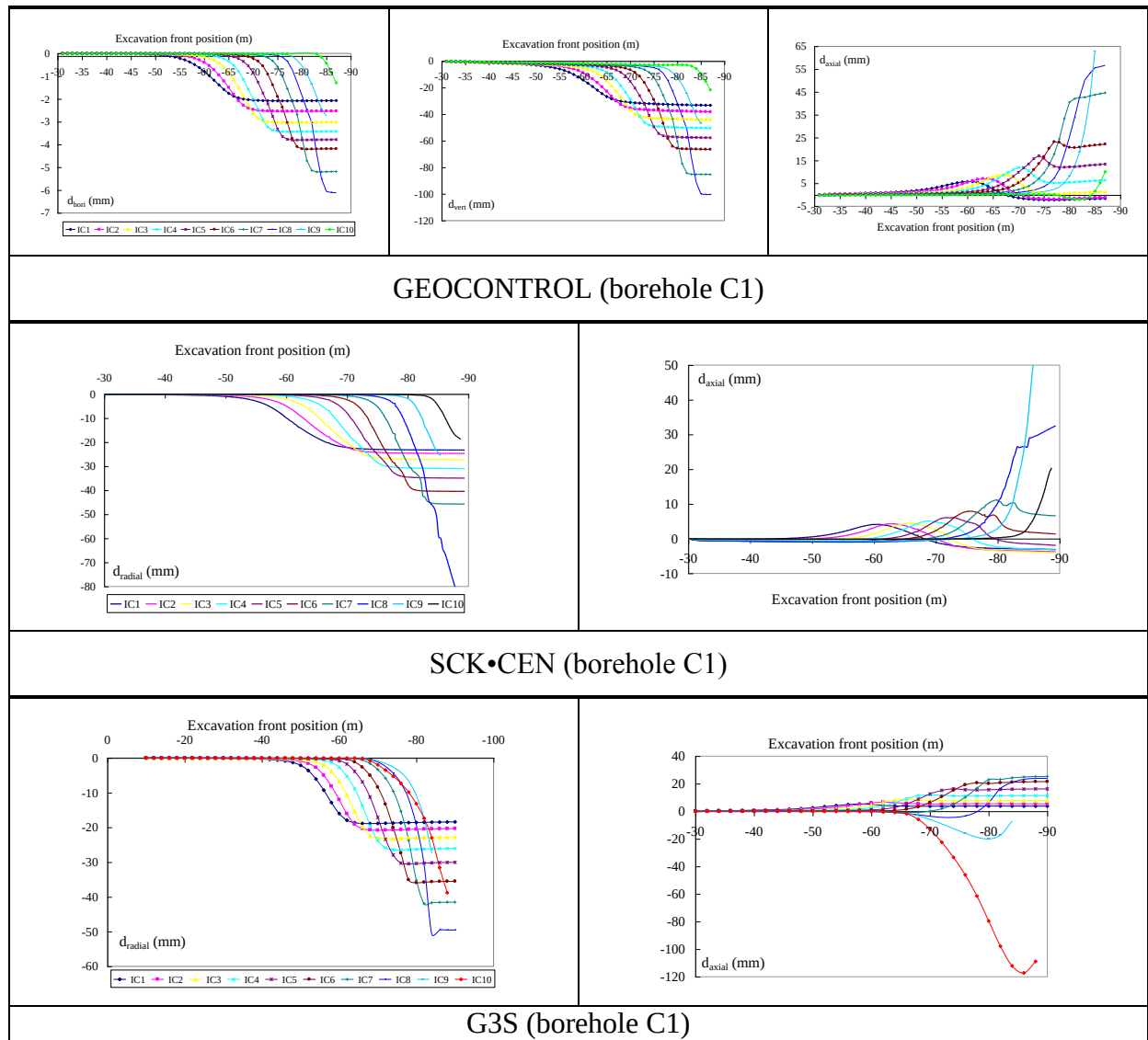


Figure 6.31 : Comparison amongst GEOCONTROL, SCK•CEN and G3S

6.6.3 Total stresses predictions

The total stress predictions are again brought together into two groups as:

- UPM's results with those of SCK•CEN (M-C-C),
- GEOCONTROL's results with those of SCK•CEN (M-C) and G3S.

6.6.3.1 Comparison between UPM and SCK•CEN with Modified Cam-Clay

Only radial and axial stresses are compared (Figure 6.32).

For the boreholes near the gallery (A2 and B2), the total stresses obtained by two teams are quite comparable. It's not the case for the boreholes faraway (C2 and D2), where the difference between two predictions keeps very big. UPM's model gives much bigger variation of stress. More detail information is then needed for further analysis for this big existing difference.

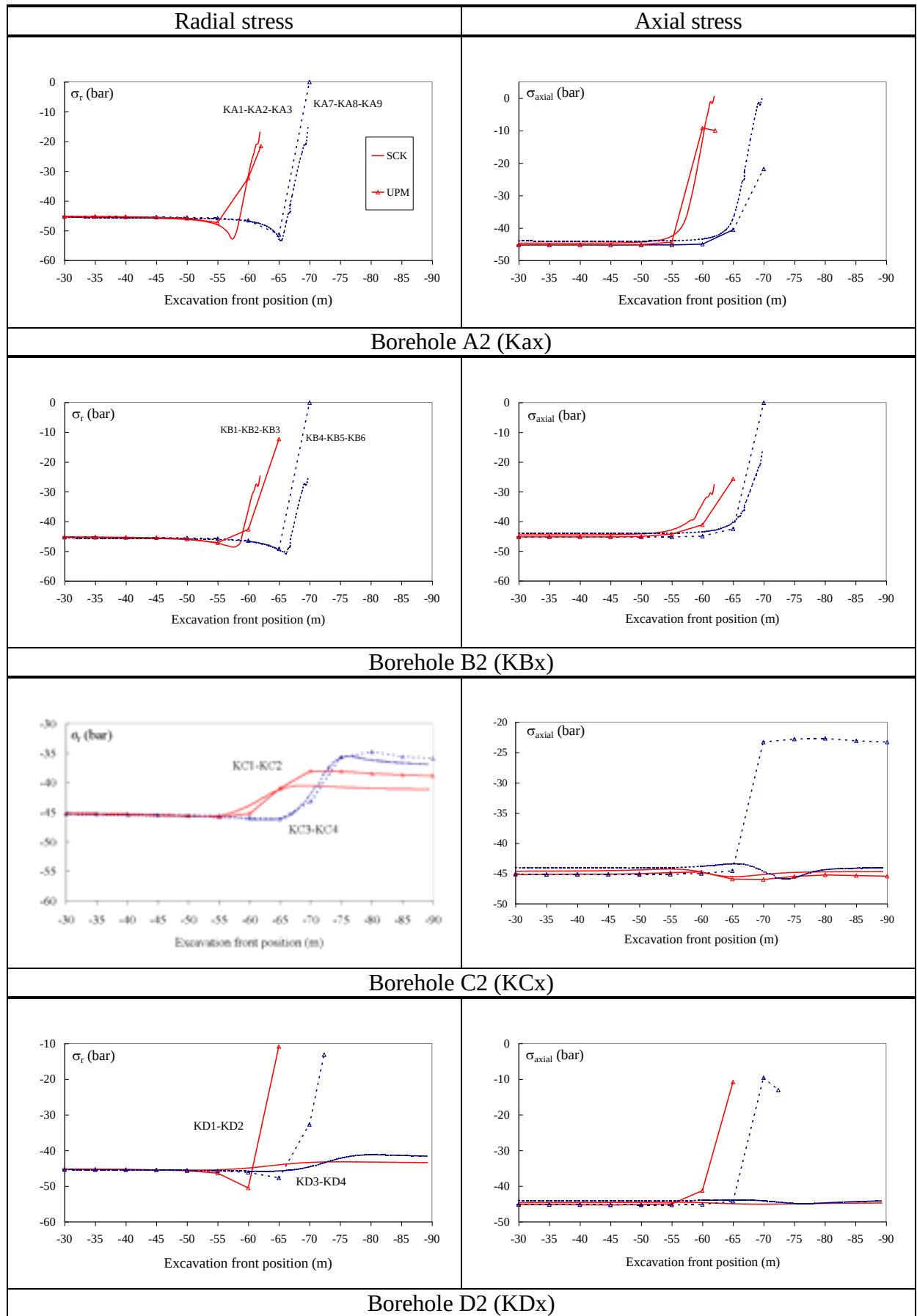


Figure 6.32 : Total stresses comparison between UPM and SCK•CEN cam clay

6.6.3.2 Comparison between GEOCONTROL, SCK•CEN (Mohr-Coulomb) and G3S

To be able to compare the results between GEOCONTROL, SCK•CEN and G3S, it's imperative to express the total stress to be compared in a same axis system. Consequently, the results of SCK•CEN and G3S are converted into the 3D Cartesian system with the help of following formula:

$$\sigma_{YY} = \sigma_r \cos^2 \alpha + \sigma_\theta \sin^2 \alpha$$

$$\sigma_{ZZ} = \sigma_r \cos^2(\alpha + \pi/2) + \sigma_\theta \sin^2(\alpha + \pi/2)$$

$$\sigma_{YZ} = \frac{\sigma_\theta - \sigma_r}{2} \sin 2\alpha$$

where σ_r and σ_θ are the total radial stress and the total orthoradial stress obtained by 2D axisymmetrical model respectively. α is the polar co-ordinate of sensors. σ_{YY} and σ_{ZZ} represent the horizontal and vertical components of total stress in the 3D Cartesian system chosen by GEOCONTROL. σ_{YZ} is the shear stress.

The total axial stresses stay the same in two systems.

The results of SCK•CEN and G3S obtained thus are putted together with that of GEOCONTROL as shown in Figure 6.33.

It can be observed that, when using same constitutive law, same parameters, and without taking into account the 2nd shaft excavation, the results obtained by a 2D axisymmetrical model (SCK•CEN) are rather coherent with that gained by a 3D model (GEOCONTROL):

- The total stresses for the near face boreholes A2 and B2 are almost without difference.
- If there exists some difference for the boreholes faraway C2 and D2, their **evolution** with the advancing front are remarkable same (except σ_{YY} in KC3-KC4). The existing difference may simply due to the slight different initial conditions.

The G3S's results stay quite comparable with those of two other teams: same tendency can be observed. The contrast with others comes mainly from the same reason as mentioned before, the lower value of the frictional angle brings the soil yielding earlier. Actually, the "peaks" in the G3S's results appear several meters earlier.

We observe that in both boreholes A2 and B2, the non-axial stresses present a peak that, in fact, represents the development of plastic zone ahead of the excavation front (about 5-6m). This fact is more visible on the Figure 6.34, which indicates that the plastic zone is generated about 5-6 m ahead of the front. It's noted that the peak of the pore pressure evolution (see Figure 6.21 above) is also an echo of this plastic zone development.

However, it's difficult to analyse the evolution of the total stresses recorded in the boreholes C2 and D2.

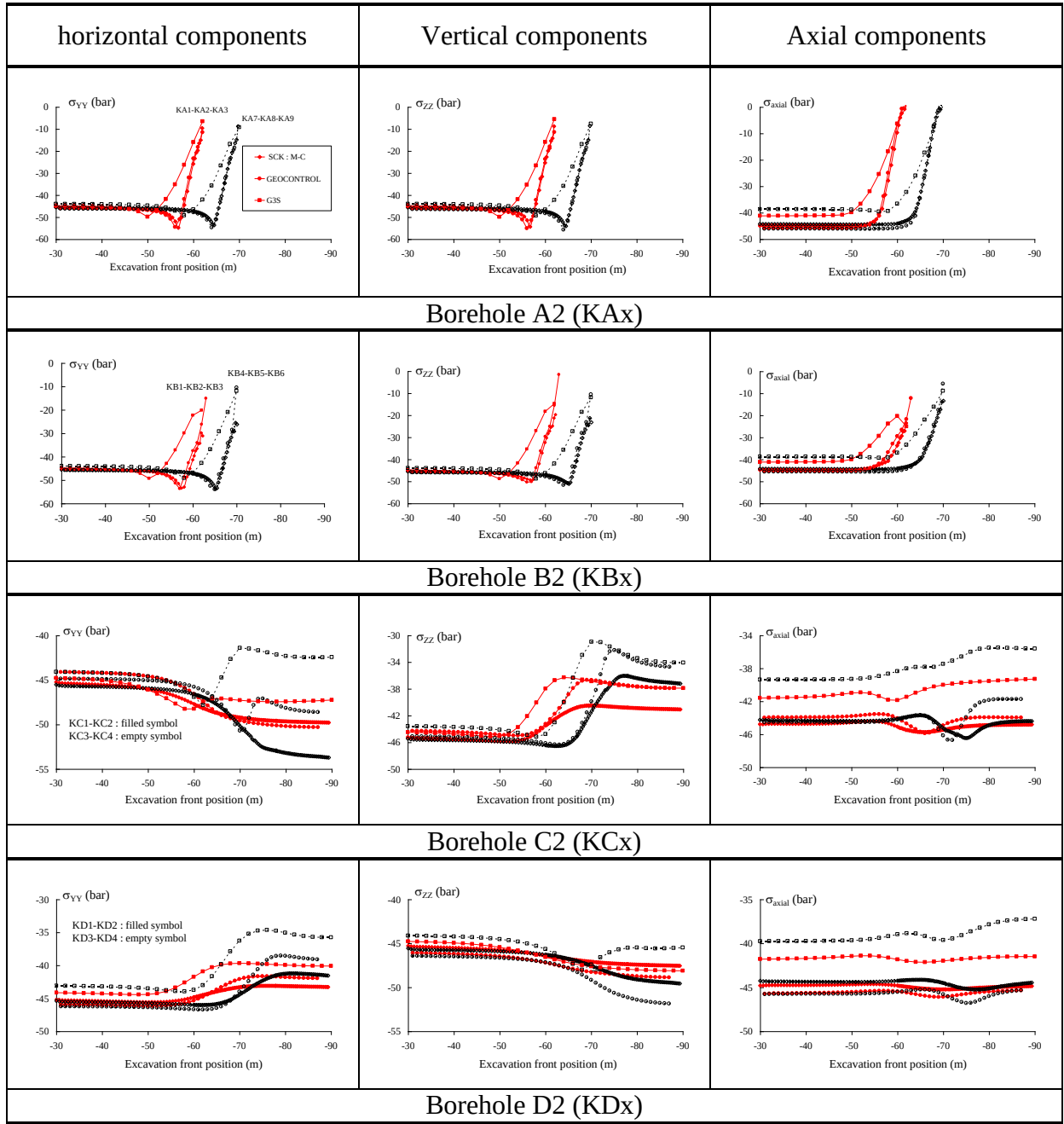


Figure 6.33 : Total stresses comparison between GEOCONTROL and SCK•CEN

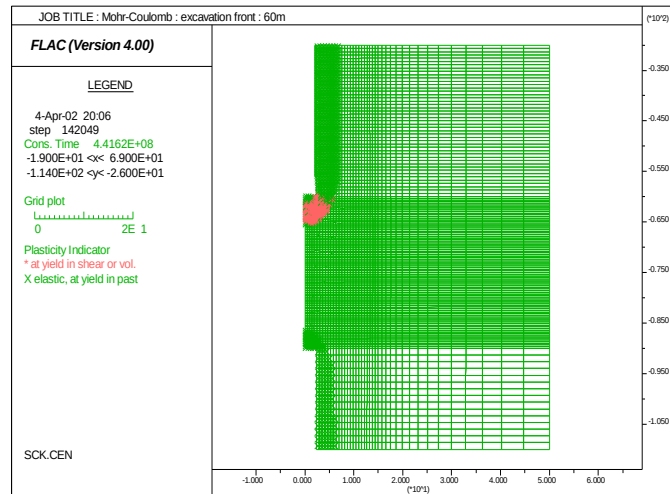


Figure 6.34 : Plasticity indicator when the front is at -60 m (SCK, M-C)

6.6.4 Conclusions on the numerical results

Generally speaking, the results obtained by all partners are coherent. The discrepancy exists however, which asks further studies. At first moment, we may attribute the observed dispersion to following factors:

- The different size of mesh may have an important influence;
- Different integration method of constitutive law may result in the dispersion;
- The diameter of the gallery (about 4 m in UPM, 5 m in other three teams) may be the one of the reasons which yields the UPM's results diverge often from others;
- The different lining properties and corresponding overcut used in the model may cause the difference also.
- The permeability has certainly an effect.
- Of course, a 2D axisymmetrical model has his limit with respect to a 3D model.

Nevertheless, based on the comparison between GEOCONTROL and SCK•CEN, it appears that, if not consider the influence of the 2nd shaft and the isotropic condition is considered, a 2D axisymmetrical model can produce quite challenging results with respect to a 3D model. At least from point of view of pore pressure and total stress. Concerning the displacements, a 2D model gives somewhat smaller value for the sensors faraway.

- The displacement is more sensible to the numerical model used: it varies following the constitutive law and model type (2D or 3D) adopted.
- The characteristics of Boom Clay inputted in the model has without doubt a significant influence on the prediction results, such as the frictional angle, preconsolidation pressure, etc.

In order to get a better sight of the numerical results during the excavation of gallery, some representative results taken from the SCK's simulations are given in Figure 6.35 as an example.

It can be concluded:

- The total pressure reaches up to 5.5 MPa
- The pressure exercise on the lining is about 2 MPa.
- The axial displacement ahead of the face depends largely on the constitutive law used: around 7 cm based on a modified cam-clay, it can be doubled when using a Mohr-coulomb model.
- An initial radial convergence at gallery circumference ranges between 2.5 cm to 4 cm according the law used. It seems to stabilise at about 6 cm (modified cam clay model) and 10 cm (Mohr-Coulomb model) after installation of lining.
- The plastic zone extends around 5 – 6 meters according to the model used. The disturbed zone extends to about 25 m in radial direction.

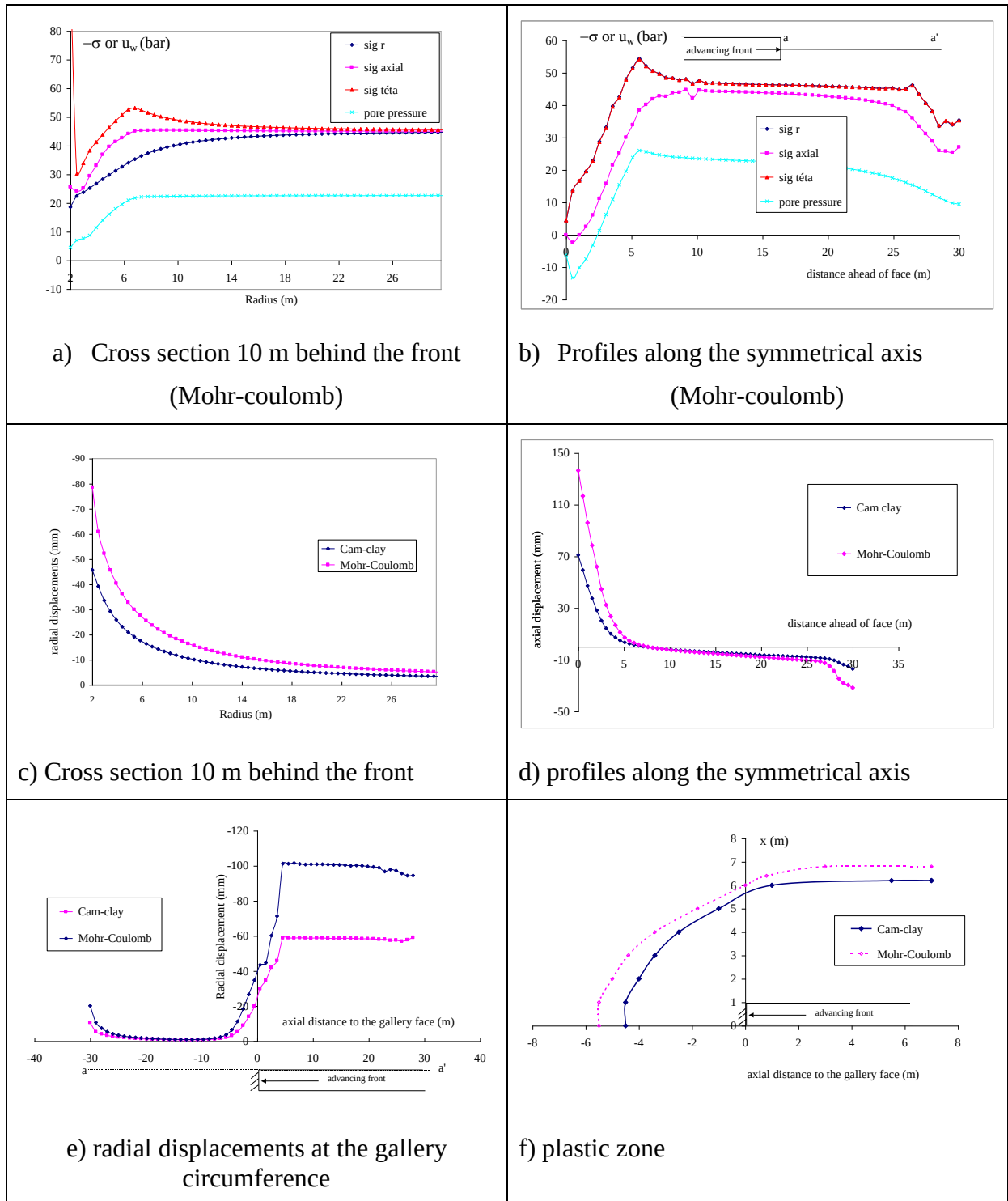


Figure 6.35 : Numerical results when the front is at -60 m position

7 The excavation of the connecting gallery

Works were carried out by the Joint Venture SCM (Smet-Tunnelling • Wayss & Freytag AG • Deilmann-Haniel GmbH) on behalf of the Client EIG EURDICE, and supervised by SECO. Tractebel Development Engineering was in charge of the overall concept, lining design and tender documents.

The construction of the connecting gallery has been realised with a Tunnel Boring Machine (TBM) composed of a 2.3 m long shield, a road-header for the excavation of the rock and a bird-wing erector system for the installation of the lining (see Figure 7.1). The shield is equipped with a cutting head to ensure a smooth excavated profile. The wedge block system is used for the lining, which is an expanded lining system. In order to reduce the plastic zone created by the tunnelling process a fast advancing rate is required, the overexcavation is reduced to a minimum, and the lining is installed as soon as practical possible to minimise radial movement of the clay wall.

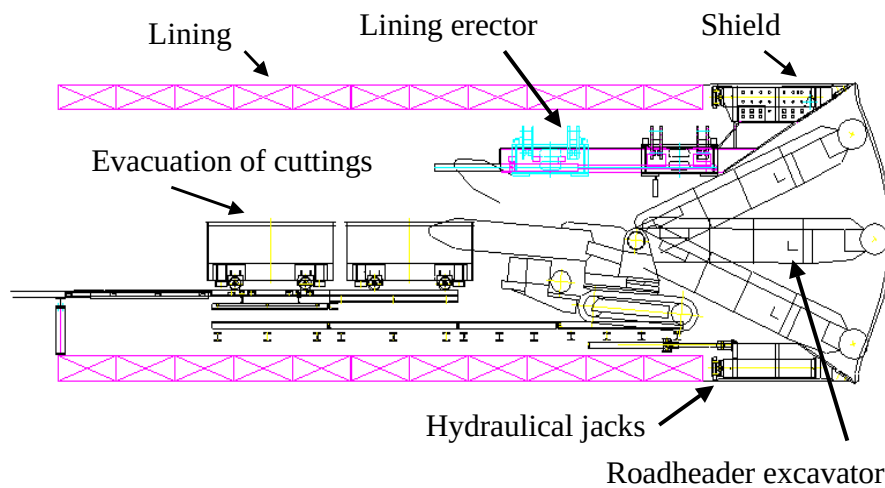


Figure 7.1: The design of the tunnel machine

Works are carried out at 230 meters depth, the only access being the existing 2nd shaft ($\varnothing$ 3.0 m). The lifting capacity of the hoisting equipment is limited to 3,7 tons and has a free clearance of 1.2 x 2 x 2.9 m, hence restraining all dimensions of the TBM components, muck lorries and concrete lining segments. The internal diameter of the gallery is 4.0 m; total length between the starting chamber and the HADES laboratory is 80 meter.

7.1 The Mounting Chamber

Starting from the existing Northern onset, the excavation front is reinforced with glass fibre anchors of 3 and 8 meter prior to the start of the excavation works. Then a cavern of approximately $\varnothing$ 6.40 m x 3,15 m is dug, using Atlas Copco Brokk-180 equipment (hydraulic hammer) and a temporary lining of sliding ribs (TH44) and shotcrete is installed. This chamber enabled to mount the tunnelling machine and minimal backup equipment (see Figure 7.2 to Figure 7.4).

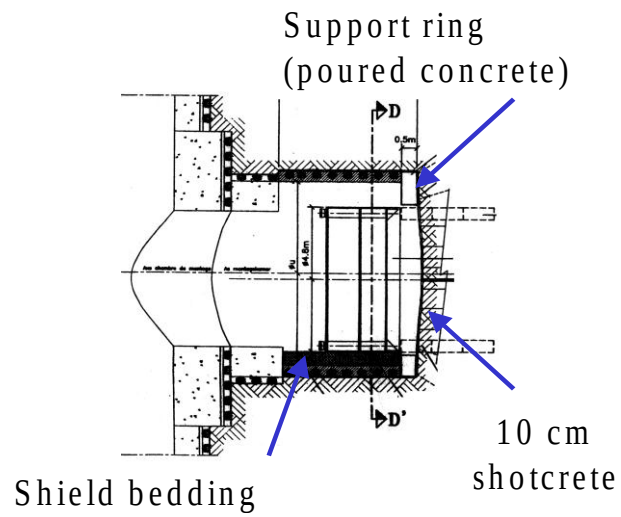


Figure 7.2: The design of the mounting chamber



Figure 7.3: The start of the excavation of the mounting chamber

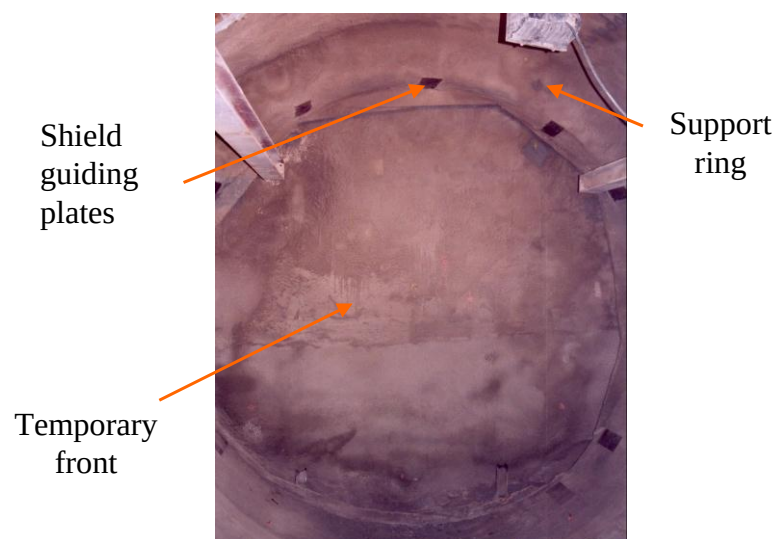


Figure 7.4: The mounting chamber

7.2 The Wedge block lining

The lining used is of an expanded type. The expanded lining or wedge block technique has been developed for cohesive soils with a stand-up time of several hours (as the over-consolidated London clay). The shield used are provided with a sharp cutting edge which serves also as an over-cut. The edge of the shield cuts smoothly the excavation profile to a perfect circle while the inside is excavated with a roadheader. At the rear of the shield this leads to a smooth cylindrical unsupported zone which facilitates the building of a ring of concrete segments. A ring consists of a number of concrete segments and two wedge shaped key segments. The introduction of the two wedges (Figure 7.5) expands the ring against excavated profile inducing pre-stress in the lining. The absence of bolts and grouting behind the ring considerably reduces the cycle time for excavation and erection of a ring.

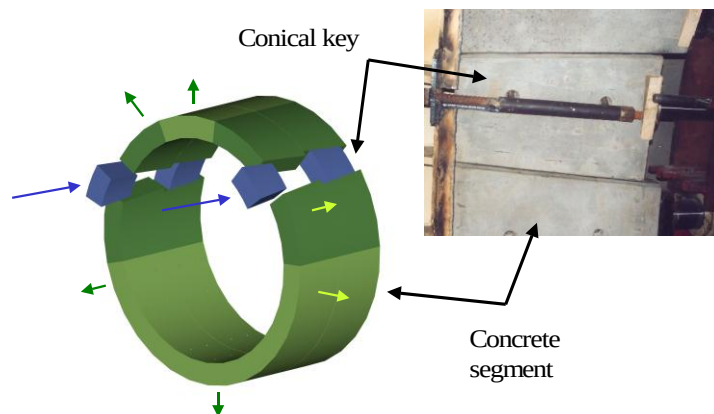


Figure 7.5: The wedge block system

The cohesive Boom clay, which can be classified as a 'soft rock' is perfectly suited to apply this technique. The main difference with the tunnels in the London clay is the depth and the resulting higher pressure relief, convergences, and convergence rate. The CLIPEX computation has allowed to predict the convergence and therefore to optimise the geometry of the shield. To cope with the high convergences, occurring in front of the excavation, from the front to the rear of the shield and further in the unsupported zone, the shield is provided with an variable over-cut (0-30 mm), a conicity of 10 mm on diameter and finally the rear diameter of the shield (4820 mm) is 20 mm greater than the average lining diameter (4800 mm) due to further convergence at the back of the shield.

7.3 The tunnelling machine

The entire equipment (Figure 7.6) consists of a circular cutting shoe with an external diameter of 4820 mm at the rear and a maximal cutting diameter at the front of 4890 mm including the variable oversize (0-30 mm) and the conical shape (10 mm). Twenty hydraulic jacks at 200 tons each enable the shield to penetrate the stiff clay and execute the necessary steering corrections. The jacks have a stroke of 1200 mm, compatible with the length of the segments (1000 mm). Two extra hydraulic jacks of 80 tonnes are foreseen for the introduction of the two wedges or key segments to complete and pre-stress the ring.

The steel cutting shoe has a wall thickness of 50 mm, and is reinforced with extra stiffeners, hence withstanding to the earth pressures up to 2.4 MPa and 4000 tonnes of thrust capacity.

A Bird Wing erector is hang up to this steel structure of the cutting shoe to pick up and install segments (weight 1.25 tons), within a range of diameters up to 4890 mm (in the case no convergence should occur).

Excavation of the clay at the inside of the cutting shoe is done with a Fuchs roadheader, equipped with special teeth. The Fuchs has been modified to the required excavation diameter, and the capacity of the machine is calculated with respect to the maximum capacity of the hoisting equipment. A total of 6 muck truck of 1.5 m³ are supplied for horizontal and vertical transport of the muck.



Figure 7.6: Global view of the TBM machine

7.4 Instrumentation of the shield

Pressure and displacement of the hydraulic jacks and the deformations of the shield (using strain gages) are recorded in real time. Three sections of the shield are instrumented to measure the space between the Boom Clay wall and the shield. These measurements allow to determine the convergence occurring between the excavated front and the rear of the shield. Diameter measurements are performed in the unsupported zone between the rear of the shield and the last installed lining ring.

7.5 The realisation

The realisation of the connecting gallery at 223 m depth using wedge block system was really a challenge. Modelling has allowed to predict correctly the convergences between the excavated front and the lining and to choose the optimal geometry of the shield. No main problems were encountered during excavation. This has allowed to reach a progress rate about 3m/day. During the construction systematic observation of the excavated front and of the wall was performed (Figure 7.7). The fracturation scheme has been fully characterised.



Figure 7.7: Typical fracturation scheme observed at the excavated front

8 The in-situ measurements

Before and during the construction of the connecting gallery, up to its completion, the in situ measurements were recorded in real time.

The data obtained during excavation provide a good database for the understanding of the hydromechanical behaviour of Boom clay. The sensor readings of the CLIPEX instrumentation installed at the front of the Test Drift permit to characterize the hydromechanical behaviour ahead of the excavated gallery front. Especially the evolution of the pore water pressure obtained during excavation has provided us with very valuable information, because the pore pressure measurement in the Boom Clay is quite reliable and precise, and because it is very sensitive to mechanical variations (volumetric part) due to the strong hydro-mechanical coupling existing in the Boom Clay.

The displacement measurements in the CLIPEX instrumentation programme provide a valuable complementary database to characterise the hydromechanical behaviour even though their correct interpretation asks more effort.

In principle, the total pressure combined with the pore pressure measurements should permit to deduce the effective stress, which is the essential variable for establishing the constitutive law of the massif.

In addition, the convergence measured through the shield provides a good characterisation on the instantaneous behaviour of Boom Clay under decompression.

Finally, the instrumentation installed in the gallery lining, which allows to monitor the stress build up in the lining and the pressure exerted by the clay host rock, permits to evaluate the short and long term hydro-mechanical behaviour of Boom Clay and interaction between the lining and the host rock.

8.1 Measurement results

Here we will concentrate only on the measurement results during excavation to check their validity.

8.1.1 Measurements from the Test Drift Front

8.1.1.1 Pore waterpressure

8.1.1.1.1 Piezometer A2

Figure 8.1 shows the pressures of the A2 piezometer during the excavation of the connecting gallery. Figure 8.2 shows a detail of the suction pressures. Figure 8.3 shows the piezometer when uncovered at the excavation front.

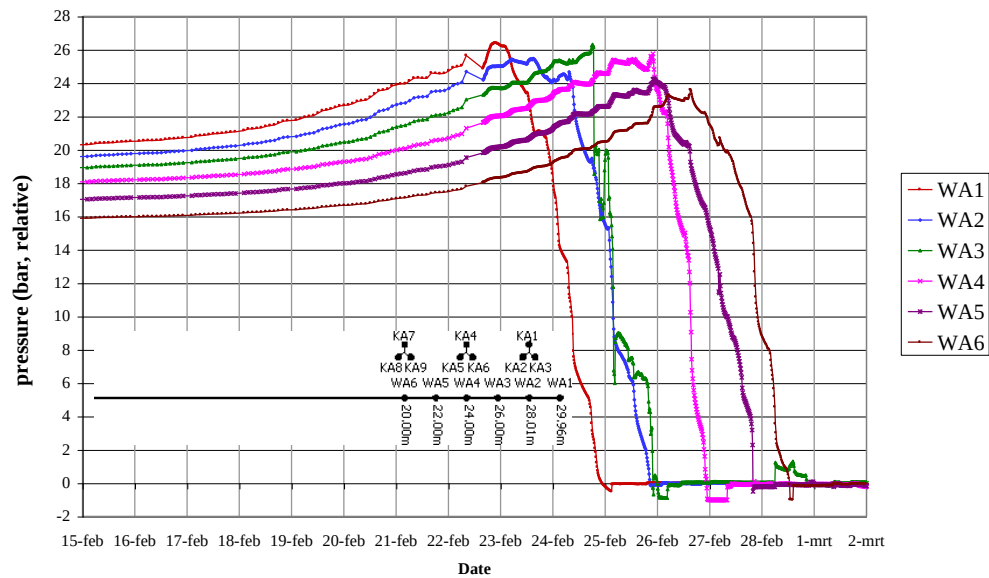


Figure 8.1: Piezometer A2 : evolution of the pore water pressure

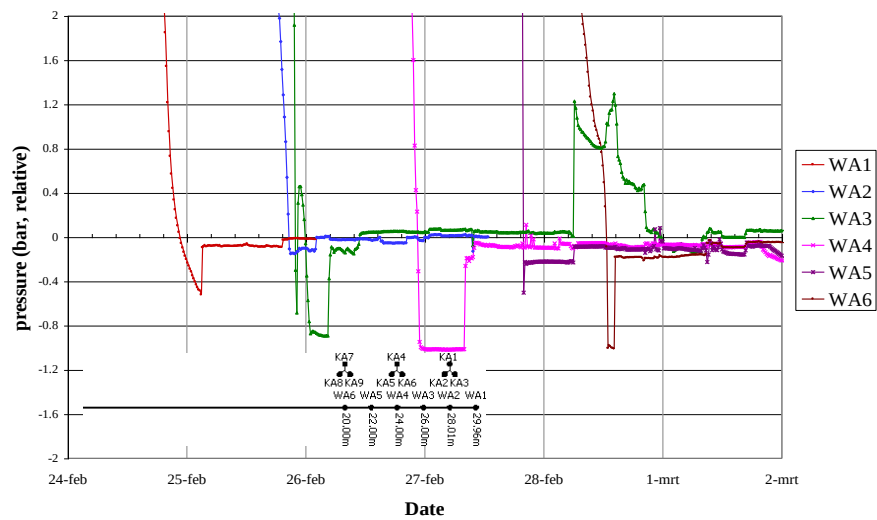


Figure 8.2: Piezometer A2 : detail of the suction pressures



Figure 8.3: piezometer A2 uncovered at the excavation front

8.1.1.1.2 Piezometer B2

Figure 8.4 shows the pressures of the B2 piezometer during the excavation. Figure 8.5 shows a detail of the suction pressures.

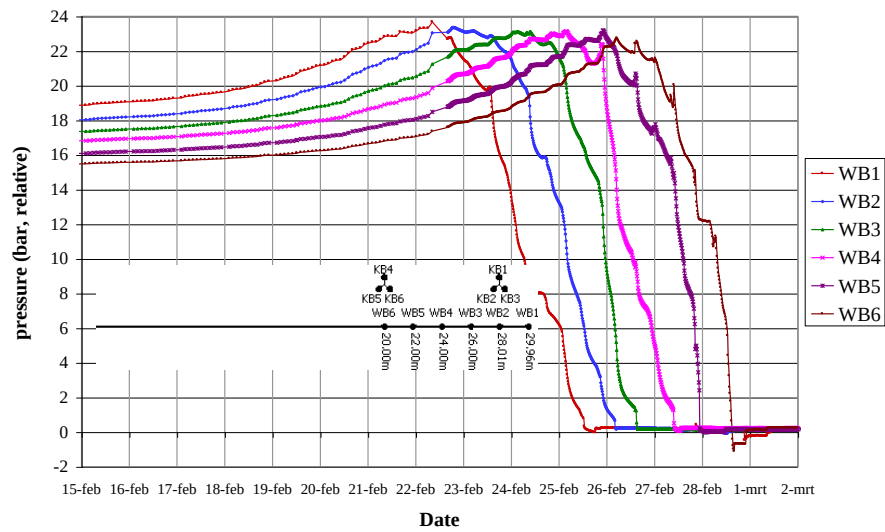


Figure 8.4: Piezometer B2: evolution of the pore water pressure

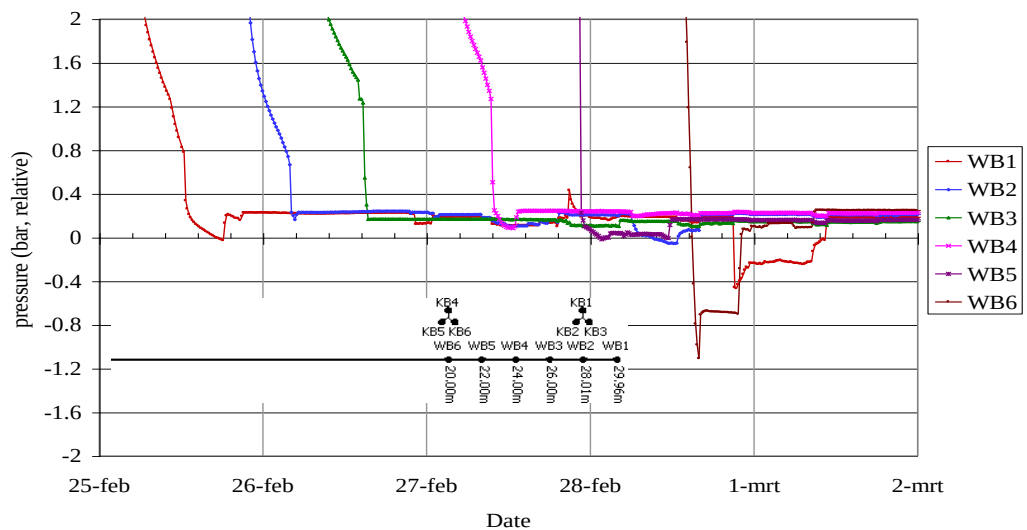


Figure 8.5: Piezometer B2 : detail of the suction pressures

8.1.1.1.3 Piezometer C2

Figure 8.6 shows the pore water pressures during the connecting gallery construction. The casing was cut on 6 March by the tunnelling machine.

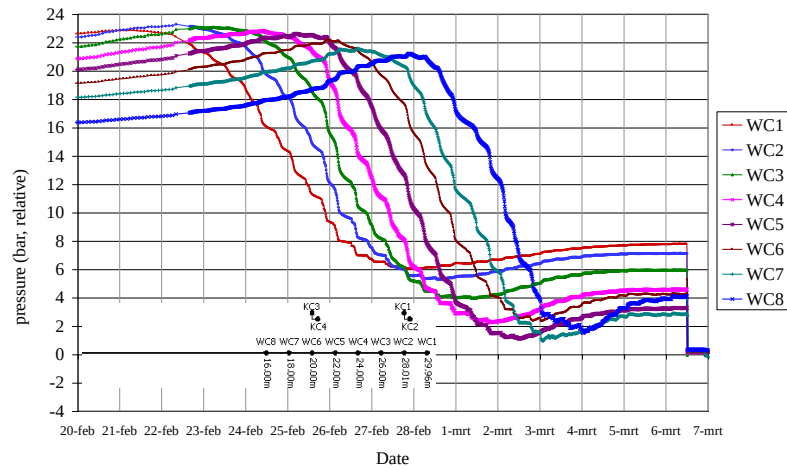


Figure 8.6 : Piezometer C2: evolution of the pore water pressure

8.1.1.1.4 Piezometer D2

Figure 8.7 shows the pore water pressures during the connecting gallery excavation. The casing was cut on 7 March by the tunnelling machine (Figure 8.8).

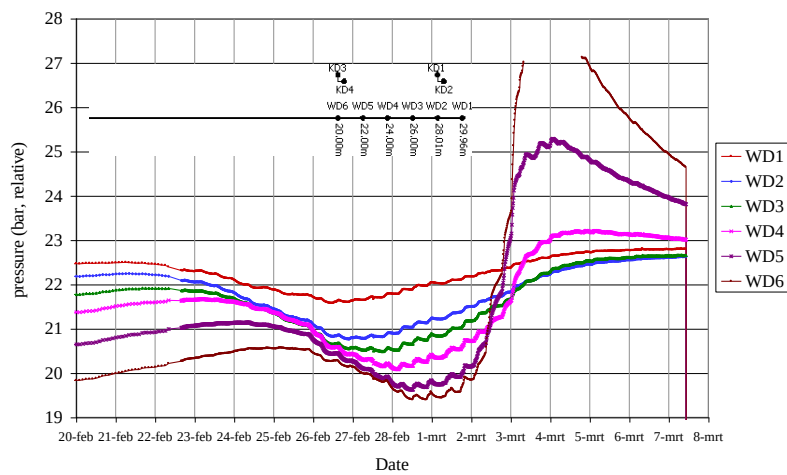


Figure 8.7 : Piezometer D2: evolution of the pore water pressure



Figure 8.8 : Cutting of the D2 piezometer at the excavation front

8.1.1.2 Total pressure measurements

The total pressure evolution measured from boreholes A2, B2, C2 and D2 during the connecting gallery excavation are given in Figure 8.9 to Figure 8.12. All total stress pressure transducers gave regular measuring evolution except KD1.

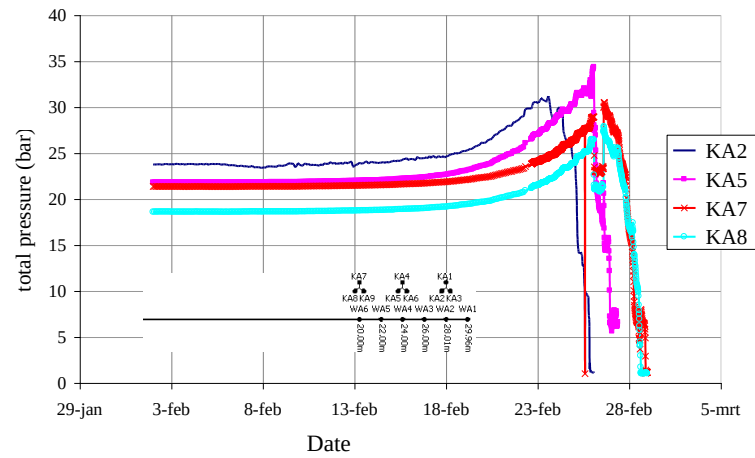


Figure 8.9: Borehole A2: evolution of the total pressure

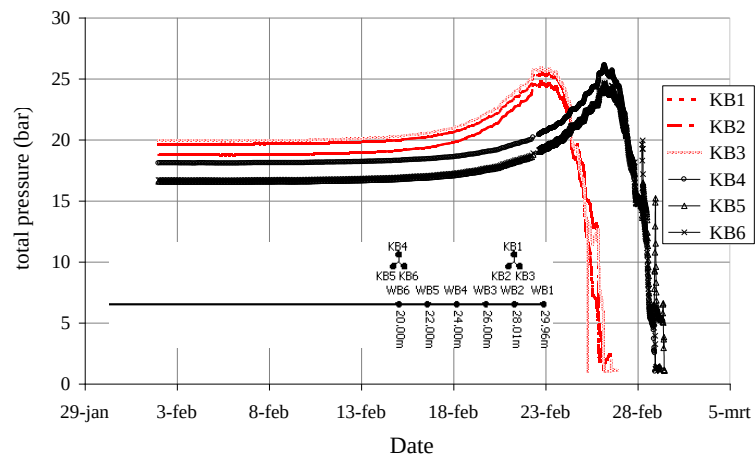


Figure 8.10: Borehole B2: evolution of the total pressure

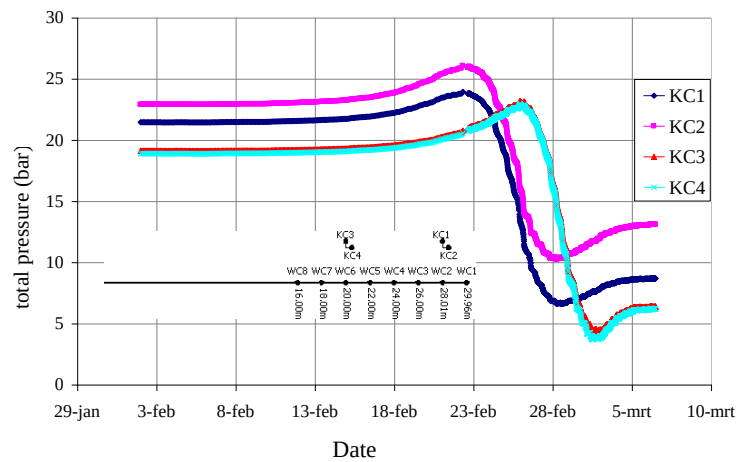


Figure 8.11: Borehole C2: evolution of the total pressure

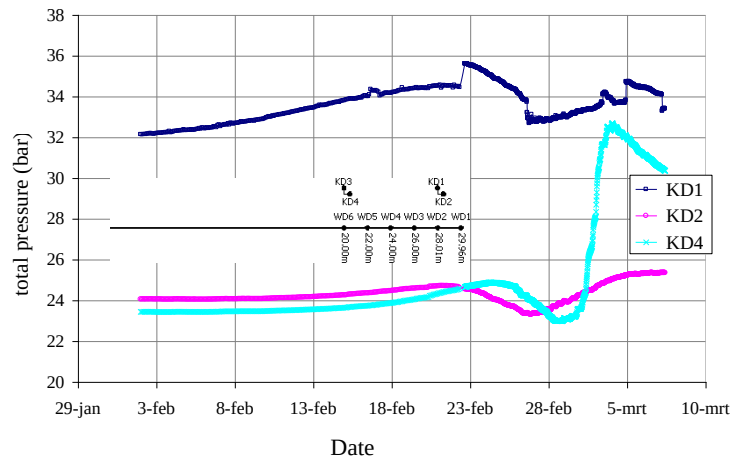


Figure 8.12: Borehole D2: evolution of the total pressure

8.1.1.3 Displacement measurements

8.1.1.3.1 Extensometer A1

Five of the six extensometer readings (Figure 8.13) show a simultaneous movement. This raises doubts on the independent movement of each anchor, and rather suggests that the different connecting rods behave as one piece, and are related to a point rather close to the test drift front (5-8 m before the test drift front). This could explain why e.g. EA1 (the deepest anchor) still shows a movement, even when the anchor itself yet has been removed at the excavation front.

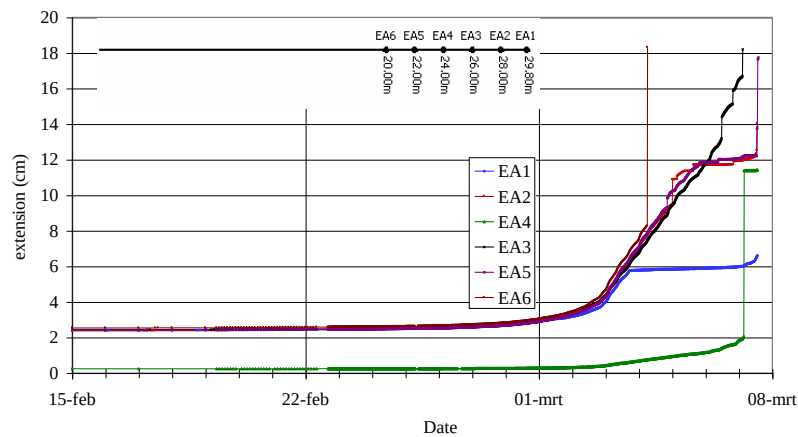


Figure 8.13: Extensometer A1: displacements as a function of time

8.1.1.3.2 Inclinometer B1

The inclinometer readings of B1 (Figure 8.14) show stable and clear movements; their interpretation requires however extra hypotheses concerning the reference points to be considered when integrating the individual movements of the chain segments.

During the dismantling, it's observed that the chain has been dragged in the horizontal direction, which may result in the bending of the chain. This adds the difficulty for the integration of the data. A further calibration work is necessary also when integrating the readings of each segment.

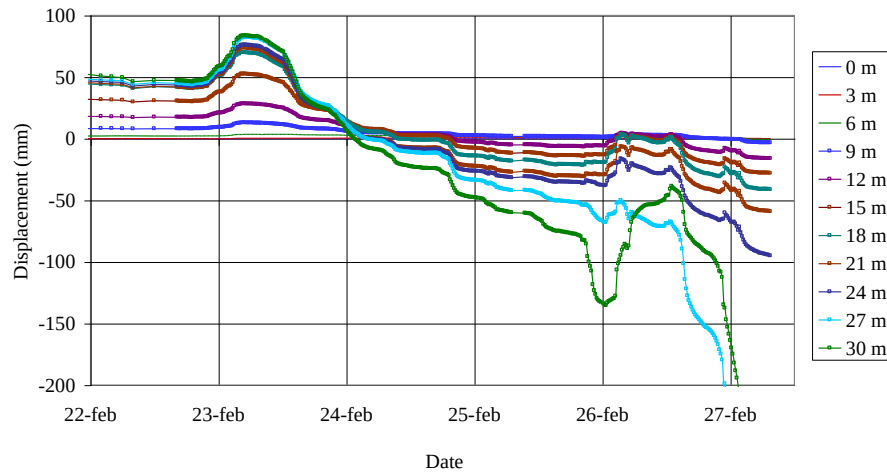


Figure 8.14: Inclinator B1: accumulated displacements in of the different points



Figure 8.15: Inclinator B1 : casing as seen from the tunnelling shield (left); the chain is partly recovered from the Test Drift side (right)

When the tunnelling shield met the inclinometer casing (during the day of 27 Feb 2002), the inclinometer chain was recovered from the Test Drift front (Figure 8.15). This could only be performed partially, as the shield had deformed yet the casing.

8.1.1.3.3 Inclinator C1

Many readings of inclinometer C1 are out of range (Figure 8.16). However, part of them (last 4 sensors IC6 to IC10) gave some meaningful readings. More efforts will be given in the future for integrating these measurements when finding out a method to fix the reference point.

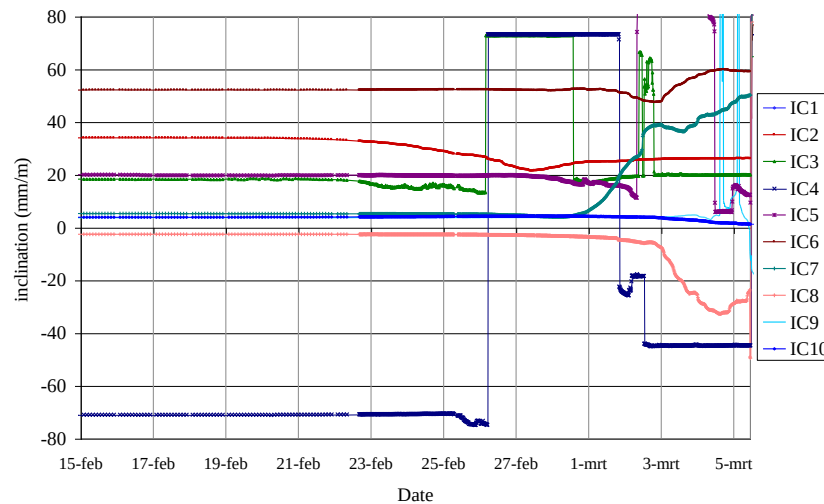


Figure 8.16 : Inclinomètre C1 : sensor data (incremental) in function of time

8.1.1.3.4 Deflectometer D1

All readings of Deflectometer D1 stay in the measuring range of the sensors as shown in Figure 8.17. We remark that the adjacent sensors gave regularly the opposite responses: for example, in comparing the evolution of DF7h to DF8h and DF6h, DF7h deviated in the opposite direction with respect to DF8h and DF6h. This led to suppose that there existed the relative bending between the beams inside chain, which yield the integration of these data extremely complicated due to the difficult for the reference point choice. Further calibration works are needed thus to interpret these data.

It is felt necessary to mention that, during restoring the deflectometer, one observed that the entire chain has been rotated, it implies that it is meaningless to decompose the deviation in a horizontal and vertical one. The directions of the two components depend so on the rotation angles of the entire chain. A correct integration of the readings should give however precise radial displacements at each sensor. The difficulties reside again in the choice of the reference point when integrating the individual movements of the chain segments. For this same reason, a further calibration work is necessary for the integration of readings to get the accumulate displacements taken place during the excavation.

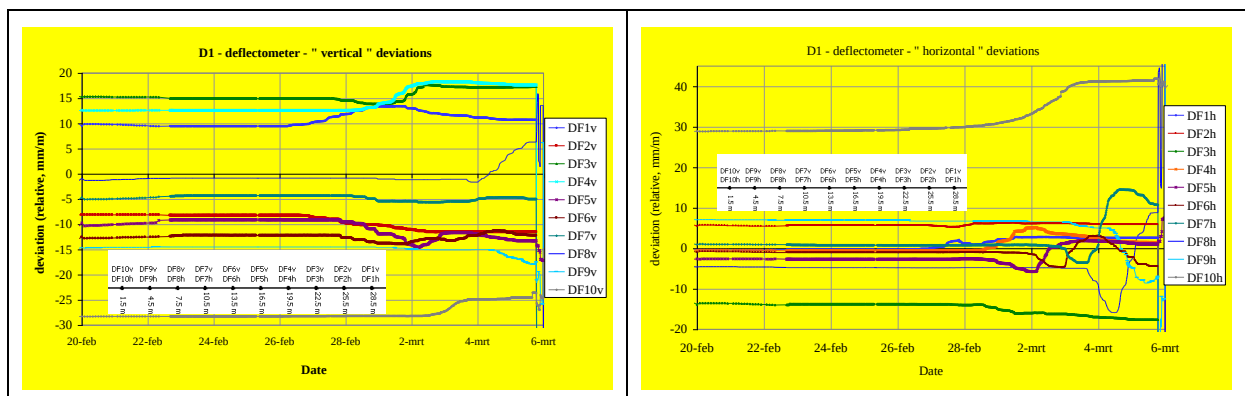


Figure 8.17 : Deflectometer D1 : relative deviations

8.1.2 Measurements from the second shaft

8.1.2.1 Pore waterpressure (Piezometer E2)

The evolution of the pressures since the installation is shown in Figure 8.18. We clearly observe the redrilling of the nearby borehole E1 (at some 1.5 m parallel to E2) in the beginning of March 2001, and the excavation of the mounting chamber in August 2001. Figure 8.19 shows the evolution of the pore pressure during the connecting gallery excavation. All sensors response clearly to the advancing excavation.

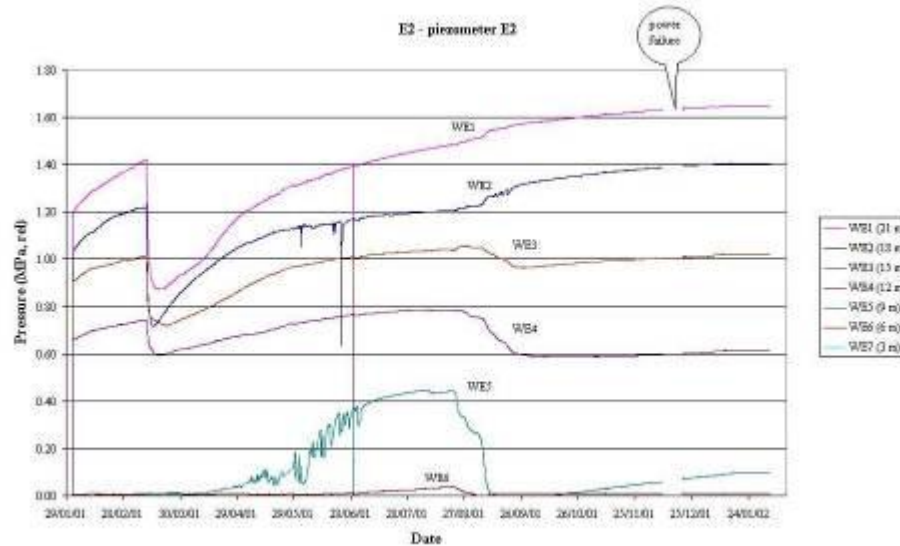


Figure 8.18: Piezometer E2, evolution of the pore water pressures prior to the excavation of the connecting gallery

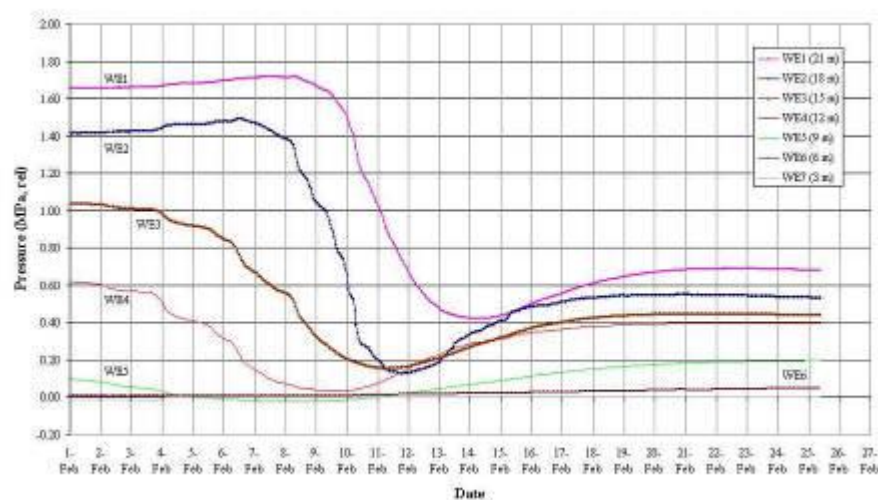


Figure 8.19: Piezometer E2: excavation response

8.1.2.2 Total stress on the lining of the 2nd shaft

The positions of different sections for the total pressure measurements have been shown in Figure 5.12.

8.1.2.2.1 Measurements on the sections AA and BB on the shaft

Totally, six sensors of total pressure are installed in the sections AA and BB (Figure 8.20). The total pressure built up on the lining are given in Figure 8.21

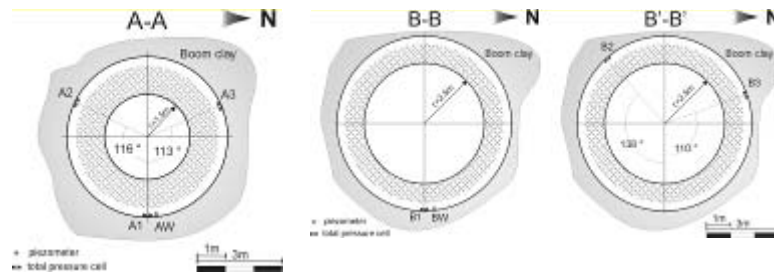


Figure 8.20 : Position of the total pressure sensors installed in sections AA and BB

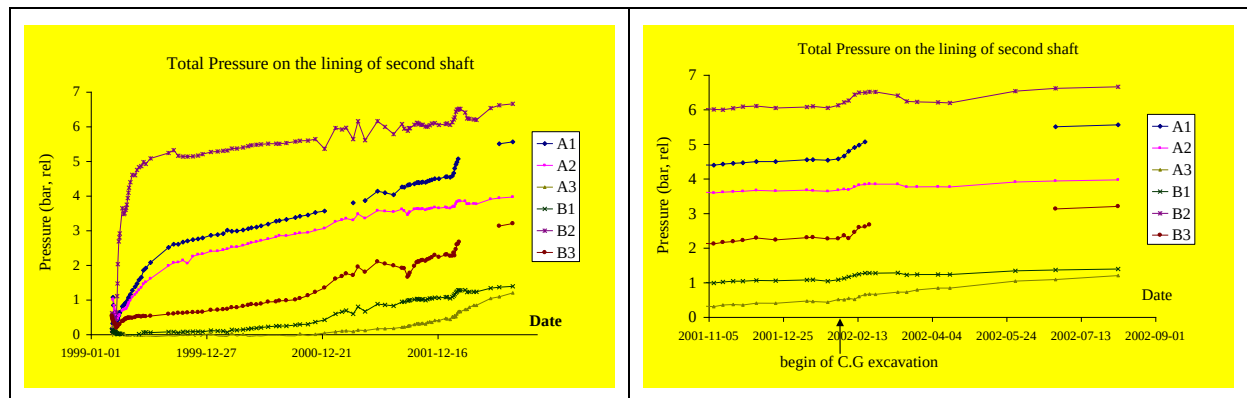


Figure 8.21 : Total pressure evolution on the sections AA and BB

8.1.2.2.2 Measurements on the section SS in the southern starting chamber

The position of the sensors in the section SS of south chamber is shown in Figure 8.22. The corresponding generated total stress is given in Figure 8.23.

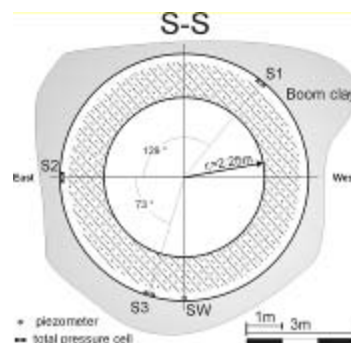


Figure 8.22 : Positions of total stress sensors on the section SS

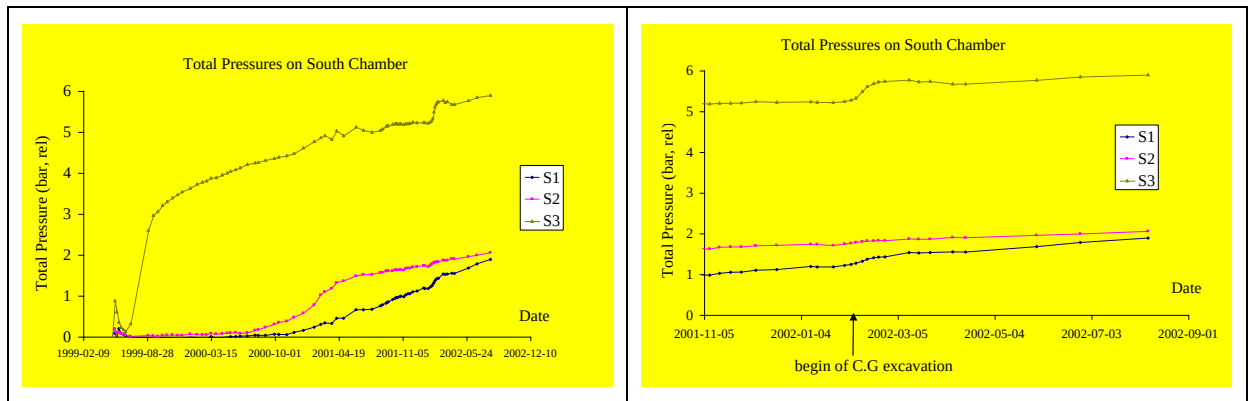


Figure 8.23 : Total pressure on the lining of the south chamber

8.1.2.2.3 Measurements on the section NN in the northern starting chamber

The position of the sensors and corresponding total pressures evolution are given below (see Figure 8.24 and Figure 8.25).

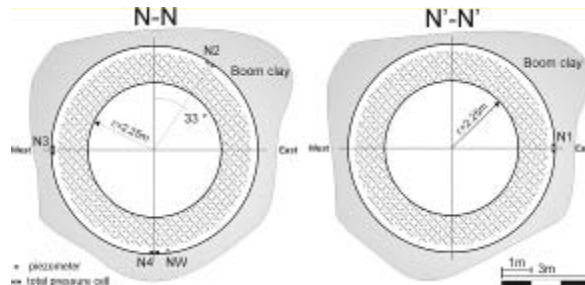


Figure 8.24 : Positions of total pressure sensors in north chamber

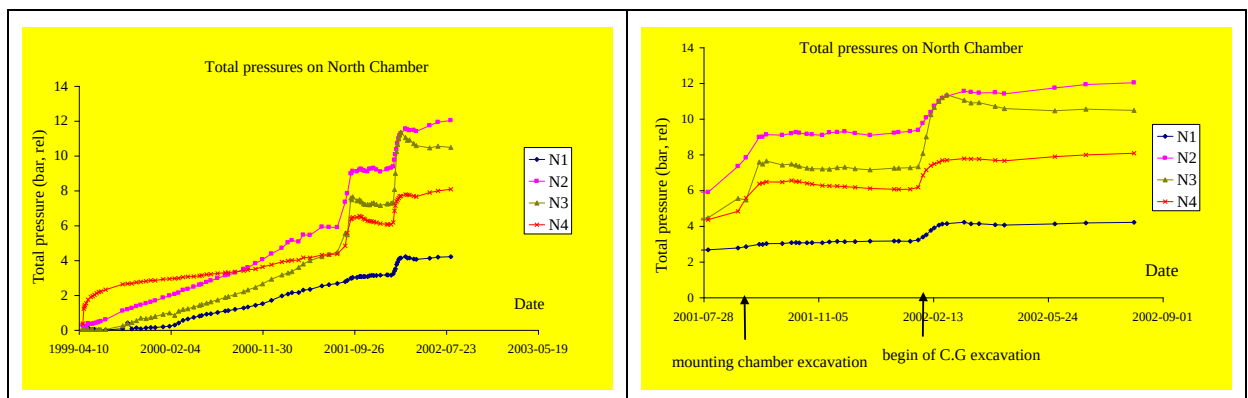


Figure 8.25 : Total pressure on the lining of the north chamber

8.1.2.3 Displacement measurements (Inclinometer E1)

Figure 8.26 shows the settlement profile along the inclinometer E1 measured at different moment. The advancing excavation of gallery can be visualised from it.

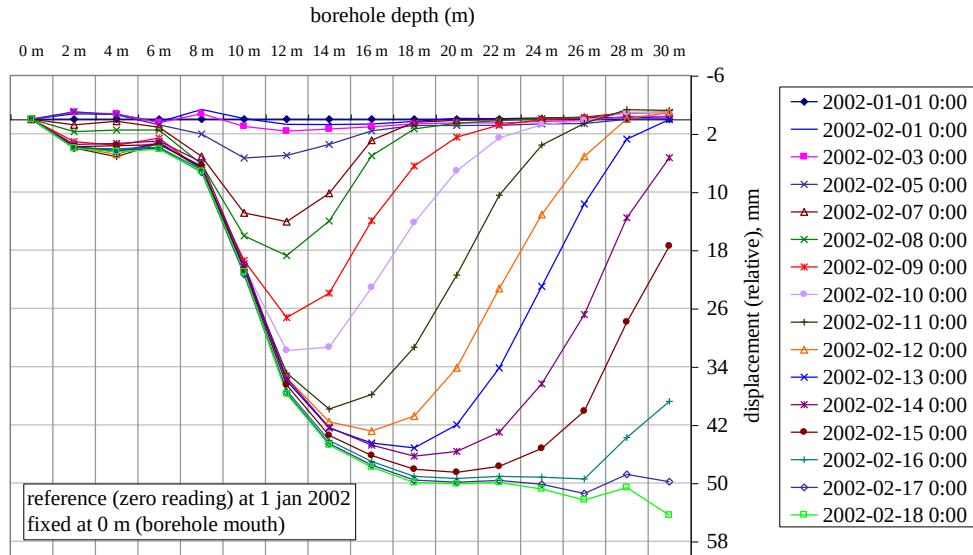


Figure 8.26: Settlement above the connecting gallery as recorded by inclinometer E1

8.1.3 Measurements in the gallery lining

We will focus on the measurements of ring 50 (Figure 8.27), as we have monitored this ring most frequently, especially immediately before and after the construction of the ring. The measurement configuration is the same for the three rings.



Figure 8.27 : Configuration of ring 50 (segment identities and sequence numbers)

8.1.3.1 Short-term readings just before, and just after the wedge-block installation:

Just before, and just after installation of the key segments, the sensors were read out. The results for all segments are given in Table 8.1. Stress calculations are performed based on a concrete modulus of 50 GPa (based on the test results realised by Laboratorium Magnel voor Betononderzoek of University of Gent).

Table 8.1: Strain and related stress in the segments due to the pushing of the key

Segment no.	Segment id.	STRAIN (*10 ⁻⁶)			STRESS (Mpa)		
		Intra	Extra	Mean	Intra	Extra	mean
1	1137	24	-9	8	1,21	-0,44	0,39
2	1019	14	3	9	0,71	0,15	0,43
3	891	15	19	17	0,77	0,94	0,85
4	1059	6	15	11	0,30	0,76	0,54
5	1013	13	17	15	0,65	0,86	0,76
6	992	29	-3	13	1,46	-0,14	0,66
7	836	16	13	14	0,80	0,65	0,73
8	1064	5	17	11	0,22	0,86	0,55
9	1083	25	11	18	1,24	0,54	0,89
10	916	24	4	14	1,23	0,21	0,71

The immediate effect of pushing in the keys is apparently limited – the stress is lower than 1 MPa. More important is however the difference between intrados and extrados, especially at the top (no. 1) and bottom (no. 6) segments, where the intrados shows a high compression, compared with the extension at the extrados. Segments no. 4 and 8 show an inverse behaviour. Looking more in detail at the different levels, we also notice that the wedge-shaped key segment apparently puts more stress at the excavation side of the segment (corresponding with the upper level), as can be seen from Table 8.2 (top segment) and Table 8.3 (bottom segment).

Table 8.2: Individual readings of segment 1137 (top): strain before and after pushing of key

	Intrados left	Intrados right	Extrados left	Extrados right
UPPER	48 (11)	67 (14)	2 (21)	5 (24)
MIDDLE	26 (12)	7 (15)	-2 (22)	-19 (25)
DOWN	4 (13)	-6 (16)	-20 (23)	-20 (26)

Table 8.3: Individual readings of segment 992 (bottom): strain before and after pushing of key

	Intrados left	Intrados right	Extrados left	Extrados right
UPPER	54 (11)	32 (14)	-1 (21)	-2 (24)
MIDDLE	24 (12)	29 (15)	-6 (22)	-6 (25)
DOWN	10 (13)	26 (16)	-9 (23)	-9 (26)

8.1.3.2 Long term external pressures build up

The outside pressures exerted on the lining are calculated based on the mean stress inside each segment assuming a thin wall approximation. So obtained external pressures are given in Figure 8.28. Figure 8.29 further shows that the intrados – extrados difference remains throughout the pressure build up.

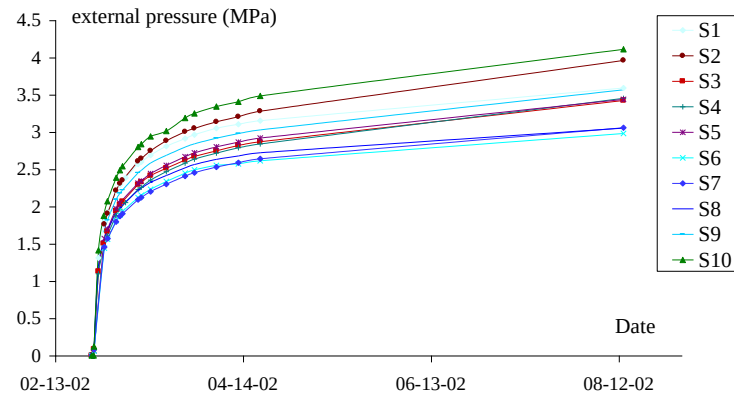


Figure 8.28 : Pressure build up on ring 50 (thin wall approximation)

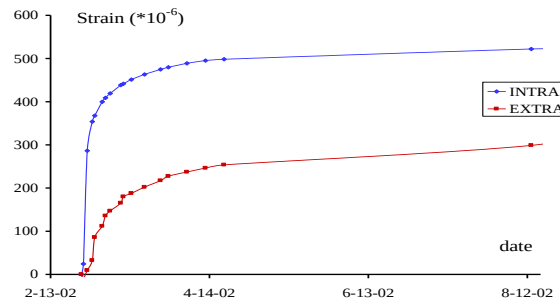


Figure 8.29 : Intrados and extrados evolution of strain (ring 50, segment 1137)

8.1.4 Convergence measured from the shield

Numerous measurements on the convergence from the shield have been taken at different moment. These measurements give information on the instantaneous mechanical behavior of the boom clay. Typical convergence profile along the shield is shown in Figure 8.30, which indicates that, given its current design, 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

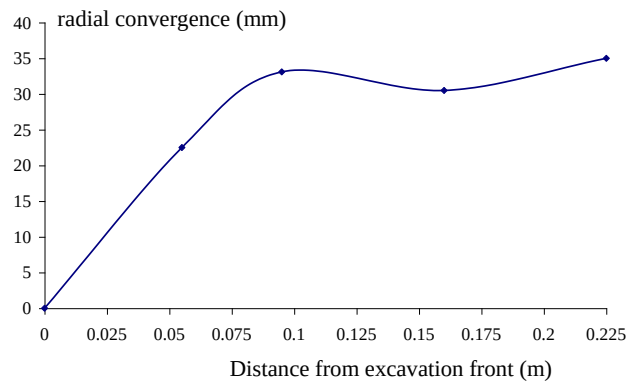


Figure 8.30 : Instantaneous convergence of the gallery wall

8.1.5 Conclusions

Generally, all sensors response clearly to the excavation of the gallery.

All piezometers gave very faithful pore pressure measurements. The transducers of total stress gave also regular recordings. The extensometer A1 gave reliable axial displacements related to one point close to the test drift front (5-8 m before the test drift front). The inclinometer B1, C1 and deflectometer D1 provided also meaningful measurements, their processing asks however further calibration work especially due to the difficulty for the choice of the reference point. The instrumentation installed on the 2nd shaft lining and gallery lining delivered valuable measurements.

8.2 Interpretation of the in situ measurements

The recordings obtained from CLIPEX instrumentation during C.G excavation will be analysed in the light of hydromechanical behavior of Boom clay under decompression due to the excavation.

8.2.1 Pore water pressure

8.2.1.1 Pore water pressure measured from Test Drift

The pore pressures evolutions, which are expressed in function of the distance between the front position and the sensor, are given in Figure 8.31

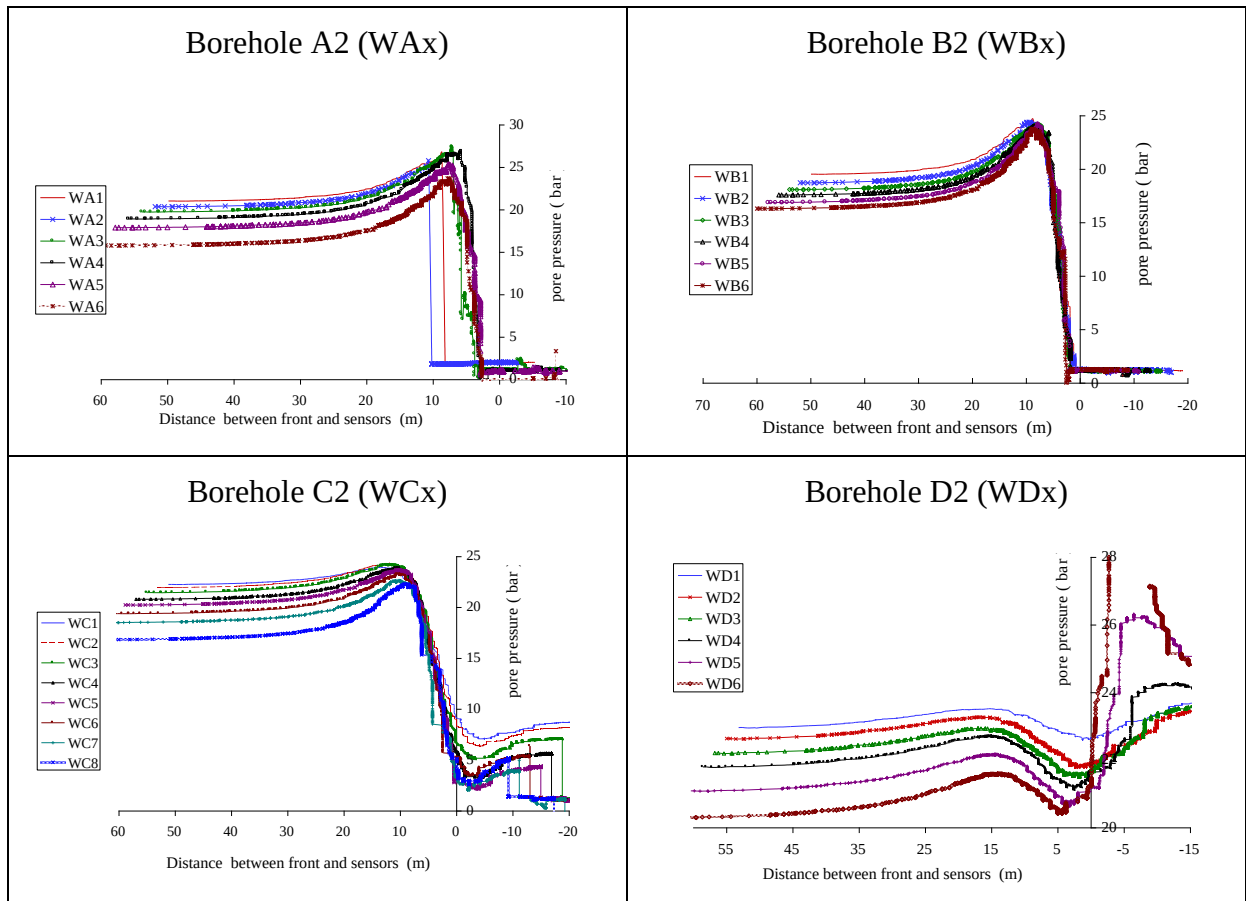


Figure 8.31 : Pore pressure evolution in function of the excavation front position

The following important aspects can be seen from these measurements:

- Important gradients before C.G. excavation

The CLIPEX measurements show clearly that the pore pressure before connecting gallery excavation presents very higher gradient at every borehole (big difference between different sensors, ex. the pressure in WA1 is quite different from that in WA6, there is about 5.5 bar difference). It is certainly the consequence of the Test Drift excavation (drop down due to the decompression). And these big gradients persist since then.

- Regular evolution with advancing front

With the coming connecting gallery front, all sensors record same evolution: increasing progressively until a peak then dropping down when the front approaches very closely.

For the sensors faraway (C2 and D2), the pore pressure perked up progressively after dropping down. Especially for the sensors of the borehole D2, which was installed more or less horizontally in the plane of the gallery axis, the pore water pressures have even outstripped the peak values. It's not the case for the sensors in the borehole C2, which was installed in a vertical plane.

The sensors in the borehole A2 and part of B2 are lost when the front crosses them, since they were installed almost along the axis of the gallery.

The increase of the pore pressure corresponds to the undrained contractant plastic behavior of the material.

The drop phenomenon is linked certainly to the decompression of the massif, and also to the fracturing around the front.

The perking up of the pore pressure is probably related to the following phenomena:

- The decompression of the massif induces a drop of the pore pressure and so creates an very strong pore pressure gradient, consequently, the local flow will takes place to re-equilibrium the pore pressure field. This recovers the pore pressure.
 - When the convergence of the gallery wall reaches to the equilibrium with the lining support, it can induce the recompression of the clay and pore pressure regains his balance.
- Each sensor presents a Peak pressure which depends on the sensor position

Each sensor presents a peak pressure, but the moment reaching to the peak depends largely on the absolute distance between the front and the sensor. One likely abnormality is observed as shown in Figure 8.32a: the farther the sensor is, the earlier the peak appears (it means the distance ahead of front where the peak appears d_{peak} is larger). The relative increase of pore pressure (peak pressure subtracts the value before excavation) depends equally on the position of the sensor (Figure 8.32b) and ranges from 8 bar (for the sensors closer to the gallery) to 1 bar (for the sensors faraway).

The fact that the peak appears earlier in the far sensors needs however further study in combined with the displacements and stress measurements. It may due to the influence of the Test Drift. The perturbation of the stress field due to the Test Drift may provoke this observed abnormality. Because the initial stress state influences certainly the effective stress paths in the massif, and so the variation of the pore pressure.

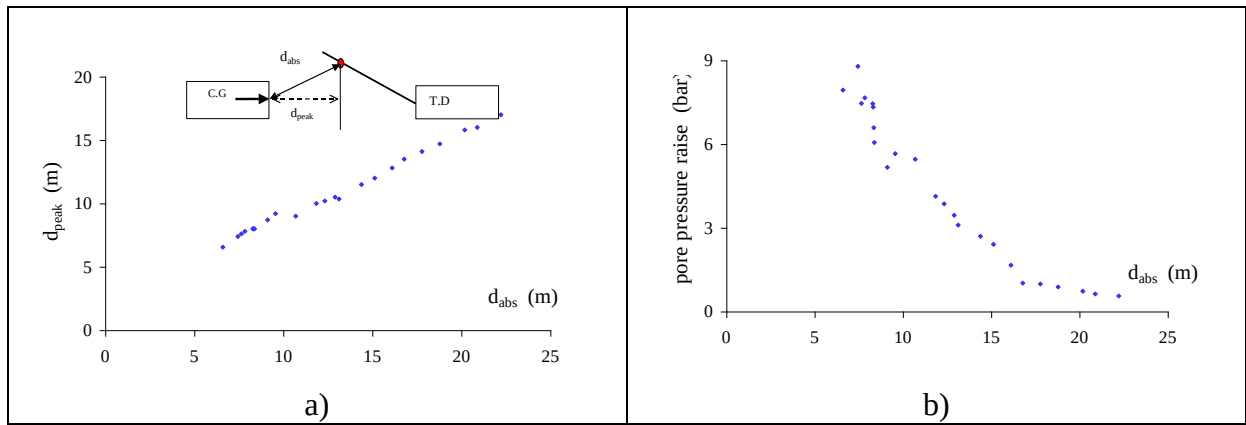


Figure 8.32 : Peak properties

- Pore pressure drop evidence due to the decompression

The pore pressure drop (difference between the peak value and the minimal value recorded after peak) ranges from 1 bar to 25 bar depending to the position of the sensors as shown in Figure 8.33, where the drop for each sensor are redrawn in function of the absolute distance between the front and the sensor.

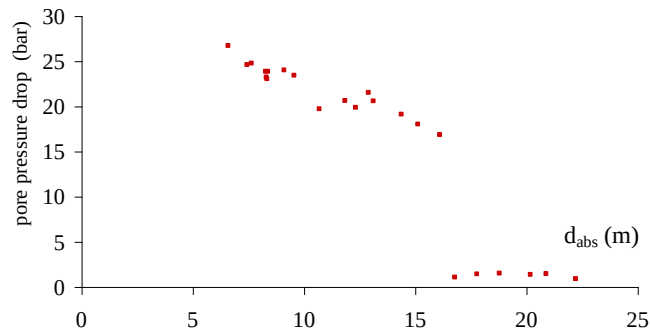


Figure 8.33 : Pore pressure drop after peak

- Very extended Hydraulic Disturbed Zone (HDZ)

It's interesting to note that all sensors react almost immediately after the start of the excavation. Two detail plots are given in Figure 8.34 from where we remark that all sensors begin to increase when the front is still 60 m before them. It implies that the hydraulic disturbed zone due to the excavation extends at least 60 m.

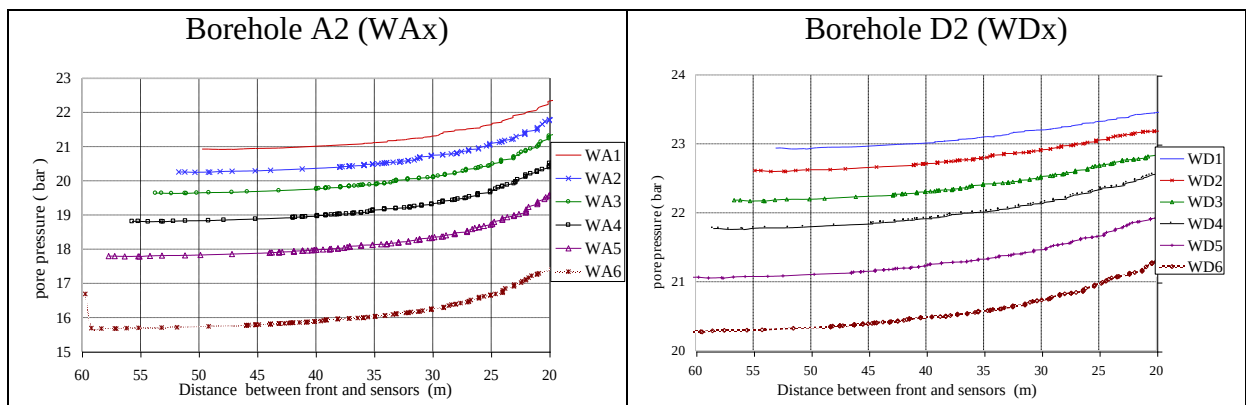


Figure 8.34 : Far field reactions to excavation

A first effort has been made trying to bring some explications to this phenomena (Barnichon and Volckaert, 2000). Two following factors have certainly effects:

- The effect of the fracturing

The fracturing due to the excavation has been observed. The fracture zone around the gallery excavated can be seen as an apparent increase of the excavated radius. As a result, the zone influenced by excavation extends.

- The viscosity of skeleton

The viscosity of skeleton of the clay massif may be responsible for part of this far field behavior. A preliminary calculation with an elasto-visco-plastic law has been realised by Barnichon and Volckaert (2000), which shows effectively the viscosity of the Boom clay may plan a role in the far field observation.

The parametric study made by G3S using Rousset's viscoplastic model gave also a support for this hypothesis (G3S, 2002).

- Suction generation – fractures opening evidence

The suction has been registered in the boreholes A2 and B2. Figure 8.35 gives the detail on the suction generation with the coming gallery front.

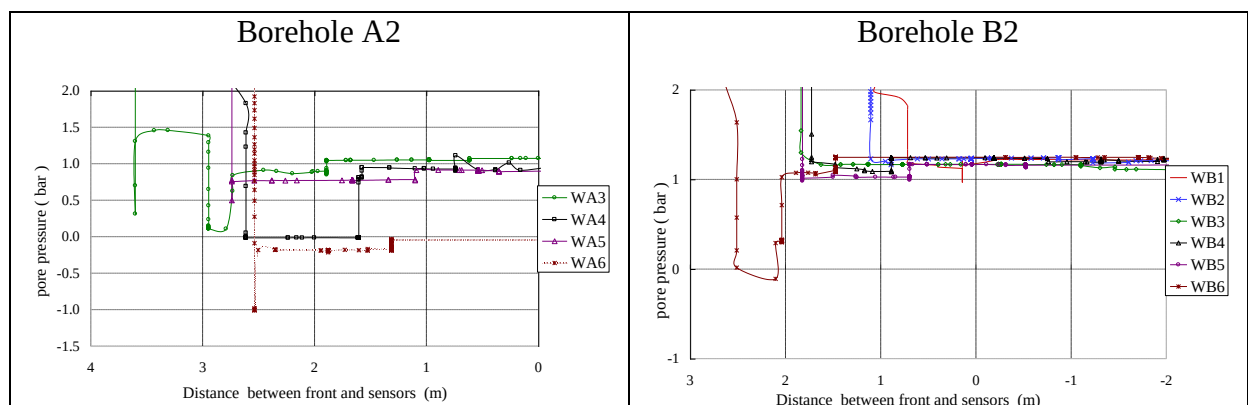


Figure 8.35 : Suction generation

We remark that several sensors have recorded the suction before 2 –3 meters ahead of front. An abrupt increase of the suction up to about 1 bar, which corresponds to the atmosphere pressure, has also observed. The same phenomenon has been observed already in the RESEAL in-situ shaft seal test (see Volckaert et al. 2000). This sudden re-equilibrium of water pressure with atmosphere corresponds to the opening of the fractures in the clay massif.

- Influence of the excavation phase

From Figure 8.36 below, it can be seen that the pore pressure continues to vary during the installation of the lining but with lower magnitude. This phenomenon is related to two effects:

- Time evolution due to water flow which may be non-negligible as the massif is in the highly disturbed state and the permeability may increase locally.

- Effect of viscosity of the clay massif may contribute also to the variation of the pore pressure.

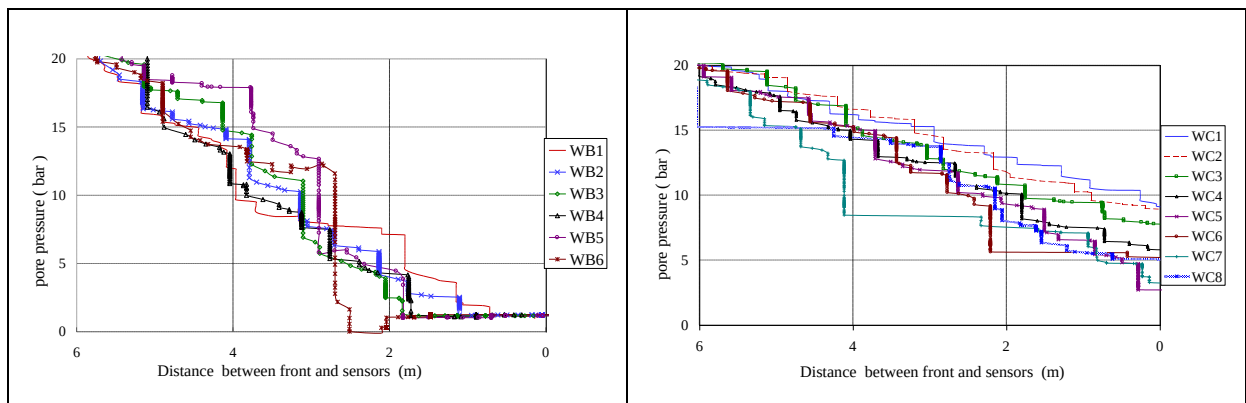


Figure 8.36 : Influence of the excavation phases

8.2.1.2 Pore water pressure measured from 2nd shaft

The pore water pressure evolution by the pizeometer E2 installed from 2nd shaft, paralleled to the gallery axis and 5 m above this later is given in Figure 8.37a.

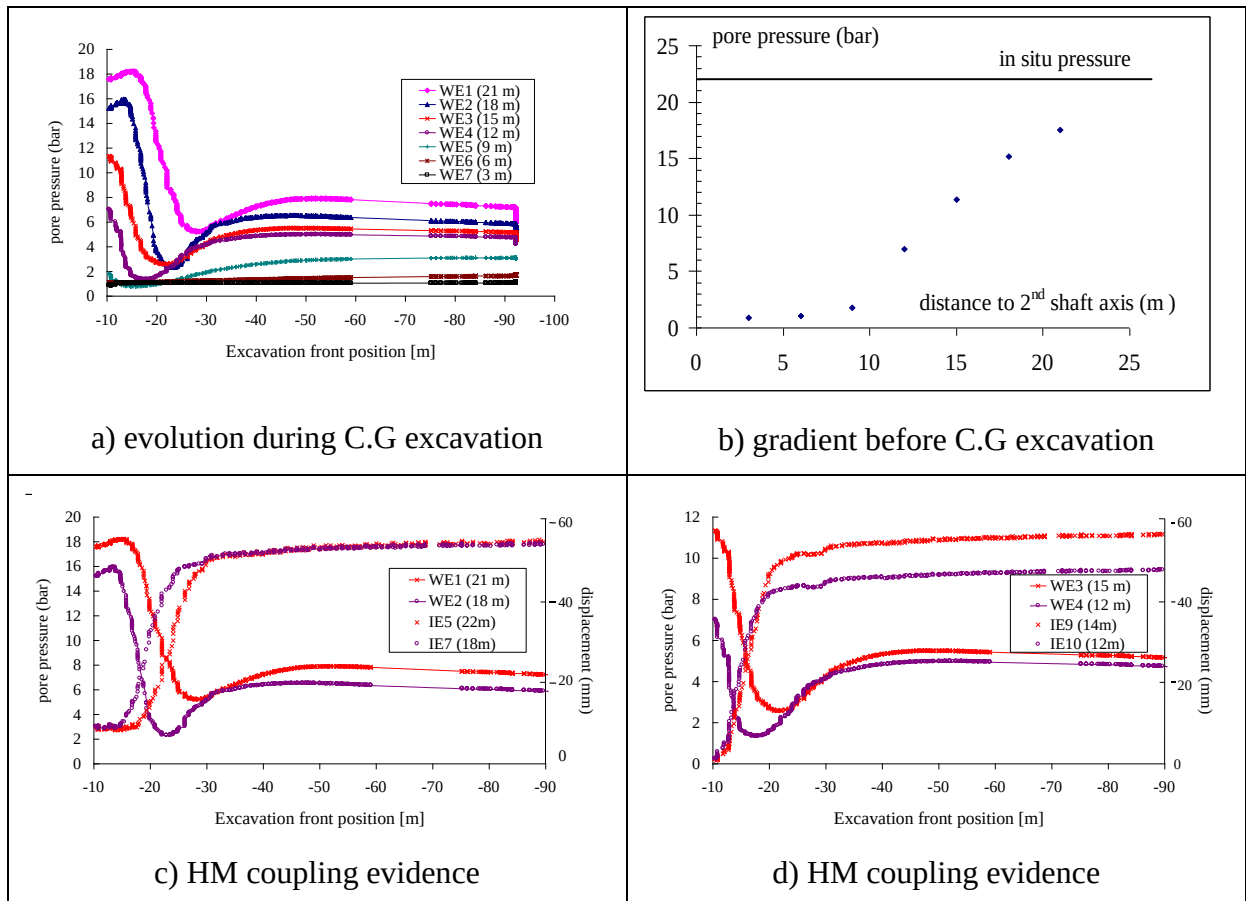


Figure 8.37 : Pore water pressure evolution measured from 2nd shaft (borehole E2)

Following aspects can be noticed:

- Influence of the excavation of the 2nd shaft

The influence of the excavation of the 2nd shaft is reflected by the gradient of the pore pressure before C.G excavation as shown in Figure 8.37b from where we observe an extensive HDZ due to the 2nd shaft excavation (at least much than 25 m).

- The hydro mechanical coupling evidence

The hydro mechanical coupling can be visualized via Figure 8.37c and d thanks to the faithful measurement of inclinometer E1 (see §8.2.3). The close relationship between the deformation and pore pressure is clearly put in evidence: large deformation (IEx) induces strong pore pressure variation (WEx). WEx indicates the pore water pressure measured from piezometer E2. IEx represents the displacement measured from the inclinometer E1, which is paired to the borehole E2.

- Very big drop of the pore pressure

The drop down at the deepest sensor reaches to about 15 bar.

- Slight recovering of the pore pressure due to the lining installation

The pore pressure seems to tend to increase after dropping down due to the re-equilibrium between the massif and the lining.

- Opening fractures evidence

We observe that the pore pressure measured by the sensor WE7 (3 m deep from the 2nd shaft axis) stays at the atmospheric pressure during all excavation procedure, this implies that the fractures created by the 2nd shaft excavation stay opened. This is in agreement with the extent of the oxidated zone around the shaft observed during the excavation.

8.2.2 Total stresses measured from Test Drift and its relation with pore pressure

Until connecting gallery excavation, the Kulite sensor (total stress) measured in fact the pore pressure. To verify this aspect, the Kulite recordings ("total stress ") obtained during the C.G excavation are putted together with the corresponding Druck (piezometer) recordings (pore water pressure) as shown in Figure 8.38. As already observed, the " total stress" curves still run parallel to the pore water pressure curves. The given " effective stresses " are extreme feeble even if it's not zero.

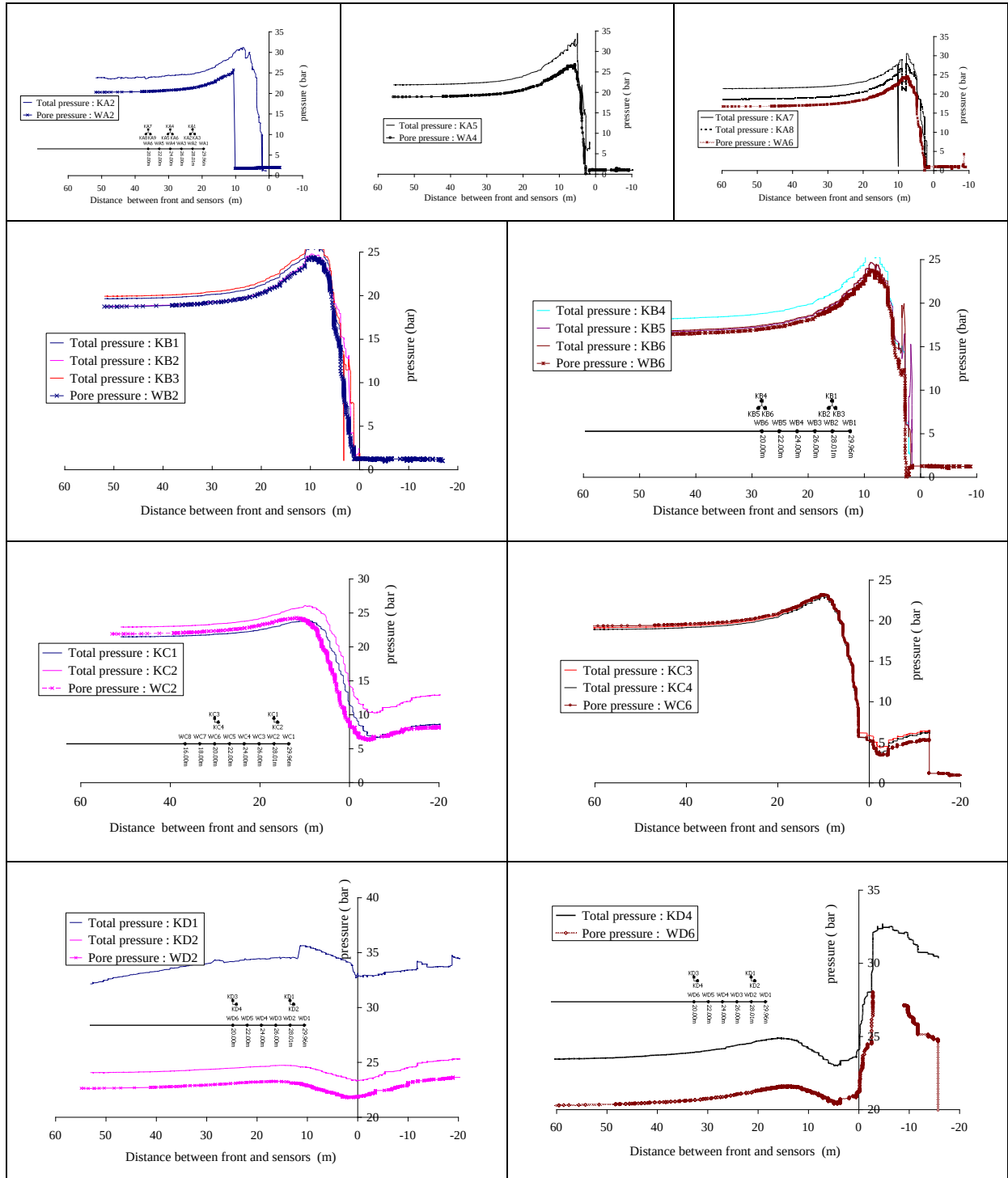


Figure 8.38 : Parallel evolution of " total stress" and pore pressure

The reason why the Kulite transducer measures nearly same things as the Druck sensor does is still an open question. The previous test of pore water pressure drop at filter WC2 supposed that the effective pressure is close to zero at the borehole/casing boundary. But, it couldn't conclude that the effective stress in the massif is zero. In reality, the effective stress can't be zero.

Consequently, the technique to measure the total stress remains a difficult measurement, since the borehole drilling process disturbs the sensors. Non interpretations of these measurements are therefore realised in the following text.

8.2.3 Displacements

8.2.3.1 Displacements measured from Test Drift

The displacements evolutions are redrawn in function of the front position as shown in Figure 8.39 below. Noting that the extensometer in the borehole A1 measures effectively the axial displacements along the axis of gallery. The measurements obtained by the inclinometer B1 give the displacements in two directions: vertical one and axial one.

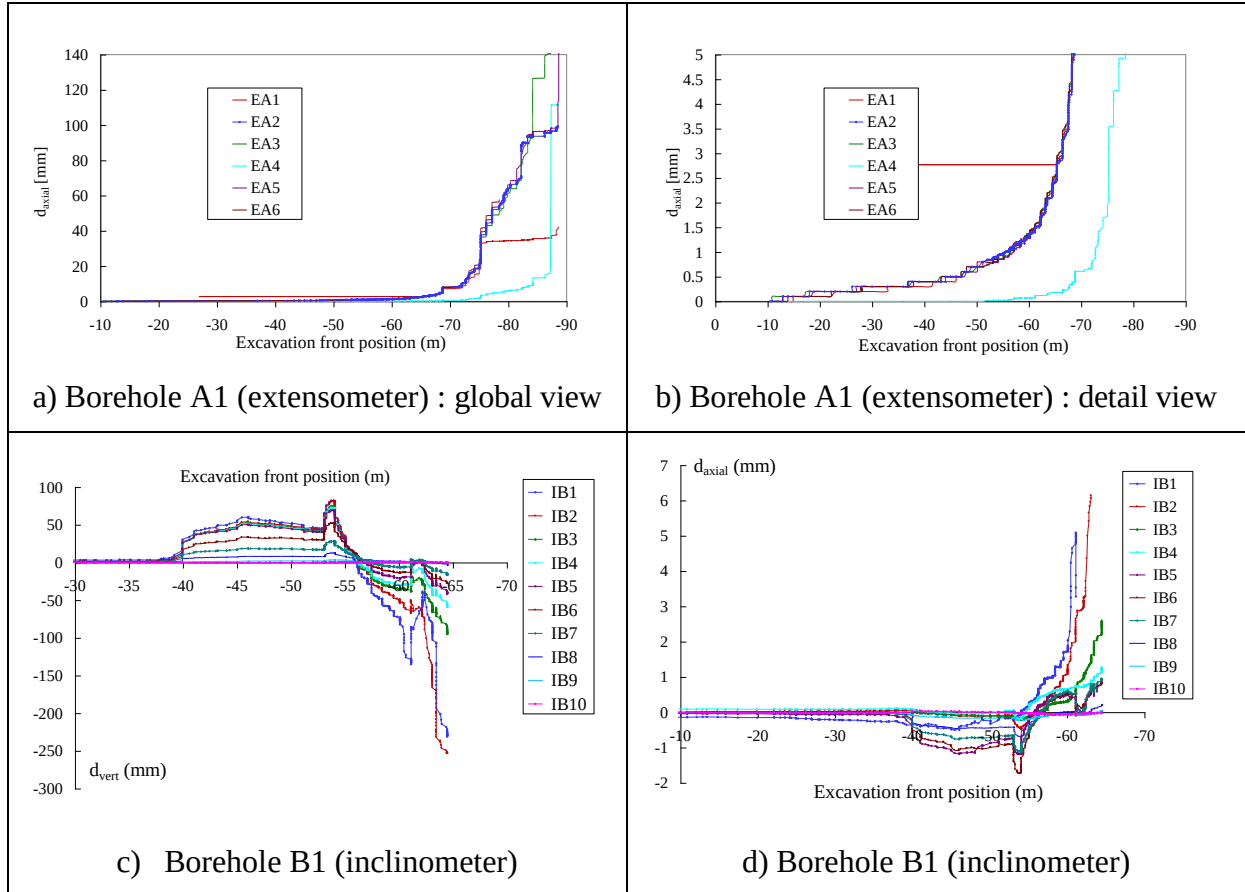


Figure 8.39 : Displacements evolution during C.G. excavation (from Test drift)

Although the interpretation of these measurements is difficult, some precious information can be gained nevertheless:

8.2.3.1.1 Extensometer A1

- The axial displacement along the gallery axis is about 120-140 mm

The obtained recordings are consistent with the hypotheses that all sensors were attached together and the signals received by all sensors were transferred to one point: the disk installed about 5 m ahead of the T.D. front. All sensors gave instantaneous recordings and terminated the registration when the front reaches about 5 meter before the T.D front. A final axial displacement about 120 -140 mm was recorded (see global view in Figure 8.39a).

- Very extended EDZ of Boom clay :

The detail view (Figure 8.39b) shows clearly the immediate reaction of sensors: the sensors began to react as soon as the excavation debuted. It means that the EDZ extends at least about 75 meters ahead of front (taking into account that the signals receiving point is situated about 5 m ahead of T.D. front as explained above).

- Lining installation effect

From the detail view, we observe that the lining installation effect is clearly echoed by the sensors: all curves present a saw-toothed pattern.

Although the extensometer A1 did not give the information for each expected position, it gave however without doubt faithful measurements for one point situated on the gallery axis. This is quite important to characterize the hydro mechanical behaviour of Boom clay.

8.2.3.1.2 Inclinator B1

The inclinometer B1 got actually some meaningful recordings. But the data management is not evident due to difficulty for fixing a reference point and the bending of the chain as mentioned above. The interpretation is thus only qualitative.

It's very hard to explain the earliest rebounding of all sensors. Is it a real phenomenon? Is it just a consequence of the movement of the reference point chosen actually that induces a false data processing? Are there some consequences due to the chain bending? Further calibration work of the sensors is needed to get explanation of these recordings.

Anyway, the downward displacement tendency with the coming front was clearly recorded as shown by Figure 8.39c. The maximum relative downward displacements as high as 200 –300 mm were registered in the deepest sensors IB1 and IB2. Nevertheless, the axial component of displacement seems very feeble comparing to the recorded axial displacements by the extensometer A1. The maximum one recorded by IB1 reaches only to about 8 mm (Figure 8.39d). This was probably the consequence of the bending of chain, which could modify the relative angle between the beams and thus lead to a false calculation of the axial component of displacement.

8.2.3.1.3 Inclinator C1 and Deflectometer D1

As the most of recordings of inclinometer C1 are out of measuring range (± 50 mm/m) (see Figure 8.16) and the data processing is not yet accomplished. The interpretation is therefore on standby. The interpretation of the data for the deflectometer D1 (Figure 8.17) is on standby too due to the ultra delicate data integration, to which some calibration works for the chain performance are required.

8.2.3.2 Displacements measured from 2nd shaft (Inclinometer E1)

The inclinometer E1 installed from the 2nd shaft delivers the settlement at 5 m above the gallery (Figure 8.40). We note that:

- 60 mm of vertical displacement recorded above 5m the axis of connecting gallery

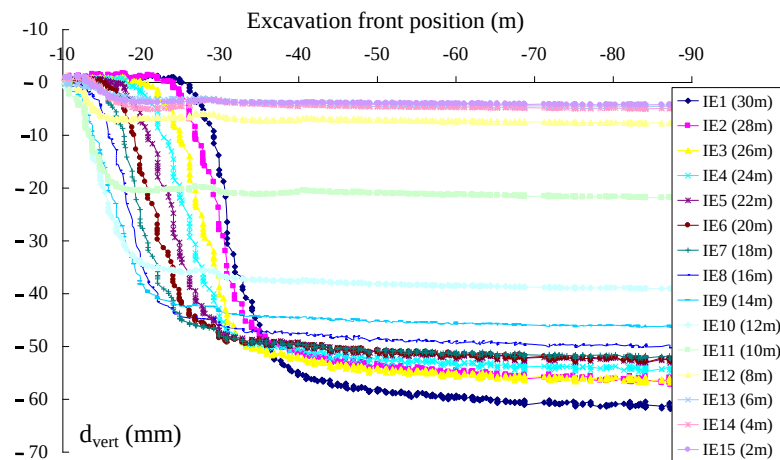
The deepest sensor IE1 (30 m deep from 2nd shaft) recorded a vertical displacement as high as 60 mm above 5m the axis of connecting gallery. This value seems to be realistic taking into account that the total radial displacement on the excavated wall is about 90 mm.

- Influence of the excavation phase

The saw-toothed pattern of the curves is again marked. We note that the continual evolution of the displacement during the lining installation.

- Displacement development slows down after the front passing by the sensor

The displacement generation tends to slow down after the excavation front passes by the sensors. The rapid lining installation limited largely the short-term disturbance of the massif.



Borehole E1 (inclinometer)

Figure 8.40: Displacements evolution during C.G. excavation (from 2nd shaft)

8.2.4 Total stress on the linings

8.2.4.1 Total stress on the lining of the 2nd shaft

The total pressures built up on the lining of the 2nd shaft were given in Figure 8.21 (sections AA and BB), Figure 8.23 (section SS in the southern starting chamber) and Figure 8.25 (section NN in the northern starting chamber) above. We remark that:

- Immediate build up of the total pressure following the sensor installation

As soon as the installation of the sensors, the total pressure built up rapidly, and continued to increase afterward.

- Marked excavation phases

Each excavation phase was marked by the total pressure variation. At the end of August 2001, the mounting chamber excavation provoked a sudden increase of the total pressure (Figure 8.25). The step changes observed at the beginning of February of 2002 marked out the beginning of the excavation of the connecting gallery. It is noted that the effect of the connecting gallery excavation was more pronounced in the north chamber (Figure 8.25)

than in the south chamber (Figure 8.23). It is an evident fact since the connecting gallery extends from the north chamber.

- relatively feeble pressure generated

We note that the pressures generated on the lining of sections AA and BB (Figure 8.21) were relatively feeble (several bars) comparing to the in situ total stress (around 40 bar) due to the high decompression rate of the 2nd shaft excavation wall. Effectively, the 2nd shaft has stayed long time with temporary soft support allowing convergence before concreting the permanent lining. The difference of pressure amongst different sensors in the same section may result from the irregular excavation wall of the 2nd shaft, to which the contour of the decompression was not homologue.

It was true for the section SS (Figure 8.23): the total pressures generated on it were not strong neither.

We remark however the total pressure generated on the section NN (Figure 8.25) of northern chamber were increased since the mounting chamber and connecting gallery excavations. In fact, the decompression due to those excavations has induced the redistribution of the total stress in the massif around excavation front.

- The continuing increase of the pressure

With the time, most of the sensors show a stable increasing tendency. The creep of the massif has been manifested by these total pressures evolution with time.

8.2.4.2 Total stress on the gallery lining

The external pressures exerted on the gallery lining are given in Figure 8.28 above. We notice a very rapid build up of stress around the lining. Two months after the installation, we can derive a pressure from the segment readings between 2.5 and 4 MPa. The speed at which the external pressure builds up, can be seen more clearly at segment 963, shown in Figure 8.41. Only 9 hours after pushing the key, the strain in the segments indicates an external pressure of several bars. Its evolution with time will provide again valuable database for investigating the long term hydromechanical behaviour of the host rock.

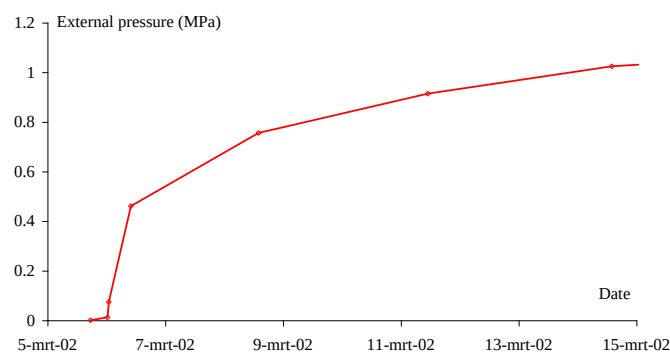


Figure 8.41 : Pressure build up during first days (ring 75, top segment 963)

8.2.5 Conclusions

The CLIPEX instrumentation programme has allowed us to obtain a very valuable data bank for the hydro-mechanical behaviour of Boom Clay.

The faithful recordings of the pore pressure measurements indicated a rather larger extent of the HDZ in the Boom clay massif (far field reaction, big variation of the pore water pressure, etc.).

Even though not all displacement sensors can be interpreted as absolute values, the relative movement around the gallery has been gotten. Thanks to the interpretation of the measurements of the extensometer A1, a good knowledge about the axial displacement on the gallery axis has been acquired. In addition, the inclinometer E1 actually gave also a good awareness of the displacement for the position above 5 m of the gallery. However, the interpretations of the inclinometers B1, C1 and deflectometer D1 are much more delicate, further efforts have to be investigated to valorise the recordings obtained.

The total stress transducers measure actually the pore water pressure in the massif instead of the total stress. This raise doubt on the reliability of the measurements of total stress in small diameter boreholes. The technique to measure the total stress remains a difficult measurement due to the borehole drilling process.

The long time measurements of the total pressure exerted on the lining provided precious database for the evaluation of the short and long term hydro-mechanical behaviour of Boom Clay and interaction between the lining and the host rock. Besides, the convergence measured from the shield during the excavation furnished a good characterisation on the instantaneous behaviour of Boom Clay under decompression.

9 Comparison of the blind predictions with the in-situ measurements

9.1 Pore pressure

9.1.1 Comparisons between SCK•CEN prediction results and in –situ measurements

For the sake of simplicity, only the prediction results obtained by SCK•CEN with Mohr-Coulomb model are compared with the in-situ measurements. Since the SCK•CEN prediction results are quite coherent with those of two other teams (GEOCONTROL and UPM) who used classical model and their corresponding normal range parameters. The pore pressure evolutions are given in function of the distance between front and the sensors as shown in Figure 9.1.

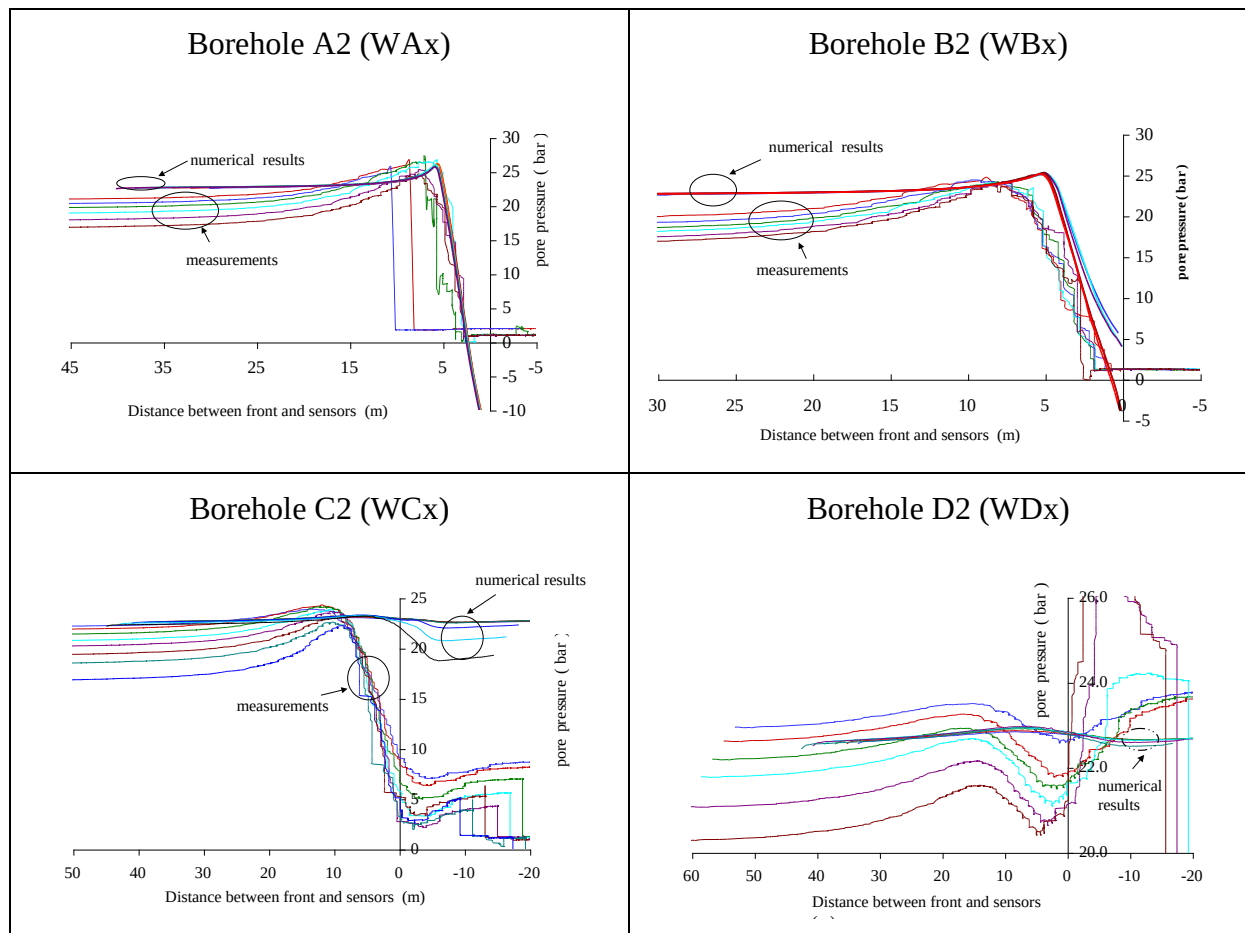


Figure 9.1 : Pore pressure comparisons between in-situ measurements and SCK's predictions

Through comparison, it can be seen

- Prediction can't produce the in-situ important gradients before excavation at each borehole

As mentioned before, the CLIPEX measurements show clearly big gradients of pore pressure before excavation of the connecting gallery. The modelling of the Test Drift excavation with a classical elastoplastic model and their corresponding normal range parameters can't produce this big difference, and the 14 year's long term water flow modelling with the given low constant permeability didn't give more water pressure drop around Test Drift. In fact, as analyzed before, the pore water pressure variation is only linked to the plastic volumetric

deformation with an elastoplastic model. The predicted plastic zone is too small to give the reaction to the distant sensors with respect to the Test Drift.

- Prediction gives the same evolution tendency as CLIPEX measurements

Numerical model predicts however the general evolution tendency: with the front progresses, the pore pressure increases progressively until a peak value before dropping down. For the sensors in the borehole C2 and D2, the model gives also an increasing appearance.

- Prediction gives comparable peak values with CLIPEX measurements, but less absolute raises.

The Figure 9.2 gives some comparisons related to the peak properties between CLIPEX measurements and numerical prediction.

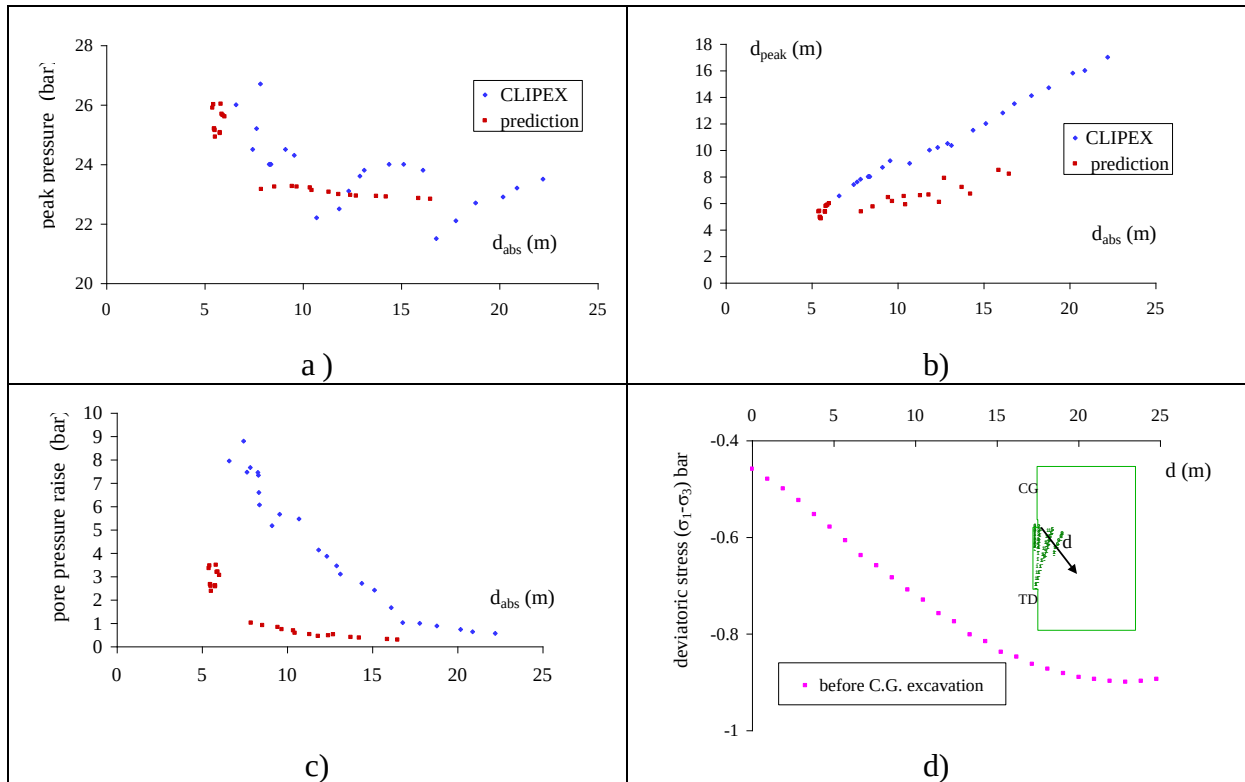


Figure 9.2 : Comparison of the peak pressure values

It can be seen:

- The predicted peak values are quite comparable with that of CLIPEX measurements (Figure 9.2a).
- But the peak appears much later numerically (the peak distance ahead of front is much smaller)(Figure 9.2 b).
- The predicted absolute raises (difference between the peak pressure and pressure before excavation) are much less important than that of CLIPEX measurements (maximal 3.5 bar compare to 8 bar, see Figure 9.2 c).

- However, the presuming abnormality was predicted numerically: the peak appears earlier in the far sensors than in the near sensors. The Figure 9.2d, where a profile of deviatoric stress along a direction d has been drawn, may bring one argument for explaining this anomaly phenomenon. As a matter of fact, as mentioned before, due to the excavation of the Test Drift, and also due to the long term drainage of the Test Drift, the deviatoric stresses increases with the absolute distance from the front. This means, from numerical point of view, the stress state in the distant sensor (so closer to Test Drift) is closer to the yielding surface of the constitutive law, consequently, it comes into the plasticity earlier than the nearer sensors.
- Predicted HDZ is much smaller than in – situ HDZ

The numerical model can't predict this far field behavior. Numerical predicted HDZ is at least two times smaller than in-situ observation. The sensors begin to react clearly only when the front approaches to about 30 meters. The borehole A2 and B2 give the responses only when the front is about 25 meters before them. The borehole C2 and D2 seem to give reaction earlier (40 meters) but the variations are very insignificant.

This fact puts again in evidence that the plastic behavior in the Boom clay can't be produced correctly with a classical constitutive model like Modified Cam-Clay and Mohr-Coulomb model. As we know, the pore water pressure variation due to excavation is closely linked to the plastic behavior. That is to say that the Boom clay needs a more sophisticate constitutive model, which is able to present more precisely the plastic behavior, to characterize it. More generally speaking, it needs a model being able to take into account correctly all irreversible behaviours (viscoplasticity), since the viscosity may be one of the causes of such extended HDZ as demonstrated by Barnichon and Volckaert (2000).

Of course, the fracturing effect should also be considered for such extensive HDZ modelling as mentioned before.

- Pressure Drop near tunnel face (boreholes A2 and B2): prediction produces correctly the Drop behavior but with higher suction generation

It's noted that the numerical model predicts correctly the drop for the boreholes A2 and B2, but generates a big suction due to the high hydro-mechanical coupling formulation (up to 15 MPa for the borehole B2). It seems not realistic. Even if the piezometer has not limitation for the measurement of the suction, it's felt that this big value of suction is beyond the reality. The fact that the CLIPEX records an abrupt increasing confirms this feeling.

- Pressure Drop in the far field (boreholes C2 and D2): the model predicts somewhat pressure drop but much less important than CLIPEX measurements for the boreholes C2 and D2.

This is again an evidence for the lack of precision on the plastic behavior modelling.

- Pore pressure variation during lining installation can't be reproduced by numerical model considering only plasticity.
- Prediction gives also the pore water pressure perking up after drop down

It's worth to note that the regains of pore pressure are also modelled for the borehole C2 and D2, but very slightly comparing to the CLIPLEX measurements.

9.1.2 Comparisons between G3S prediction results and in –situ measurements

The major difference between G3S and other teams resides in two aspects:

- G3S has adopted a non linear permeability law: the permeability varies in function of the plastic dilatation. In this way, the permeability increases in the plastic zone around the gallery excavation.
- The G3S has decreased significantly the internal frictional angle of Boom clay to increase the plastic zone and so extend the EDZ and HDZ.

Part of the pore pressure prediction got by G3S is compared to the in situ measurements as shown in Figure 9.3.

We observe that,

- The pressure gradient before C.G. excavation is well produced numerically. For those near face boreholes (A2 and B2), the prediction gradient fits well in-situ one. But for the faraway boreholes (C2 and D2), the prediction even overestimates the actual gradient.
- The numerical pressure evolution matches well the in situ evolution. But the fluctuation of pore pressure during C.G. excavation is still less than the measured one (absolute pressure raise, pressure drop, perking up, etc.).
- The far field responses during C.G. excavation is not yet produced numerically

Actually, the parameters used by G3S were obtained on the base of several " trial and error " calculations of the Test Drift excavation in seeking for the " best solution " for the initial pressure profile measured by the piezometer. According to the report of G3S, using a classical elastoplastic model like Cam-clay and Mohr-coulomb with their normal range parameters, the non linear permeability law didn't contribute very much the HDZ generation. The "best solution " was obtained thanks to the significant diminution of the frictional angle. However, in all appearance, G3S's results didn't give more pore pressure fluctuation during the C.G excavation in spite of the lower value of the frictional angle used. The generated HDZ during the C.G excavation is not larger than other teams predictions.

All these analysis led us to conclude that the hydro-mechanical coupling behavior in Boom Clay may be very complex. A simple classical elastic perfect plastic model is certainly not sufficient to characterize it. The G3S's results showed that the consideration of a non linear permeability in the damage zone is not enough neither to seize all observation in situ. Something else is still missing in the model: viscosity, hardening - softening, visco-plasticity coupling, de-saturation around the gallery etc. are some traces to follow.

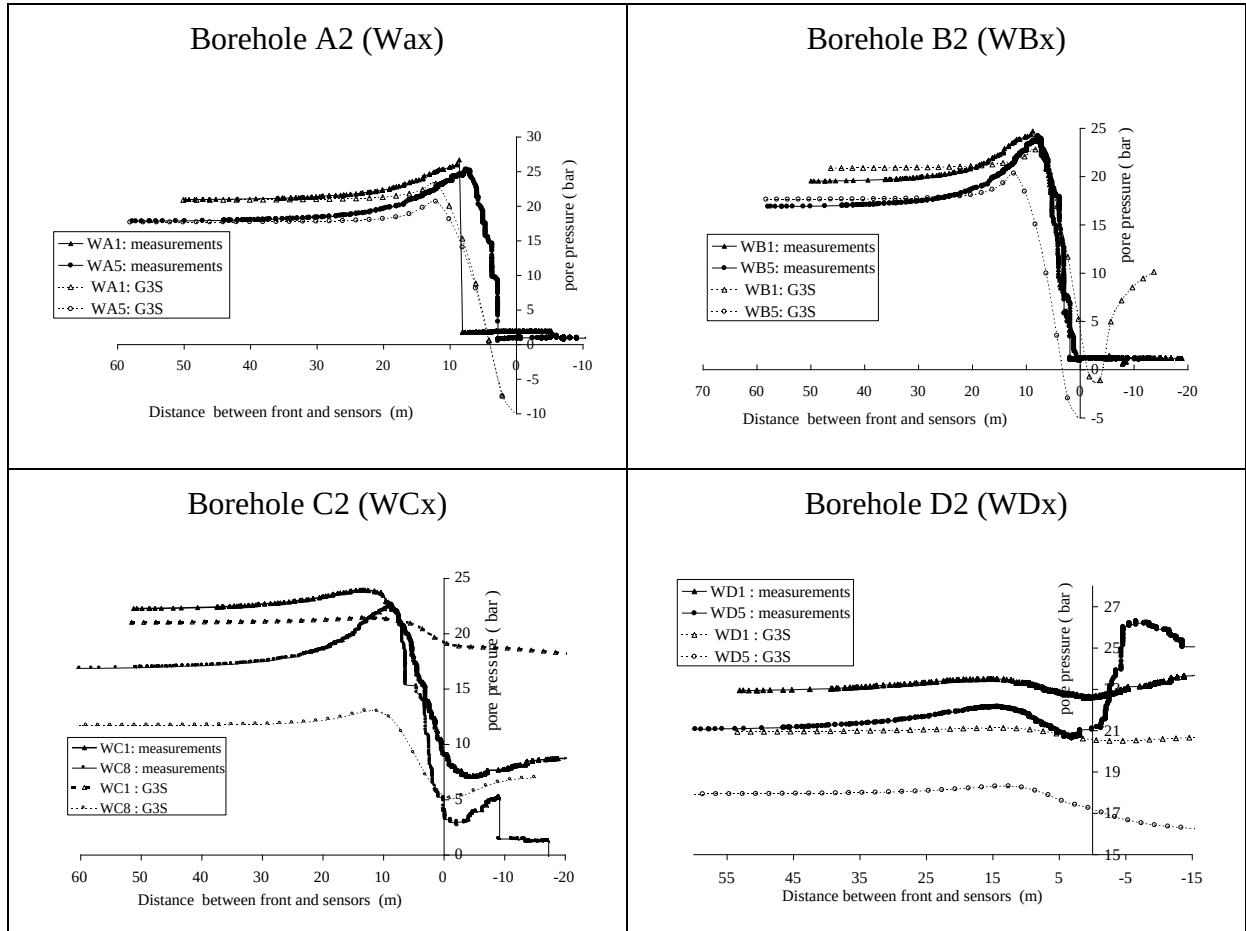


Figure 9.3 : Comparisons between in-situ measurements and G3S's pore pressure prediction

9.2 Displacements comparisons

As noted before, the extensometer A1 and E1 have provided faithful measurements. The actual interpretation of the inclinometer B1 offered only the tendency of the displacements. Efforts for the comparison between numerical prediction and in – situ measurements will be concentrated on these three boreholes.

9.2.1 Extensometer A1

The simulation results (obtained by SCK.CEN, GEOCONTROL and G3S) show that the axial displacements on the different point of axis response to the excavation in a very similar way: the curves obtained for different sensor are more or less parallel (Figure 9.4). As explained before, all CLIPEX sensors measure in fact the displacements of one point situated about 5 m before T.D. front. Based on this, an extrapolation of the numerical results to this point is drawn and putted together with in situ measurements as shown by Figure 9.4. We observe that the in situ measurements look very consistent with extrapolation result. The final axial displacement is well produced (about 120-140 mm). However, as the pore pressure prediction, the model "reacts" much later than in situ sensors: the model begins to record the axial displacement only when the excavation front is about 20 meter before them. The in situ sensors gave the responses nearly as soon as the excavation begins. The EDZ predicted by the model is consequently much smaller than in reality.

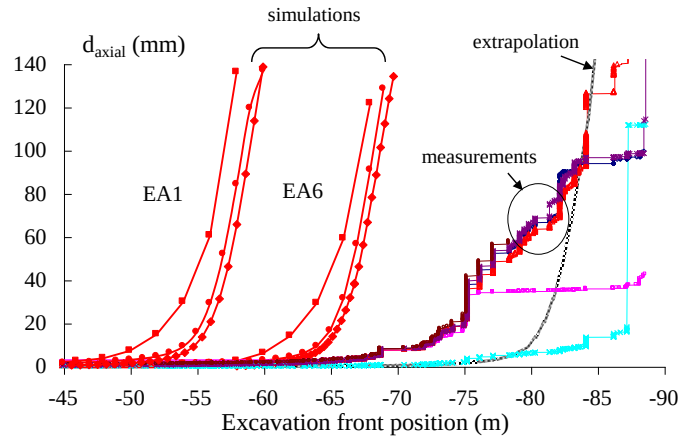


Figure 9.4: Displacements comparisons between in-situ measurements and predictions (A1)

9.2.2 Inclinator B1

Due to the difficulty for the interpretation of the measurements obtained by inclinometer B1, we consider only the relative movement recorded by the sensors. To do so, the measurements are shifted to "muffle" the abnormal recordings as shown by Figure 9.5. The simulation results obtained by GEOCONTROL are put beside for the comparison. We observe that the tendency is well predicted. But the vertical displacements measured by inclinometer seem extremely high compare to the numerical ones. Contrarily, the measured axial displacements are nearly insignificant compared to the numerical ones. As explained before, the bending of the chain in the inclinometer may cause likely false interpretation. So, it's meaningless to do the absolute comparison at this moment. More effort will be investigated for the interpretation of these measurements.

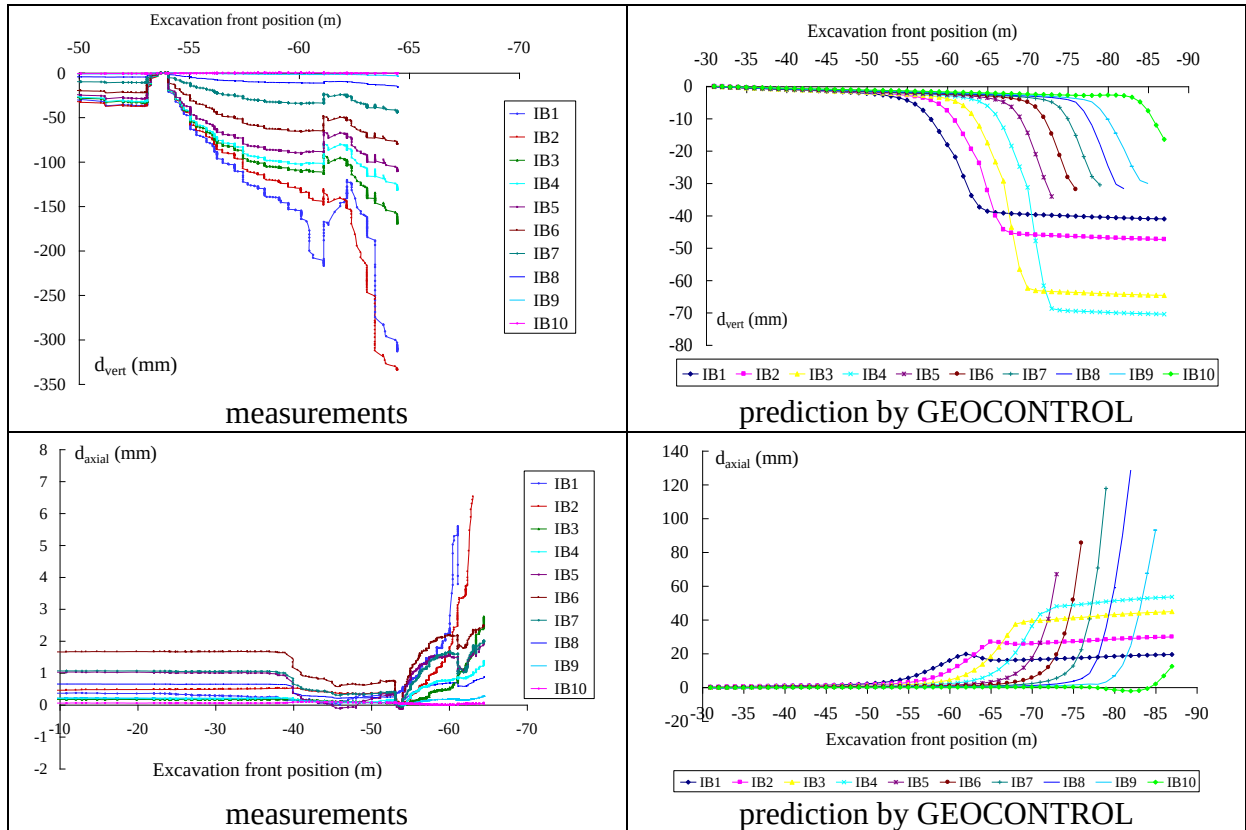


Figure 9.5: Displacements comparisons between in-situ measurements and predictions (B1)

9.2.3 Inclinator E1

The inclinometer measurements compared to the GEOCONTROL prediction results obtained with their model for zone 2 as shown in Figure 9.6 (GEOCONTROL, 2002).

We note that the in situ displacements are much more important than the numerical ones, nearly 8 times higher for the deepest sensor (IE1, IE2 for example) have been recorded. The 2nd shaft influence seems not be very significant in the numerical prediction. Firstly, the high rate decompression of the 2nd shaft due to the lately installation of the lining was not taken into account numerically. Secondly, the classical elasto perfect plastic model can't produce the extended EDZ in the Boom clay.

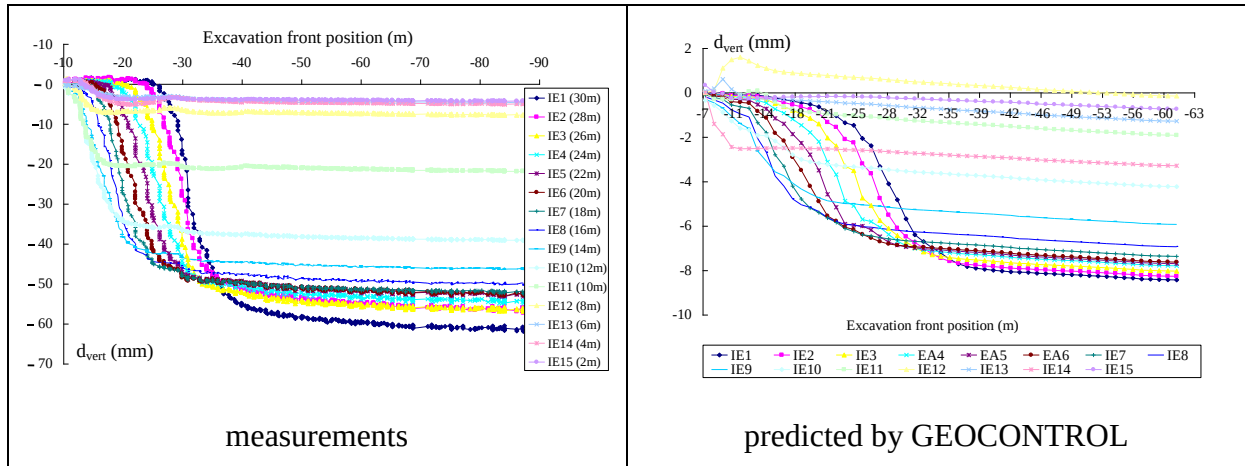


Figure 9.6 : Displacements comparisons between in-situ measurements and predictions (E1)

9.3 Conclusions

Generally speaking, the whole tendency of the pore pressure evolution during the excavation can be predicted, but still diverge largely from the measurements. As analysed above, these divergences are due to many factors, apart from the pure numerical aspects such that the initial and boundary conditions, modelling procedure etc., the following factors may be considered as most influential:

- A classical elasto perfect plastic model can't represent the plastic behavior of Boom clay.
- The fracturing phenomenon, which should modify consequently the permeability in the massif and, especially modify the hydro-mechanical boundary conditions, should be taken into account.
- The viscosity behavior must play a role in the pore pressure prediction.
- Non saturated hydromechanical behavior around the gallery due to the de-saturation should be considered also.
- The Test Drift excavation procedure should be re-examined: taking into account the real advancing velocity and hydraulic boundary condition.

It is important to highlight that the pore pressure variation due to the excavation depends essentially on the non-elastic behavior of the massif, since in the elastic zone, there is not pore water pressure variation due to the null volumetric deformation. Consequently, modelling correctly the plastic behavior is of first importance whatever other considerations.

Concerning the displacement comparison, the final axial displacement on the gallery axis was well produced (about 120-140 mm) thanks to the extrapolation of the numerical results and in-situ data interpretation for the extensometer A1. However, the EDZ predicted by the model is much smaller than in reality.

On the contrary, the comparison for the displacements around the gallery measured from the test drift stayed meaningless up to now due to the difficulty of the interpretation of in-situ data obtained by inclinometers B1, C1 and deflectometer D1.

The displacements measured from inclinometer E1 are 8 times higher than the numerical ones. The 2nd shaft influence seems not be very significant in the numerical prediction. The high rate decompression of the 2nd shaft due to the lately installation of the lining was not taken into account numerically. Furthermore, the classical elasto perfect plastic model can't produce the extended EDZ in the Boom clay.

All these analysis led us to conclude that the hydro-mechanical coupling behavior in Boom Clay may be very complex. A simple classical elastic perfect plastic model is certainly not sufficient to characterize it. For this reason, a new simulation with a more sophisticated elastoplastic model has been initiated. Its results will be given hereafter.

10 New tentative simulations

As mentioned before, modelling correctly the plastic behavior is certainly an important issue for the comprehension of the hydromechanical behaviour Boom clay. An effort has been made to do the prediction trying with a constitutive law more sophisticated and probably more capable to match the real Boom clay behavior: the Dafalias-Kaliakin Bounding surface model (see also Barnichon, 1998).

This generalized constitutive model, based upon the concept of the bounding surface in stress space, is developed with the frame of critical state soil mechanics.

The prominent feature of this concept is the fact that inelastic deformation can occur for stress points within the bounding surface. This concept of bounding surface has been used in a variety of constitutive laws for material such as metals, isotropic/anisotropic cohesive soils, sands and concrete. More detail description of the model can be found in (Kaliakin et al. 1987, Kaliakin and Dafalias 1990).

10.1 Simulation results

Parts of the numerical results are given in Figure 10.1.

It's felt necessary to point out that, until now, the additional parameters required for this model have not been determined experimentally. However, they are crucial for differentiating the present bounding surface model from other classical model. On the other hand, these parameters offer more possibilities for getting closer the numerical results to the measurements, that is to say, there exists the possibility to calibrate these parameters using the CLIPLEX measurements. It's not the case for other classical models used.

It can be seen that:

- The evolution of total pressure is smoother than that obtained by classical model. The sharp transition of the elasticity to plasticity is not evident in the bounding surface. The maximum total stress seems lesser than that of classical model.
- The pressure exercise on the lining stays nearly the same: about 2 MPa.
- However, the present parameters used in the model predict very large displacements with respect to other classical models. The axial displacement ahead of the face reaches to 40 cm, the radial convergence of the gallery face exceeds 45 cm. An initial radial convergence about 20 cm at gallery circumference has also registered.
- The plastic zone extends more than 30 meters in radial direction according to the preconsolidation pressure value.
- Two profiles of pore pressure are put together with those obtained by Mohr – Coulomb as shown in Figure 10.2. The difference is marked: the bounding surface predicts more pore water variation. The HDZ obtained by Dafalias - Kaliakin model is much more extended than that of Mohr-Coulomb (see Figure 10.2 a). The profile along the symmetrical axis shows a very stretched " peak " unlike in a classical one, but with a lower "peak" value (about 5 bar lesser, see Figure 10.2b). This difference of pore pressure can be visualized more clearly with the help of Figure 10.3 and Figure 10.4.

- The suction is also predicted by the bounding surface model.

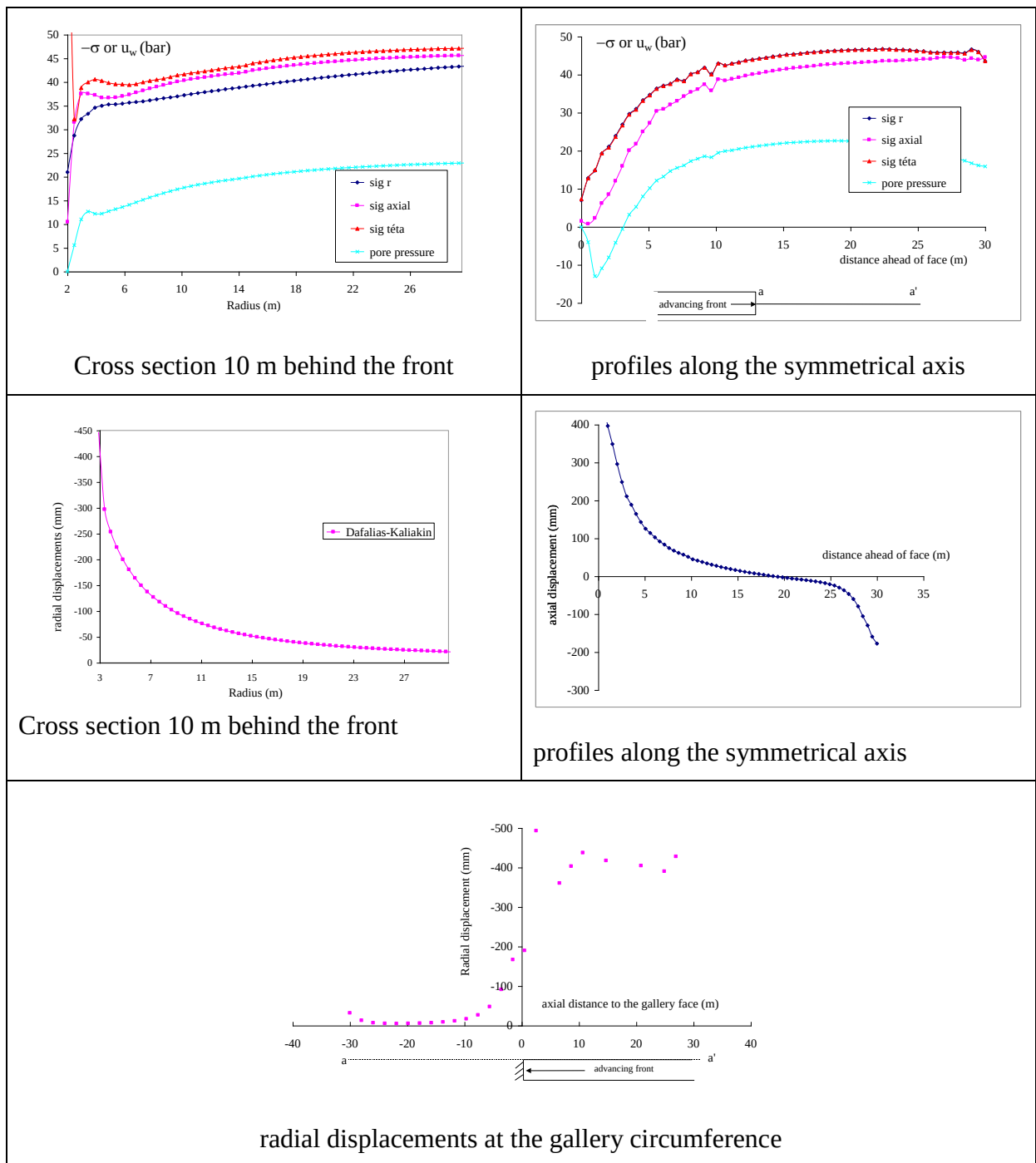


Figure 10.1 : Numerical results obtained with Dafalias –Kaliakin bounding surface model when the front is at -60 m position

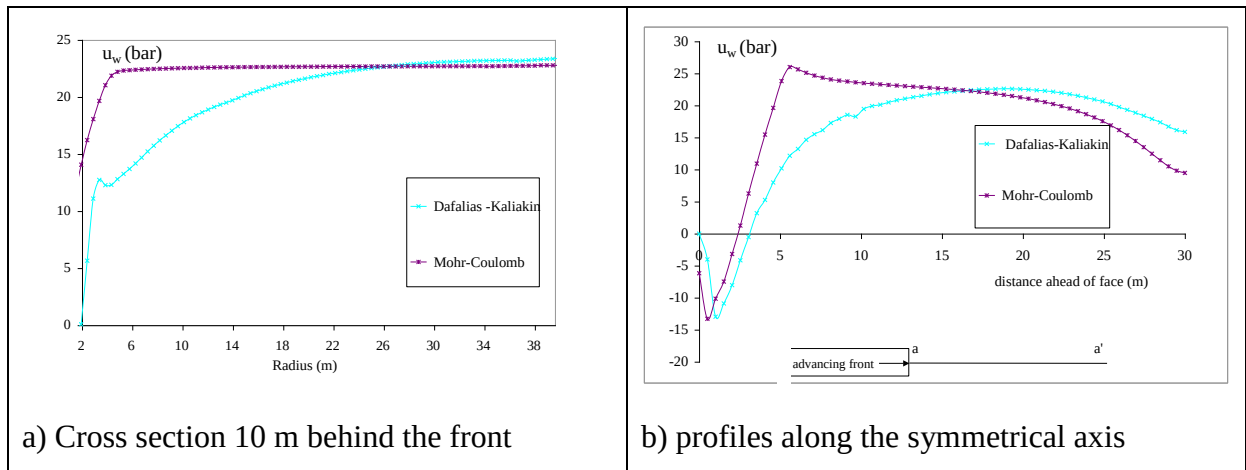


Figure 10.2 : Comparison : Dafalias-Kaliakin –Mohr-coulomb (front position: -60m)

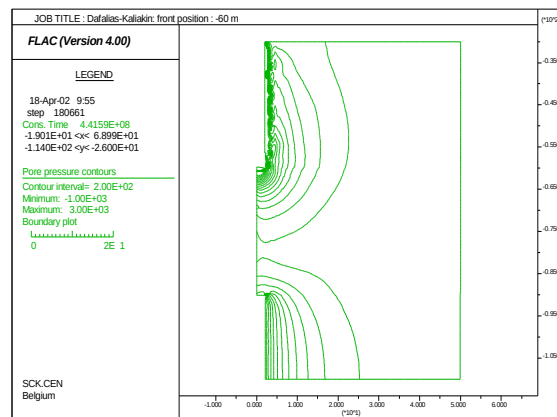


Figure 10.3 : Pore pressure predicted by bounding surface model

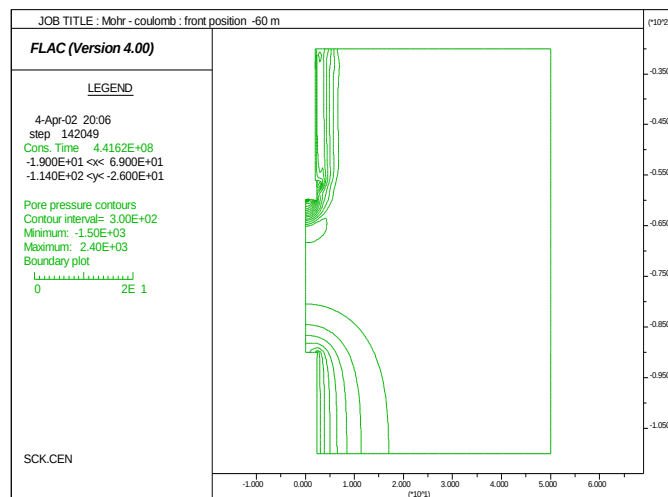


Figure 10.4 : Pore pressure predicted by Mohr -Coulomb model

10.2 Comparison with CLIPEX measurements

The predicted pore pressures with bounding surface model are compared to the CLIPEX measurements in the same manner as shown in Figure 10.5 hereafter.

We remark that, with respect to the classical model, some results go better, some ones go worse in comparing with the CLIPEX measurements;

For the near boreholes A2 and B2, the evolution goes more smoothly. The "peak" is not so marked numerically, but it exists. The pore water pressure reaches to his " peak " value much more earlier: at about 25 m ahead of front.

Nevertheless, for the far sensors in the borehole C2 and D2, the bounding surface model produces the results much closer to the CLIPEX measurements: the tendency of drop down and perking up approaches to the reality with respect to the classical model. But, the drop down in the borehole D2 seems too important with respect to the CLIPEX measurements.

It's important to note that the bounding surface model still not reproduces the high pore pressure gradient before connecting gallery excavation. It signifies that a correct Test Drift excavation modelling asks other considerations: water flow, viscosity etc.

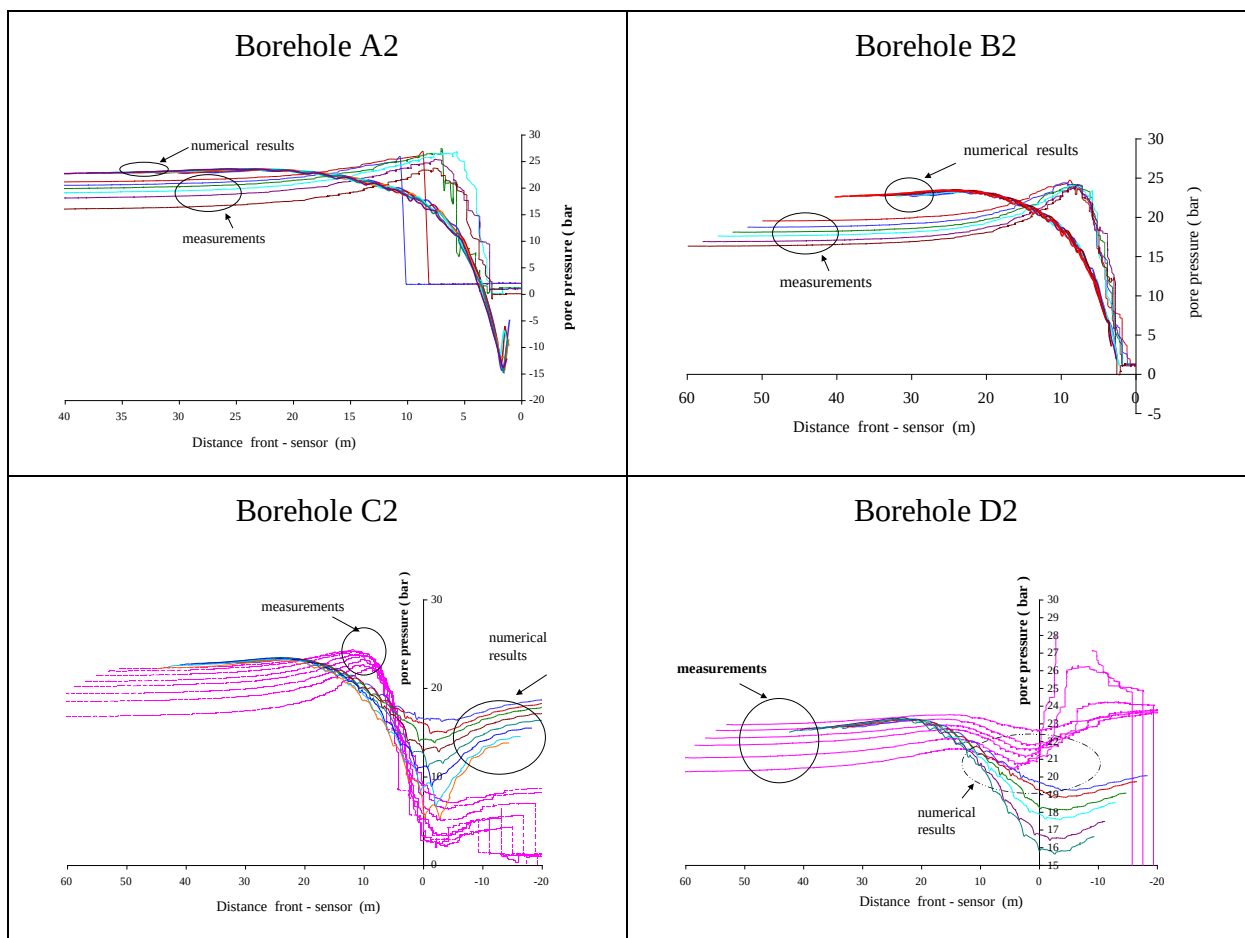


Figure 10.5 : Comparison bounding surface model with in-situ measurements

10.3 Comments

The numerical results obtained by Dafalias –Kaliakin model are not more satisfactory, but they differentiate from the classical models. This offers some hopes to ameliorate the plasticity behavior modelling. It should be pointed out that this is a first attempt. More attention should be paid to the determination of the parameters.

11 Lessons learned

Through the instrumentation programme, we have obtained a lot of experience, both from the application/installation of the sensors, as from the measurement performance, and consequent use of the results in the modelling.

The design of the instrumentation programme based on a preliminary modelling has proven to be very valuable. It gives a firm basis for the specification of the instrumentation programme (type of measurements, location, measurement range,...) and will enhance the link with the modelling through the back-analysis.

The specifications have to be met by the technical and experimental conditions, such as the installation of sensors in boreholes.

Measurement of pore waterpressure through the piezometers has shown to be a very reliable and mature technique. This method is based on convergence of the clay after drilling and immediate installing of the borehole. The twin-tube design allows further purging the filter and active test procedures to determine other hydraulic parameters. The pressure peak at some filters was higher than expected – the design allowed to replace some pressure transmitters to increase the measurement range.

Total pressure measurement in small diameter boreholes still is a pending issue. First, the reliability of the Kulite total pressure sensors is not very high – a significant number of the sensors failed over the experiment period. Apart from this, the total pressure sensors on the piezometers do not observe any significant effective pressure. Similar observations have been made at the RESEAL borehole test (Volckaert et al. 2000). In this test, the total pressure cells in contact with the clay host rock only indicated the pore waterpressure, whilst the same type of sensor in contact with the backfill material reacted to the swelling (effective pressure) of it – indicating the correct functioning of these sensors. The observation on the lined gallery, showing a rapid development of the (effective) pressure, indicates the possibility that this phenomenon is subject to the scale: the effect of arching is apparently larger in small diameter holes.

The measurement of the displacements in clay can also be improved, both on the reliability of the instrument set-up, as well as on the data interpretation.

The multipoint borehole extensometer is apparently not suited to cover deep anchors in (converging) clay, as the link between the anchor and the transducer (guiding rods in protective tubes) seems to be obstructed. A type with a continuous casing, which was initially not retained due to possible interference with the excavation, is probably a better option to cover longer distances in plastic clay.

The borehole casing is further the most fragile aspect of the displacement measurements. In none of the three boreholes (B1, C1, E1), the plastic casing of the inclinometer (ABS) was strong enough to resist the pressures, requiring an extra drilling of E1, and putting several sensors of the C1 inclinometer out of service. The B1 casing showed leakage of the casing. In addition, grouting of the borehole needs to be performed very carefully. No grouting could have decreased somewhat the final pressure on the lining, but would also have disturbed more the surrounding clay. The final solution adopted for the E1 inclinometer – installing a steel casing prior to the actual (ABS) inclinometer casing, is required for a reliable functioning in clay. The aluminium casing, which is not offered anymore by the instrument manufacturers

due to corrosion problems, is a viable option, as corrosion is apparently not a major problem in Boom Clay.

The CLIPEX instrumentation programme has allowed us to obtain a very valuable data bank for the instantaneous hydro-mechanical behaviour of Boom Clay: The pore water pressure and the displacements evolutions have been followed in three dimensions before and during the construction of a gallery.

It is the first time that pore water pressure could be measured outside the zone of influence of the HADES laboratory. This measurement allowed to confirm the initial undisturbed pore-water pressure about 2.2 MPa.

The evolution of the pore water pressure obtained during excavation has provided us with very valuable information, because the pore pressure measurement in the Boom Clay is quite reliable and precise, and because it is very sensitive to mechanical variations (volumetric part) due to the strong hydro-mechanical coupling existing in the Boom Clay.

The displacement measurements in the CLIPEX instrumentation programme should provide a valuable complementary database to characterise the hydromechanical behaviour. A good knowledge about the axial displacement on the gallery axis has been acquired.

The instrumentation installed in the lining permitted to assess the short and long term hydro-mechanical behaviour of Boom Clay and interaction between the lining and the host rock. In addition, the convergence measured through the shield of the tunnel machine provided an accurate characterisation of the instantaneous behaviour of Boom Clay under decompression.

It was remarkable to observe effect of the different excavation phases from the pore water pressure and displacement measurements. This proves the sensitivity and the reliability of the measurements.

The propagation of the fractures ahead of the excavated face was also observable. Fractures were opened ahead of the excavated front up to a distance of 4 m along the gallery axis.

The in-situ measurements indicated a very large extent of the Hydraulic Disturbed Zone.

The comparison of the blind predictions obtained by different partners shown that, when using a classical elastoplastic model, various aspects can still influence the numerical results: size of mesh, integration method of constitutive law, hydro-mechanical parameters, initial boundary conditions, etc. Based on the comparison, a 2D axisymmetric model can produce quite challenging results with respects to a 3D model in considering the isotropic stress condition. The confrontation of the numerical prediction with the in situ measurement indicated that improvements would be made for the prediction of pore water pressure.

12 Perspective research programme

The comparison of the blind predictions obtained by different partners shows that, when using a classical elastoplastic model, various aspects can still influence the numerical results:

- a) size of mesh
- b) integration method of constitutive law
- c) hydro-mechanical parameters, initial and boundary conditions
- d) assumption on the geometry of the model: 2D axisymmetric or 3D model.

The first feeling is that, based on the comparison, a 2D axisymmetric model can produce quite competitive results with respect to a 3D model in considering the isotropic stress condition and without 2nd shaft influence.

The confrontation of the numerical predictions with the in situ measurements indicates, whatever improvements can be made in the above numerical aspects, the most important issues to consider are following ones:

- e) plasticity behavior:

A correct consideration of plasticity behavior ameliorates certainly the quality of prediction, especially the evolution of the pore water pressure.

- f) Time related behavior:

It is linked to two aspects: hydraulic diffusion process and viscosity of skeleton.

- g) Damage model development:

The fracturing observed around the gallery has without doubt a heavy responsibility to the hydro-mechanical behavior. Its generation, its evolution accompanying to the hydraulic permeability variation should be considered.

Consequently, future research on modelling should be concentrated on the development of points e), f) and g).

Dealing with the instrumentation, future efforts should be focused both on reliability and measurement performance. Reliability in converging clay conditions includes, apart from the external pressure on the casing, movements in weakly supported fronts, corrosion aspects and watertightness. Measurement performance mainly deals with the total pressure around small and large excavations; the performance of displacement measurements can be further enhanced by a better control of the boundary conditions to refer all movements to an absolute framework.

Finally, for the sake of the long-term safety of a repository, a significant effort should be devoted to the long-term hydromechanical behaviour study of the boom clay.

13 Conclusions

The CLIPEX project was realised in the frame of the extension of the underground research facility. The extension consisted in the realisation of a second shaft and a connecting gallery of 90 m length. As the connecting gallery was excavated from the second shaft to the existing facility, a unique and original opportunity was then given to monitor hydro-mechanical parameters ahead of the face of the advancing gallery. An important part of the project was also devoted to modelling.

The project started with the characterisation of the host-rock in the zone to be excavated. This programme consisted in the realisation of pressuremeter, dilatometer and hydro-fracturing tests to measure the natural stresses, the permeabilities, the deformability and the strength of the rock mass.

An important part of the project was then devoted to blind predictions of the hydro-mechanical behaviour of the clay massif relative to the construction of the connecting gallery. Five different modelling teams were involved: G3S, UPM, GEOCONTROL, SCK•CEN and ONDRAF/NIRAS. The blind predictions consisted of two main steps:

- Preliminary modelling (case studies) to compare the predictions of the different codes for problems related with the excavation of underground structures.
- Blind predictions of the hydro-mechanical behaviour of the clay mass during and after the construction of the connecting gallery.

An extensive in situ instrumentation programme was afterwards established. Within the context of the extension of the underground research facility, we got the possibility to measure the hydro-mechanical response ahead of the face of the advancing gallery for the first time in Boom Clay. The host-rock was therefore instrumented both in the zone to be excavated and around the gallery: from the Test-Drift (part of the HADES laboratory before its extension), from the second shaft and in the lining. In addition, the tunnel machine was also instrumented to measure the radial convergence profile on the wall between the excavated front and the last installed lining element. The choice of the location and magnitude of the sensors was based on the results of the preliminary modelling.

Furthermore, the geometry of the tunnel machine and the design of the lining elements have been optimised based on the good prediction of the displacements and of the pressure on the lining. The construction of the connecting gallery has finally been realised with a Tunnel Boring Machine (TBM) composed of a 2.3 m long shield, a road-header for the excavation of the rock and a bird-wing erector system for the installation of the lining. The shield was equipped with a cutting head to ensure a smooth excavated profile. The wedge block system was used for the lining, which is an expanded lining system. In order to reduce the plastic zone created by the tunnelling process, a fast advancing rate was acquired, the over-excavation was reduced to a minimum, and the lining was installed as soon as practical possible to minimise radial movement of the clay wall.

The CLIPEX instrumentation programme has allowed us to obtain a very valuable data bank for the instantaneous hydro-mechanical behaviour of Boom Clay.

The in-situ measurements indicated a very large extent of the Hydraulic Disturbed Zone. The far extent of hydraulic variations has no impact on the long term safety of a repository, since the permeability is not affected in this zone.

Indeed, variations of the pore water pressure were observed in the zone instrumented from the Test-Drift during the construction of the second shaft and from the beginning of the excavation of the connecting gallery. This means that pore water pressure reacts at distance about more than 60 m. This very surprising result is obviously of prime importance for the proper understanding of the hydro-mechanical behaviour of Boom Clay.

The comparison between the prediction results and in-situ measurement, considering classical constitutive laws like modified Cam-Clay and Mohr-Coulomb model, showed a very good agreement for the displacements and the equilibrium pressure on the lining. The simulations can reproduce the tendency of the pore water pressure evolution but cannot explain the large extent of hydraulic disturbance. Of course the pore water pressure is much more sensitive to mechanical changes than the displacements. First tentative simulation with a more sophisticated constitutive law (Bounding surface elastoplastic law) has brought some hopes of amelioration on the far field behaviour, it asks however more work for the calibration of parameters. The consideration of the viscosity of the skeleton and the fractured zone around the gallery could also increase the quality of the prediction of the pore water pressure.

The step by step blind predictions realised during the CLIPEX project are therefore great assets for the nuclear waste storage research:

Firstly, they gave a very precious support for the design of the excavation technique and more especially for the geometry of the tunnelling machine. The realisation of the connecting gallery at 223 m depth using wedge block system was really a challenge. Modelling has allowed to predict correctly the convergences between the excavated front and the lining and to choose the optimal geometry of the shield. No main problems were encountered during excavation. This has allowed to reach a progress rate about 3m/day.

Secondly, the fact that a classical elastoplastic constitutive law can't explain the large extent of hydraulic disturbance will stir up the further research on the hydromechanical behavior of Boom clay.

The results of the CLIPEX project showed that the technique used for the realisation of the gallery lining is acceptable both from an economical and technical point of view. The measurements of displacement and of equilibrium pressure on the lining proved that the hydro-mechanical perturbation has been minimised and that the technique allows a very good control of the excavation parameters. The measure of the pore water pressure and the observations demonstrate that open fractures do not extent more than 0.5 meter around the gallery. The observed total radial convergence, about 9 cm at the excavated wall, is considered acceptable in terms of hydro-mechanical disturbance. It is important to note that this convergence take into account the convergence occurring ahead of the excavated front.

The CLIPEX project is therefore a great success and constitutes an important step for the development of the design of an underground nuclear repository. The CLIPEX project constitutes also important issues and applications in the field of civil engineering. The results of the project have been disseminated in many international conferences.

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Appendix 1

CLIPLEX

Work Package 4

Activity 4.1 : "Blind predictions"

Test case n°1 : Undrained unloading of a cylindrical cavity in a poro-elasto-plastic medium

Framework

As it has been proposed in the first co-ordination meeting (Brussels, 14-15 January 1997), the activity 4.1 "Blind predictions" of the CLIPLEX programme will essentially consists in :

- two case studies to compare the predictions of the different codes in problems related with the excavation of underground structures. In those calculations, all the data will be specified : i.e. geometry (except the mesh), boundary conditions, constitutive model, geotechnical characteristics, and even construction sequence.
- further calibrations of constitutive models and geotechnical properties for Boom Clay, from the existing laboratory and in situ data. Based on their experience, the modelling teams will be free to use the data they consider the most appropriate.
- blind predictions of the hydro-mechanical behaviour of the clay mass during and after the construction of the connecting gallery. They will quantify the evolution with time of pore pressures, stresses and displacements in the medium as well as pressures on the gallery lining.

Within the first stage, two calculations are foreseen :

- a 1-D problem (axisymmetry and plane strain conditions) that is an idealization of the excavation of a cylindrical cavity in an infinite medium;
- a 2-D axisymmetric problem much closer to the in situ reality that will be encountered during the construction of the future connecting gallery. It will among others account for the decompression of the clay mass ahead of the excavation face, for the support laying distance and for the sequential tunnelling process.

The numerical results got from those calculations will give a first idea of the hydro-mechanical disturbances induced by the excavation of the connecting gallery. It will help to work out an efficient instrumentation programme :

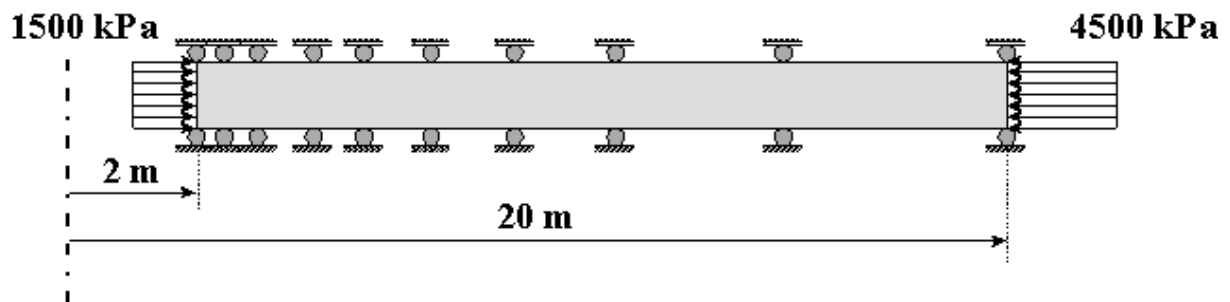
- by defining usefull locations for the instruments,
- by assessing the expected magnitude of their measurement.

Since the definitive elaboration of the instrumentation programme and the choice of the instruments are scheduled for the next co-ordination meeting in june, the modelling works of both case studies have to be completed by the end of may...

Common calculation

Geometry

The first test case is quite simple. It consists in the undrained unloading of a cylindrical cavity in an infinite poro-elasto-plastic medium. For that purpose, owing to the axisymmetry and the plane strain condition, a one-dimensional analysis is sufficient. The geometry and the boundary conditions of the problem are hereafter represented.



The internal boundary corresponds to the radius of the excavation: 2 m; and the external one is selected at 20 m from the radial symmetry axis (i.e. five tunnel diameter). Before excavation, the in situ stress field is hydrostatic : total stress p_0 of 4500 kPa and pore water pressure p_{p0} of 2250 kPa.

The excavation is idealized reducing the inner total pressure from 4500 to 1500 kPa. On the other hand, the outer total pressure is kept constant and equal to 4500 kPa. A fast unloading is assumed and the problem can be considered as undrained ($\equiv$ no flow of water).

Two constitutive models have been chosen, and consequently two runs have to be performed. For comparing purposes, both simulations should be carried out as small-strain analyses.

First set of data

- circular tunnel	radius	R_i	2	[m]
- homogeneous, isotropic medium				
- hydrostatic in situ stress field :	initial total stress	p_0	4500	[kPa]
	initial pore water pressure	p_{p0}	2250	[kPa]
- soil behaviour :	<u>linear-elastic perfectly-plastic model</u>			
	<u>with a Mohr-Coulomb failure criterion</u>			
- <u>drained</u> soil parameters :	Young's modulus	E'	300	[MPa]
	Poisson's ratio	ν'	0.125	[]
	friction angle	ϕ'	18	[°]
	cohesion	c'	300	[kPa]
	dilation angle	ψ	0	[°]
- pore water data :	porosity	n	0.39	[]
	water bulk modulus	K_w	2	[GPa]
- total pressure on the gallery wall :		p_i	1500	[kPa]

Second set of data

- circular tunnel	radius	R_i	2	[m]
- homogeneous, isotropic medium				
hydrostatic in situ stress field :	initial total stress	p_0	4500	[kPa]
	initial pore water pressure	p_{p0}	2250	[kPa]
- soil behaviour :	<u>Modified Cam-clay model</u>			
- soil parameters :	Slope of the normal consolidation line	λ	0.13	[]
	Slope of the elastic swelling line	κ	0.02	[]
	Critical state friction angle	ϕ'_c	21	[°]
	Preconsolidation pressure	p'_c	6	[MPa]
	Poisson's ratio	ν'	0.2	[]
	Specific volume at 1 kPa on the NCL	N	2.75	[]
- pore water data :	porosity	n	0.39	[]
	water bulk modulus	K_w	2	[GPa]
- total pressure on the gallery wall :		p_i	1500	[kPa]

Results

For both soil models, the results that are expected from the participants are the distributions in the medium ($2 \text{ m} \leq r \leq 20 \text{ m}$) of the following variables:

- radial total stress	Φ_r	Represented as a function of the distance to the tunnel symmetry axis : r
- tangential (or circumferential) total stress	Φ_t	
- longitudinal (or axial) total stress	Φ_l	
- water pore pressure	p_p	
- radial displacement	u_r	

N.B. Remember that the stresses and displacements are probably not calculated at the same location...

For an easier comparison, three characteristic values are as well required: the radial displacement and pore water pressure at the gallery wall ($r = 2 \text{ m}$) as well as the damaged zone extent.

Complementary calculations

As agreed in the first co-ordination meeting, each modelling team is free to perform complementary calculations based on its own experience. Other constitutive models and/or other geotechnical data could for instance be tried.

At CEN•SCK, at least two complementary calculations with the "Mohr-Coulomb" case will be realised:

- one with a pore water bulk modulus K_w of 0.2 GPa.
- one with a dilation angle ψ of 10°.

Appendix 2

CLIPEx

Work Package 4

Activity 4.1 : "Blind predictions"

Test case n°2 :

Undrained excavation of the connecting gallery in the Boom clay formation

Framework

This second test case is much closer to the in situ reality that will be encountered during the construction of the future connecting gallery. It accounts among others for the decompression of the clay mass ahead of the excavation face, for the support laying distance and for the sequential tunnelling process.

The numerical results got from those calculations will give a first idea of the hydro-mechanical disturbances induced by the excavation of the connecting gallery. It will help to work out an efficient instrumentation programme:

- ✓ by defining useful locations for the instruments
- ✓ by assessing the expected magnitude of their measurement.

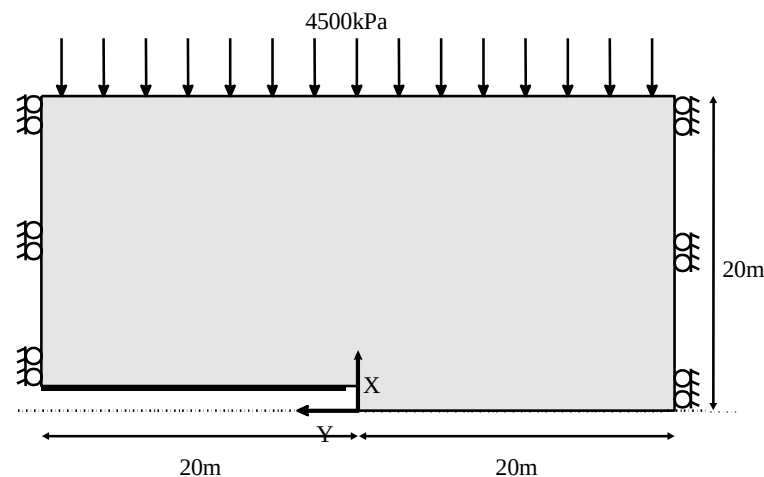


Figure II-1 : Geometry and boundary conditions

Data

Geometry

Figure II-1 represents the geometry (at the end of the run) and the boundary conditions of the problem:

- ✓ the axis of the tunnel ($x=0$ m) is an axis of radial symmetry;
- ✓ the external boundary is selected at 20 m from the symmetry axis (i.e. five times of tunnel diameter);
- ✓ the lateral boundaries are located 20 m on both sides from the final position of the tunnel face ($\equiv$ origin of the Y axis);
- ✓ the medium is homogeneous and isotropic;
- ✓ before excavation, the *in situ* stress field is hydrostatic: total stress p_0 of 4500 kPa and pore water pressure p_{w0} of 2250 kPa.

Since the supporting system of the connecting gallery is not yet decided, a lining consisting of cast iron segments has been assumed for the modelling. Its specifications are hereafter described:

- ✓ internal radius 1.75 m
- ✓ segments thickness 1.6 cm
- ✓ segment width 0.5 m
- ✓ Young's modulus 210 GPa
- ✓ Poisson's ratio 0.3

To account for the hollowness of the segments, the lining should be modelled with either an equivalent thickness of ≈ 6 cm or an equivalent Young's modulus of 75 GPa.

Geotechnical data

The same constitutive models and geotechnical data as in the test case n° 1 are taken.

First set of data

- soil behaviour : linear-elastic perfectly-plastic model with a Mohr-Coulomb failure criterion
- drained soil parameters :

Young's modulus	E'	300	[MPa]
Poisson's ratio	ν'	0.125	[]
friction angle	ϕ'	18	[°]
cohesion	c'	300	[kPa]
dilation angle	ψ	0	[°]

First set of data

- soil behaviour : Modified Cam-clay model

- soil parameters : slope of the normal consolidation line	λ	0.13	[]
slope of the elastic swelling line	κ	0.125	[]
Critical state friction angle	ϕ'_c	21	[°]
Preconsolidation pressure	p'_c	6	[MPa]
Poisson's ratio	ν'	0.2	[]
Specific volume at 1 kPa on the NCL	N	2.75	[]
pore water data :			
porosity	n	0.39	[]
water bulk modulus	K_w	2	[GPa]

Working sequence

With respect to the working sequence, the following assumptions are made in the calculations (cfr. Figure II-2 for a plan view of the process) :

- ✓ The entire cross section of the gallery is excavated in one single step by semi-mechanical means (backhoe or roadheader type of machinery);
- ✓ The rounds are 0.5 m long and after each round, a 0.5 m wide ring is installed;
- ✓ The distance between the front and the segments varies within 0.25 and 0.75 m;
- ✓ The tunnel advance rate is about 2 m/day.

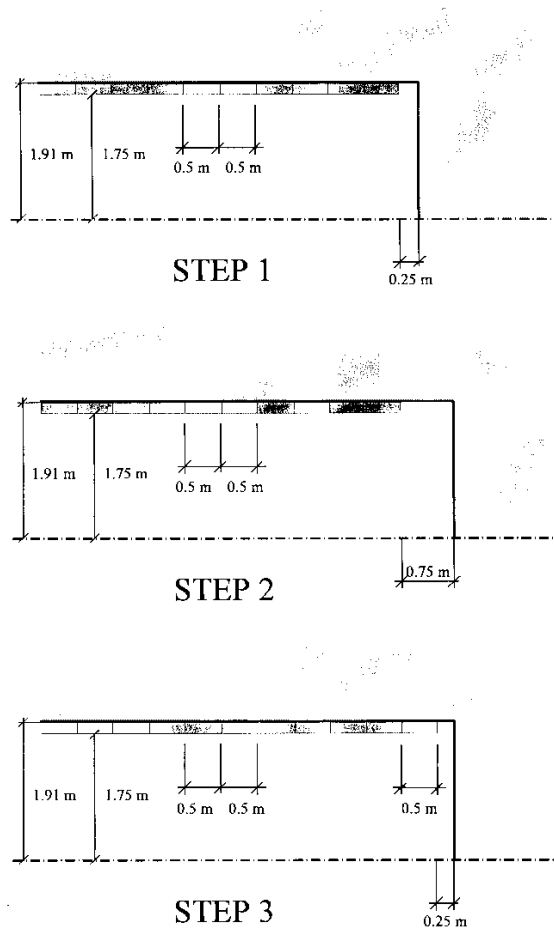


Figure II-2 : View of the sequential tunnelling and lining process

Complementary modeling assumptions

Since the CLIPEX instrumentation programme is mainly planned to assess the hydro-mechanical disturbance during the excavation of connecting gallery, this case study will consequently merely focus on the gallery construction and will not evaluate the hydro-mechanical changes arising from the pore pressure dissipation after excavation. Owing to the very low permeability of the Boom clay formation ($k \approx 4 \cdot 10^{-12}$ m/s), a 2 m/day advancing rate can be assumed as very fast, and consequently the response of the medium to the excavation can be idealized as undrained (non flow water).

The presence of some gap between the outer face of the lining and the excavation walls has a significant influence on the equilibrium state around the gallery, and more particularly on the magnitude of the hydro-mechanical disturbances. To appraise the importance of this factor, two calculations are carried out (for both constitutive models): one without any overexcavation gap and the other with an overgap of about 5 cm (figure II-3). The former situation could be theoretically achieved with special arrangement (like jacking points) that allow for an excavation of the ring against the excavation walls, but the later is probably more realistic. Indeed, it is more likely that some little gap will be present, at least owing to the uneven profile of the soil surface.

To model adequately this soil-lining interaction with a large-strain mode, the radius of the future excavation should be fixed in the initial mesh to respectively 1.95 and 2 meters. In the first case, the segments of the lining are placed directly against the wall (perfect contact) while in the second case, the grid will deform and come into contact with the liner several meters behind the face.

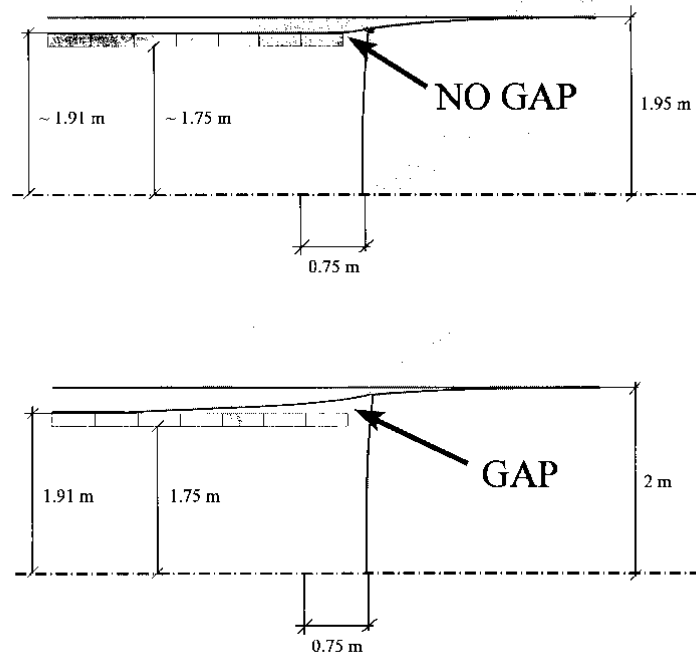


Figure II-3 : Schematic view of two configurations : no gap and a ± 5 cm of gap

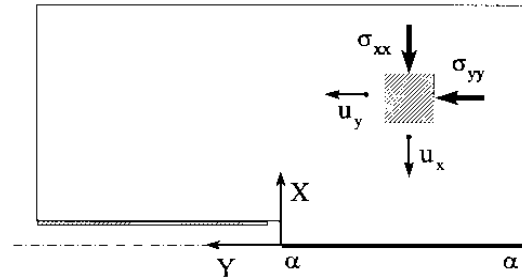
Results

For the three calculations, the results expected from the participants are the following:

αα

ahead of the tunnel face, distributions along the symmetry axis ($x=0$ and $-20 \text{ m} < y < 0 \text{ m}$) of :

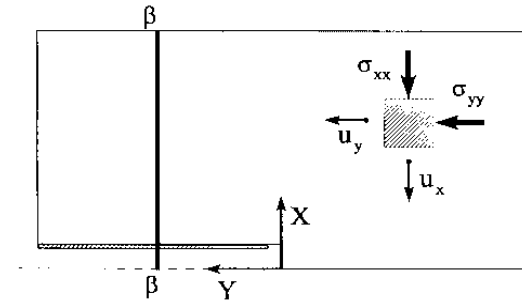
- ✓ Total stresses σ_{xx} and σ_{yy}
- ✓ Pore water pressure P_p
- ✓ (axial) displacement u_y



ββ

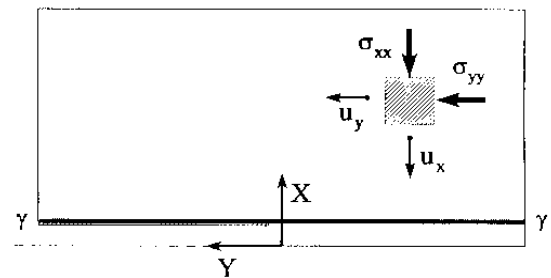
In across section 10 m behind the excavation front ($y=10 \text{ m}$ and $\approx 1.91 \text{ m} < x < 20 \text{ m}$), distributions in the caly mass of

- ✓ Total stresses σ_{xx} , σ_{yy} and σ_{zz}
- ✓ Pore water pressure P_p
- ✓ Radial component of displacement u_x



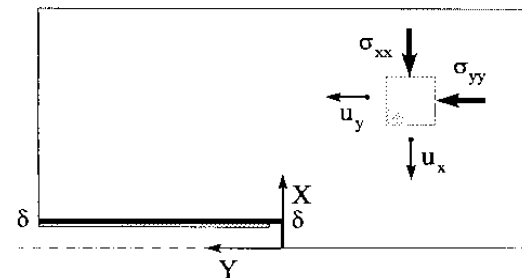
γγ

Profile of the radial component of displacement u_x at the tunnel circumference ($x=1.95 \text{ m}$ or 2 m depending on the case) as a function of the distance to the gallery face ($-20 \text{ m} < y < 20 \text{ m}$)



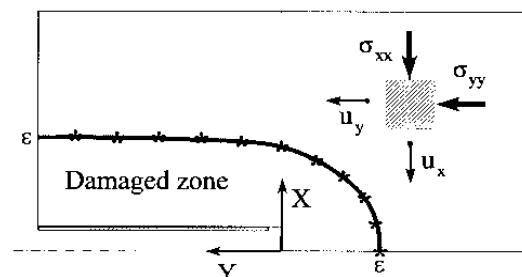
δδ

Profile of the total pressure (radial component of stress σ_{xx}) exerted by the soil on the lining as a function of the distance to the excavation front ($0 \text{ m} < y < 20 \text{ m}$)



εε

Damaged zone extend defined by X and Y coordinates of a few " boundary " points



Appendix 3

Configuration of the segmented rings

Ring 15, 30 and 50 have 10 instrumented segments according to the general plan (four 12-gauge segments, and six 6-gauge segments), as shown below in Figure III.1. Ring 75 contains two spare segments.

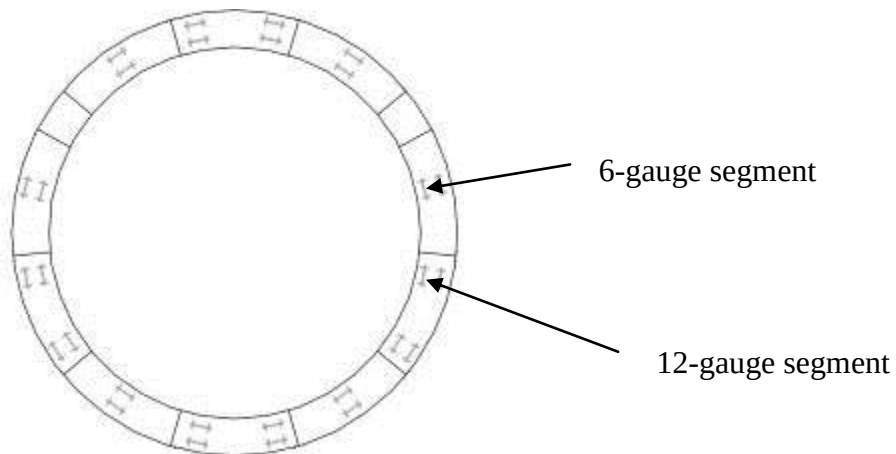


Figure III. 1 : general plan of an instrumented ring.

The configuration of a 6-gauge segment and a 12-gauge segment is shown in Figures III.2 and III.3, respectively.

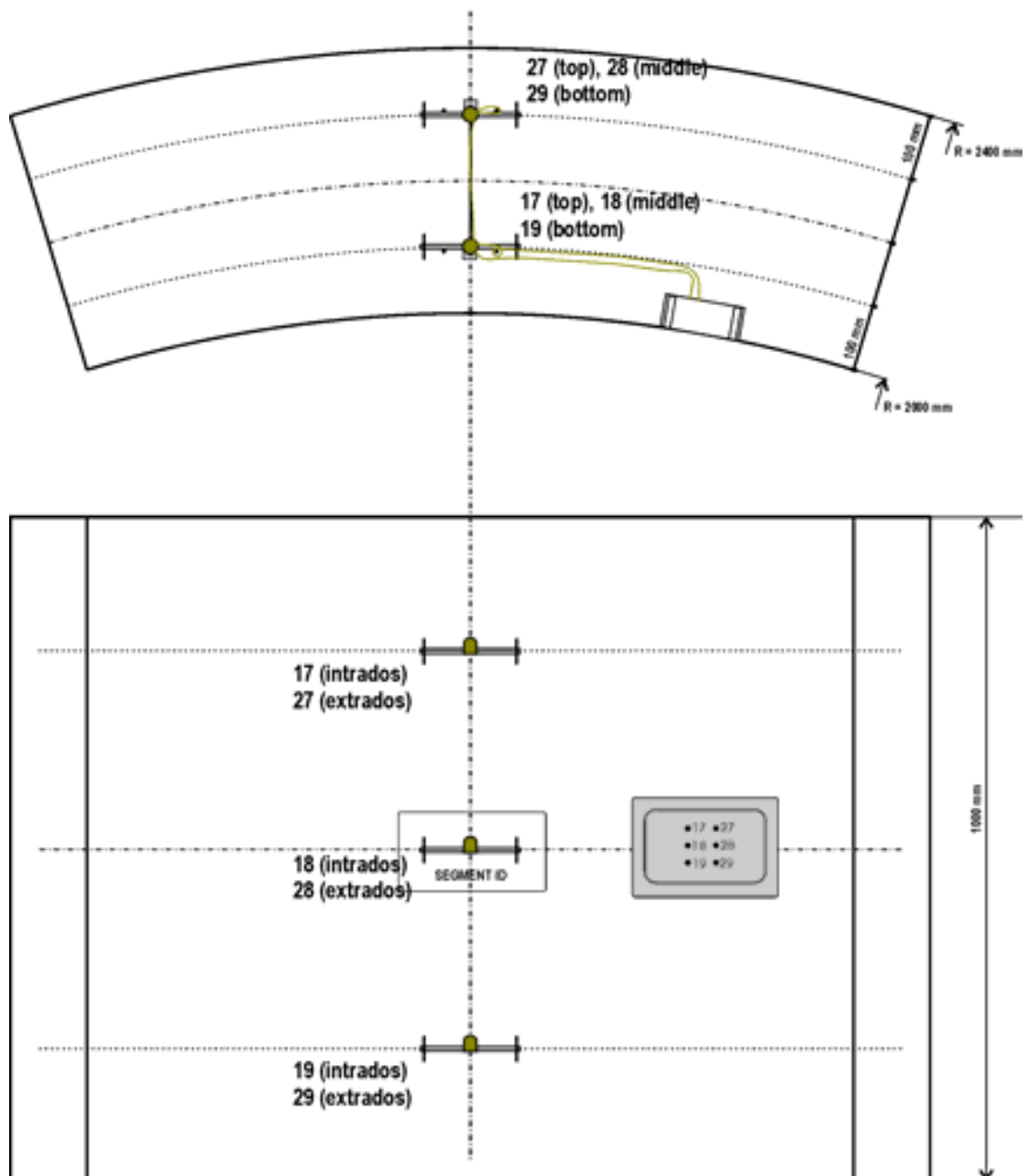


Figure III. 2: configuration of a 6-gauge segment

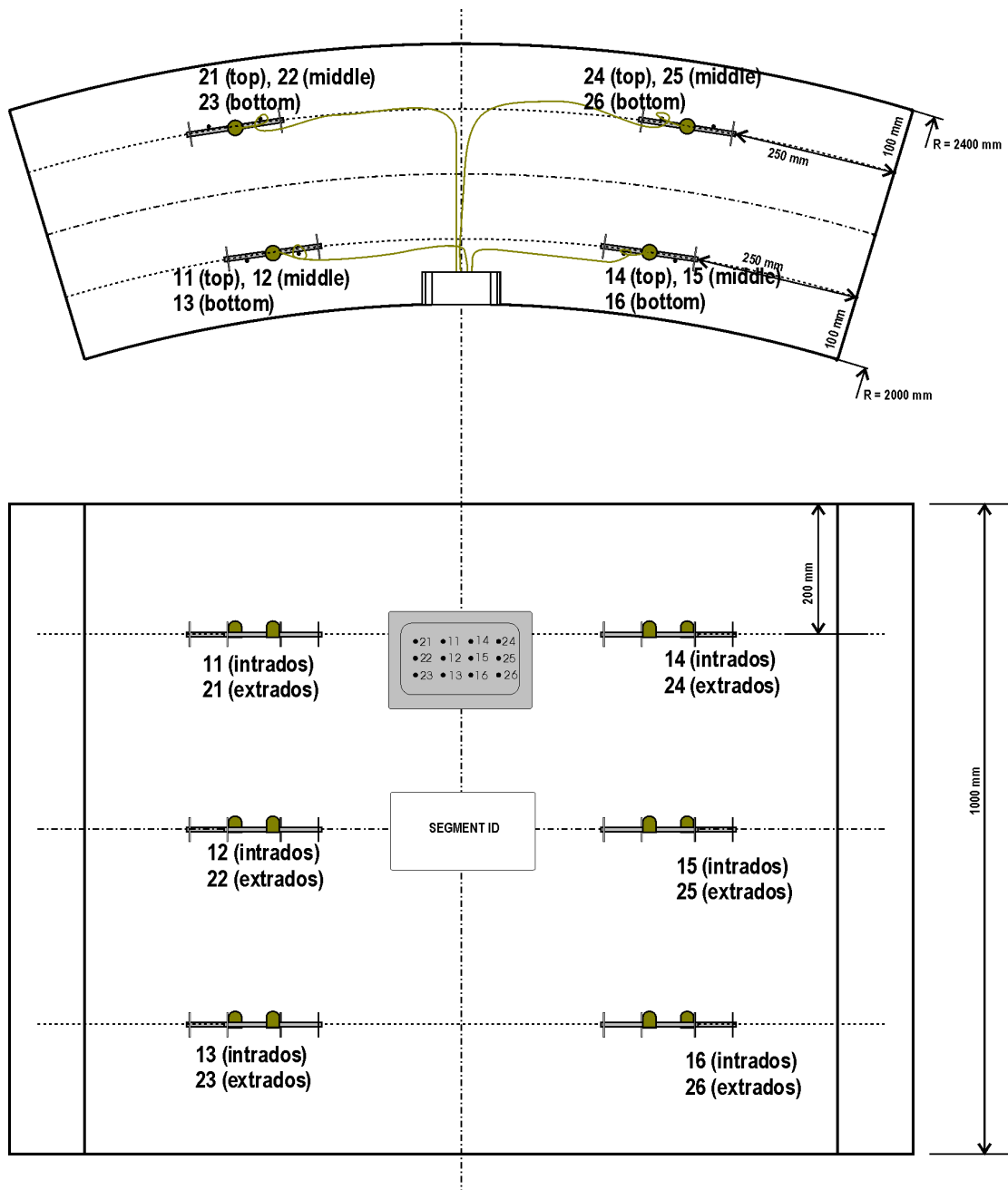


Figure III. 3: configuration of a 12-gauge segment

Figure III. 4 to 7 show the different instrumented segment rings with the individual identification of the segments.

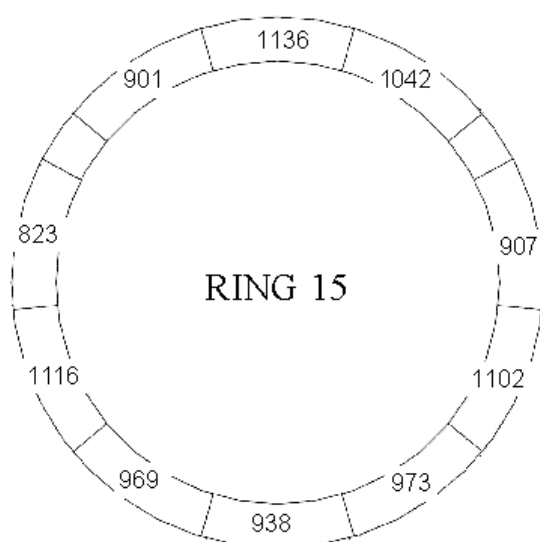


Figure III. 4: ring 15 instrumented segments

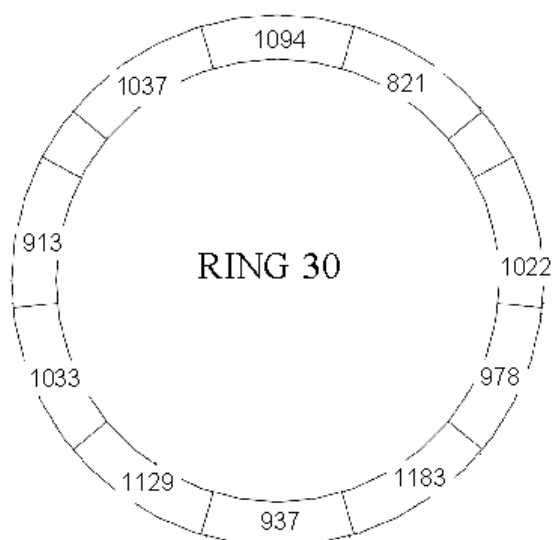


Figure III. 5: ring 30 instrumented segments

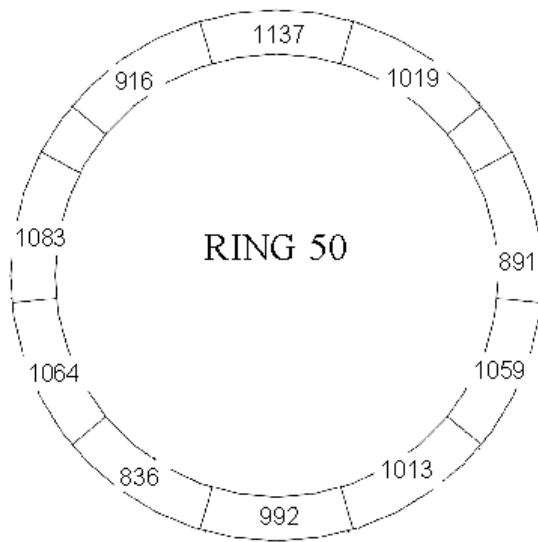


Figure III. 6: ring 50 instrumented segments

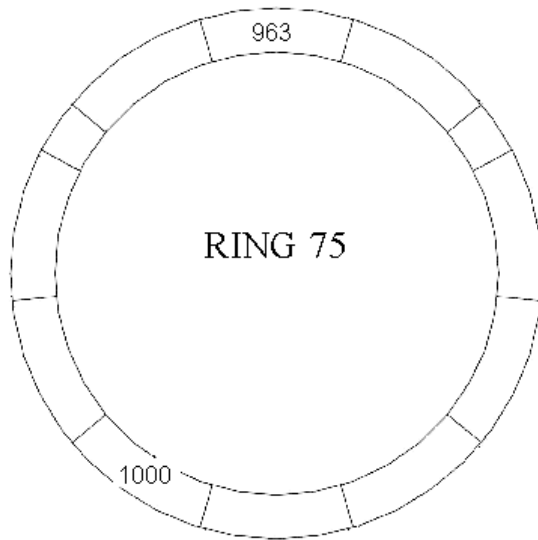


Figure III. 7: ring 75 instrumented segments

Appendix 4

Blind Prediction exercise of the Connecting Gallery

This note describes the initial data to be considered for the blind prediction exercise of the excavation of the connecting gallery. This exercise corresponds to the last topic of the activity 4.1 of the CLIPEX project.

Data

Location and geometry of the connecting gallery

The connecting gallery will join the second shaft (excavated 08/1997 – 05/1999) to the Test Drift gallery (excavated 10/1987 – 12/1987) as depicted on Figure 1.

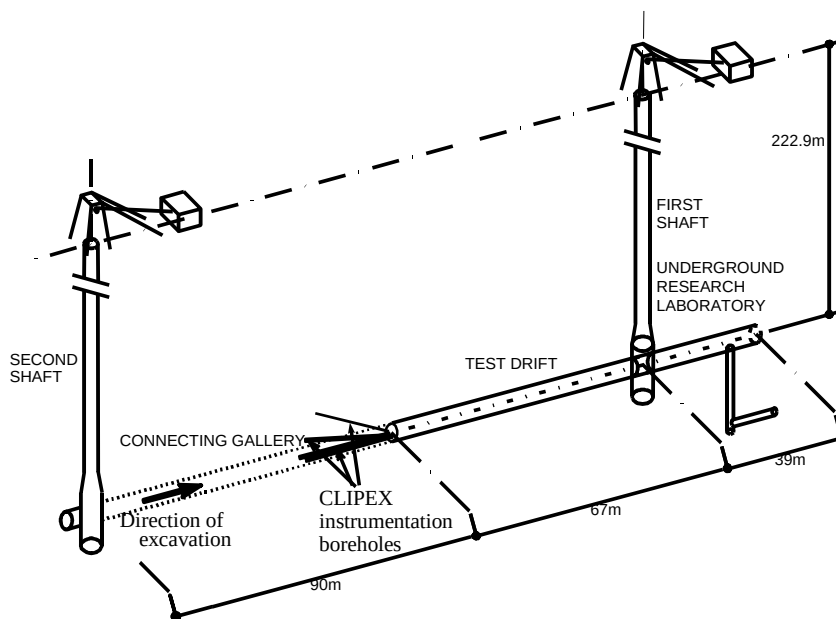


Figure 1. Representation of the co-ordinates systems used around the connecting gallery.

Host rock

0 m – 175 m : sands, silty sands

175 m – 270 m : Boom clay

The water table is at ground level, and the water pressure can be assumed constant at the roof of the Boom clay, i.e. $u_w = 1.75$ MPa at depth = 175 m below ground level.

Test-Drift gallery (as built)

Excavation period:	10/1987 – 12/1987
Excavation type:	manual
Mean excavation rate =	0.25 m/day
Depth of the axis =	222.9 m (below ground level)

Length =	67 m (from the axis of the first shaft to the TD front)
Excavated radius =	2.45 m – 2.55 m
Lining external radius =	2.40 m
thickness =	0.6 m
type:	concrete blocks
Maximum unsupported span =	1.000 m

Second shaft (as built)

Excavation period:	08/1998 – 04/1999
Excavation type:	manual
Mean excavation rate =	6 m/week
Max. depth =	233.0 m (below ground level)
Excavated radius =	see technical plan
Lining external radius =	see technical plan
thickness =	see technical plan
type:	sliding ribs + reinforced concrete

Connecting gallery (planned)

Excavation period:	? – ?
Excavation type:	semi-mechanised
Mean excavation rate =	2.0 m/day
Depth of the axis =	222.9 m (below ground level)
Length =	90 m (from the axis of the second shaft to the TD front)
Excavated radius =	2.420 m
Shield ext. radius (front) =	2.405 m
ext. radius (rear) =	2.400 m
length =	2.700 m
Lining external radius =	2.400 m (variation ± 10 mm)
thickness =	0.400 m
length of ring =	1.000 m
type:	concrete wedge block system
Young modulus E =	30 kN/mm ²
Max. unsupported span =	1.200 m
Min. unsupported span =	0.200 m

Rheological parameters for Boom clay

Each team is free to use its own mechanical model and parameters to represent the Boom clay behaviour. For this purpose, each modelling team should refer to the results obtained from the

in-situ characterisation programme carried out within the CLIPEX project (activities 3.1, 3.2 and 3.3), or to any other results that are available and that are considered relevant to the present modelling exercise.

Results required

Each participant must give the evolution of the parameters that will be recorded during the excavation of the connecting gallery. More precisely, it consists in predicting, as a function of the excavation front position, the evolution of:

- **the pore pressure u_w** at all the pore pressure transducers WAx, WBx, WCx and W Dx (the exact location of these sensors is given in [1]),

the total stress σ_{ij} (with mechanic sign convention: <0 for compression) at all the locations where a group of Kulite sensors (two at 90° or three at 120° depending on the configuration). These sensors are noted KAx, KBx, KCx, K Dx and their location is given in [1]. Let us point out that:

- the location of all the Kulite sensors always coincides with the location of pore pressure sensors,

- all the components of the tensor σ_{ij} must be given, and they must be expressed in the Cartesian frame (0,x,y,z) shown in figure 1, i.e. the z-axis is vertically upward oriented, the x-axis is oriented following the axis of the connecting gallery from the Test Drift towards the second shaft.

- **the displacement u_i** at locations corresponding to the measurement points EAx, IBx, ICx and IDx.

Although the different types of instruments that are emplaced (extensometer, inclinometer and deflectometer) do not measure the same component of the displacement field, it is asked the modelling teams to give all the components of the vector u_i at the considered locations, and they must be expressed either in the Cartesian frame (0,x,y,z) shown in figure 1.

References

[1] CLIPEX. Selection, adaptation and installation of the instruments. EC contract FI4W-CT96-0028, Topical report covering work package 2 – Ref. EIG PRACLAY/99-183