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Paper on "Underground opening in swelling rock"

by Giovanni Lombardi (Ph.D)

UNDERGROUND OPENINGS IN SWELLING ROCK

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1. INTRODUCTION

Swelling of rock masses have occurred in many underground works resulting in unforeseen incidents including heave of the invert, rupture of the lining, and complete collapse of the opening. (Bibliography 1)). Often the heave phenomenon was not completely understood by the engineers. It should therefore be of some interest to have an in-depth look at this problem.

Before entering directly into the subject of this paper, it will be useful to recall some general knowledge on two topics:

1. the swelling of a rock mass from the rock mechanics point of view, and
2. the general problem of the stability of underground openings using the "characteristic lines" to represent its fundamental aspects.

In fact, these two aspects of the question we are dealing with, are often misunderstood.

2. THE SWELLING OF A ROCK MASS

2.1 General.

By swelling we mean a volume increase of the rock mass which is not directly caused by a change of the state of stress in the rock, but which can be an indirect consequence of such change of state.*

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* Dilatancy is, on the contrary, a volume increase of the rock mass due to the change of the strain field beyond the limit of elasticity.

Such a definition corresponds to the following two phenomena:

1. the volume increase due to the chemical bonding of water with the rock as, for instance, the transformation of anhidrite into gypsum. (The problem of the solubility of these rocks in water is not dealt with here), and

2. the volume increase due to physical bonding of water with fine particles in the rock mass as, for instance, the fixing of water molecules on clayey minerals in the rock mass (for instance clays like smectites or montmorillonites are specially sensitive); this phenomenon is similar to the well-known swelling of expansive clays.

However, from our point of view, both phenomena may be considered in the same way. We are concerned by a volume increase of the rock mass by absorption of water.
We will now discuss the conditions which lead to this absorption and its effects on the behavior of underground openings.

2.2 Volume increase versus pressure. Swelling line.

Studing the behaviour of a sample of a swelling rock in an oedometer-like apparatus, one may readily determine the relationship between volume increase and confinement pressure acting on the sample; the so-called swelling line.

Figure 1 shows the principle of such a device. The lateral constraint to a cylindrical rock sample may be applied by a liquid pressure or by a very stiff steel ring avoiding any gap and any movement in the radial directions.

The swelling line shows maximum values both for the pressure and the volume increase (i.e. the expansion).

With higher confinement pressures than p_{max}, no expansion or swelling take place. At the other end of the line, the maximum value ε_{max} of the expansion is obtained in the case of no confinement pressure, this is the so called free swelling.

The normal procedure is to obtain the swelling line in steps by decreasing the pressure than by increasing it, since water is more easily absorbed by the sample than it can be "squeezed out" by the applied pressure. The question remains as to whether the swelling line is really a single line or whether it is more probably a hysteresis loop.

For our purposes, we will consider only the case of decreasing pressures, that is the case of destressing the rock sample or rock mass, since this condition occurs during the excavation of the cavity.
We obtain then the relationship:

$$\varepsilon = \varepsilon\ (p) \quad \text{with} \quad p \leq p_{max} \quad \text{and} \quad \varepsilon \leq \varepsilon_{max}$$

The condition for the swelling to reach these limits is that the required quantity of water is actually available for the swelling process.*

2.3 Time effect.

It is well known that the swelling process takes time to develop. In fact the water has to reach all particles of the rock mass which may be able to swell. It is clearly unterstood that this time will depend of the permeability of the rock, of the dimensions of the sample and of the magnitude of water pressure that is applied.

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* It shoud be noted that p_{max} is the external load acting on the sample which will prevent any swelling, not the maximum pressure which may develop in the interior of a more or less stiff testing device.

In a rock mass the conditions are even more complicated as the permeability is influenced by different discontinuities and the availability of water may be limited in some cases due to the very long path it has to travel to reach the rock mass. It is therefore of great importance to know whether the rock mass is above or below the ground water table or whether the zone of interest is protected from water, for instance, by some impervious layer.

In most cases it will not be possible to predict the time a rock mass needs to swell merely on the basis of the laboratory swelling tests.
Nevertheless we may write:

$$\varepsilon = \varepsilon \quad (p, t)$$

as a formula expressing the general behaviour of the swelling rock mass.

2.4 Conditions for swell.

Summarizing the foregoing, we may state the conditions for the swelling process to develop.
These are:

- the capability of the rock to swell,
- the destressing of the rock mass,
- the availability of a sufficient quantity of water and
- enough time for the process to take place.*

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* Cases are known where swelling in tunnels is still in progress after a century.

2.5 Swelling as a geological phenomenon.

Supposing a geological situation like the one shown in the Figure 2, one may observe that rock samples taken from locations A and B of the same rock layer will react differently in a swelling test. The two swelling lines are also indicated in the same figure.

Performing a series of tests, one may also note that the maximum swelling pressure corresponds many times to the existing overburden load at the place where the sample was obtained.

This is true provided that:

- enough time has elapsed since the previous change of the stress state, which in geological processes is generally the case* and
- enough water was available, which depends on the climate, but which is often the case.

Therefore as a rule we have:

$$p_{max} \cong p_{nat}$$

The natural pressure, p_{nat}, corresponds more or less to the overburden load.
Of course, a limitation of p_{max} has to exist.

$$p_{max} \leq p_{abs} \qquad \text{, where}$$

p_{abs} is the pressure at which the rock is absolutely dry or free of absorbed water and which will not allow any swelling. The value depends obviously on the nature of the swelling particles.

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* except, e.g., the case of the rock underlying a recent big land slide.

The normal relationship between the maximum swelling pressure P_{max} and the depth is represented in the Figure 3 where geological and man-made destressing of the rock are also shown.*
This relationship shows that the swelling pressure may reach the very high values of the overburden stresses.
In underground works this phenomenon has to be studied and protective measures taken to avoid problems. This relationship confirms also that at any outcrop the rock will have lost its swelling potential. Geological investigations limited to the surface of ground may therefore be misleading in evaluating the swelling problems of underground works.
We may now understand, that the swelling potential is in fact the residual of the original swelling potential of the rock mass after the geological swelling had occured.

We already mentioned that the limit P_{abs} of the swelling potential is given by the mineralogical composition of the swelling elements. The intensity of the swelling itself, that is the volume increase, is of course a function of the content of swelling elements in the rock mass. The higher the percentage of montmorillonite in a marl, the higher will be the volume increase of the sample under the same confining pressure.
For a complete analysis the anisotropy of the swelling process in some rocks should be considered. It will be disregarded here for simplicity.

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* Destressing may be understood as a reduction of the overburden, since we are disregarding the anisotropy of the stress field.

3. EQUILIBRIUM OF AN UNDERGROUND CAVITY

3.1 General considerations.

In a engineering structure the live loads to be supported are known in advance and the structure can be designed adequately. Generally, an elastic behaviour of the structure may also be assumed.

On the contrary the loads to be taken by the tunnel supports or by the lining cannot be defined beforehand but are the result of the construction sequence. They are the result of conditions of compatibility between the rock mass and the support system related to the acting forces and the resulting convergence movements. (See Bibliography 2), 3), 4).) Furthermore the stresses are often beyond the limit of elasticity.

The overall behaviour of an underground cavity can best be explained using the so called "characteristic lines", which are not really a method of computation but a very powerful tool to represent the results of any computation and to disclose the existing fundamental relationships.
The caracteristic line of the cavity itself is a representation of the relation: convergence movements versus confining pressure.

Figure 4 shows the simplest possible case of a circular cavity in a homogeneous natural stress field σ_{nat}, with a supporting or confining pressure uniformly distributed around the entire boundary of the cavity.

At the beginning the confining pressure p is supposed to equal the natural stress component at the boundary. By definition the convergence at that moment is nil.
The pressure p is then progressively released. The convergence movement increases obviously following the characteristic line.

The first section AB of this line is, as a rule, a straight segment which corresponds to an elastic behaviour of the rock mass for a moderate change of the stress field. The linear extension of this segment would intersect the vertical axis at the point E which is the "elastic solution for the unsupported cavity".
However, the elasto-plastic behaviour of the rock mass beyond the limit of elasticity makes the "characteristic line" deflect to higher movements according to the curved line BC. If this line cuts the vertical axis, the unsupported cavity may be considered stable; if not, the cavity is unstable, unless a sufficient confinement pressure P_S is applied by some kind of support or lining.
It is apparent that the shape of the characteristic line depends on:

- the geometry of the problem,
- the natural stress field before excavating the tunnel (Point A),
- the stiffness of the rock mass (Modulus of elasticity or deformation), (Section A-B),
- and the strength properties of the rock mass (limit of elasticity, plasticity, rheology). (Section B-C).

It is of the upmost importance to recognize that each point of the characteristic line represents a point of possible equilibrium of the cavity.
The art of the tunneling engineer consists of selecting the best suited equilibrium among all the possible points in order to solve his particular problem.

3.2 The construction procedure.

The construction of a tunnel is not a two-dimensional problem, as has been considered up to this point, but a three-dimensional one. We therefore need to examine Figure 5 which represents a longitudinal section of a tunnel under excavation.

At some moment the face is at position (2) and the convergence movements up to that moment are represented by the line I-I.
After the excavation of the rock mass M, the face is displaced from position (2) to position (2') and line I-I is shifted to II-II. This implies an additional inward movement or an additional convergence Δ in a given section A-A located near the face.
Any support or lining already placed and structurally effective in section A-A will be clamped in and additionally loaded.

The history of the convergence at section A-A with the distance from the face is represented by Figure 6.
The support placed in B with a gap "a" will be effective at C.
The convergence "s" of the supported section is, of course, smaller than it would be when unsupported "u".
The load acting on the support corresponds to its deformation "b", while "c" is the movement of the rock mass prevented by the support.
It is easily seen that the final load acting on the support depends on the distance from the face at which the support is placed and on its rigidity. Depending of these two factors, Figure 7 shows how different points of the characteristic line of the cavity, that is, different points of equilibrium, may be reached.

3.3 Time effect.

In the foregoing discussion the time effect has been omitted.
In case of swelling rocks this cannot be done, since swelling is obviously a time dependent phenomenon.
Figure 8 shows how, even with a constant confining pressure, the movement of convergence increases, provided that the conditions for swelling are fulfilled.
For each successive time interval since the excavation, there is a different characteristic line with successively greater convergences.
(This phenomenon has some similarity with the viscosity of the rock mass.)

Figure 9 indicates the manner in which final equilibrium is reached depending of the type of support used.
A softening support will lead to higher deformations but to a smaller load acting up on it.

3.4 Stress distribution and swelling.

The excavation of a cavity modifies the natural or primary stress field in the ground. Figure 10 indicates a possible secondary stress field around the cavity.
Each point of the rock mass is subjected to a different change of stress. The "destressing" of the rock is also different from point to point. The same is the case for the specific swelling made possible by the variation of the stresses as shown in the same figure. It can be seen that the swelling takes place mostly at the vicinity of the cavity.

4. SOME EXAMPLES

4.1 Buchbergtunnel.

The Buchberg highway tunnel in Switzerland was built 1974 in a marly sandstone formation. The marly elements in this formation are able to swell. Immediately after the excavation the cavity was stable by itself with very small convergencies (about 1 cm). See Figure 11b).
However the geomechanical investigations determined that a potential swelling exists corresponding to the swelling line shown in Figure 11a). The maximum swelling pressure of 150 t/m^2 corresponds approximately to the overburden of 60 m at the place where the samples were taken.
The maximum swelling potential reached about 2% volume increase.

The characteristic line at infinite time shows a maximum convergence of about 10 cm in the case of no support.
A stiff support like the initially foreseen concrete ring would have been loaded with 50 t/m^2. This load would overstress the invert to failure. The solution of the problem was found in proposing a "softening" lining. Below the invert a layer of polyurethane foam was placed. (See Figure 12.) By this means, in case of swelling, the entire arch rests on the paraments which load the rock to a higher pressure than the natural stress.
Therefore no swelling can take place at this points which may hence be considered as fix. A well defined rock mass below the invert is allowed to swell compressing the foam layer, which reacts with a smaller pressure than in the case of a stiff invert.

The characteristic line of the entire soft lining is also shown in Figure 11b), leading to a maximum pressure of 30 t/m^2 for which the swelling is about 0.9%.
The "softening" of the lining permits a reduction of load on the invert due to the swelling of the rock from 50 to 30 t/m^2. The rock mass below the invert is then loaded by 30 t/m^2. It may swell to 0.9% accordingly to the swelling line of Figure 11a). This swelling produces a heave of about 4 cm which is absorbed by the crushing of the foam plates avoiding any overloading of the concrete structure.

4.2 Seelisbergtunnel.

A very similar case is that of the few hundred meter long section of the Seelisberg tunnel along which the Valanginien marl formations were to be crossed.
After having lined the very stable bore and placed a flat lean concrete floor, a heave of the floor was observed. See Figure 13a).

Interesting is the fact that, due probably to the lowering of the water table and also to some vapor diffusion through the lining, the walls of the tunnel deformed outward.

The marl section represents only a short part of this 9 km long highway tunnel. It was therefore difficult to build a deep curved invert like the one which was foreseen from the beginning at the Buechbergtunnel.
The solution of the problem was found in the following way.
Below the sidewalks two longitudinal concrete beams were placed and anchored to the ground.
Their purpose was to build up fix points on each side of the highway lanes. A concrete slab carrying the lanes bridges over the floor of the tunnel, allowing a certain volume of rock to swell freely into a space below large enough to take the maximum possible swelling volume.
Below the swelling rock a rock invert develops by itself, being confined by the anchors and the weight of the swelled rock mass.

Due to some methane gas emanations from the rock, the space below the slab had to be ventilated.
This tunnel section has been in operation for about 7 years and appears to be functioning very satisfactorily.

4.3 The underground facilities at the CERN in Geneva.

The European Research Center in Nuclear Physics (CERN) located at the Swiss-French border near Geneva owns and operates different underground facilities. (See Bibliography 5).)

Figure 14 represents the situation of the SPS (Super Proton Synchroton) an approximately circular 8 km long horizontal bore of 4.8 m diameter with 6 long straight sections (LSS) and 6 bends.
It is located in the Sandstone Marl formations of the Swiss plateau and was built in the 1970's. Some of these formations are susceptible to swelling.

The cross section of the main tunnel as shown in Figure 15, is a circular one, having been excavated by a full face Tunnel Boring Machine.
The lining composed of a ring of precast segments and an inner placed concrete ring with a total thickness of 30 cm, is stiff and strong enough to support any possible swelling pressure at this site.

No problems have arisen since the construction in the main ring itself, but in the inclined transfer tunnels (TT10, TT20, TT60) of horseshoe sections with a flat floor the invert was lifted at different places where the swelling marl layers were crossed.

Some years later two underground experimental chambers were excavated across the SPS main tunnel at the locations LSS4 and LSS5 in order to perform Proton-Antiproton collision experiments. At LSS4 a 21,4 m diameter, 43,5 m long cavern was excavated; the bottom of which lies 63 m below the ground surface.
A solution similar to that of the Buechberg tunnel was designed using 6 cm thick polyurethane foam plates placed on a lean concrete layer below the 3 m thick foundation slab (Figure 16).

An unusual solution was selected for the LSS5 Experimental station, as shown in Figure 17. At this location the ground surface is lower than at LSS4, and since a layer of 8 m of moraine covers the rock, it was obvious that the only suitable way to build the chamber was the cut and cover method. Two shafts of 20 m inner diameter were sunk. The access shaft is located below a laboratory building. The second shaft is covered by a dome made of precast concrete elements acting as formwork and by an exterior concrete vault. An earth mass on the top of this dome acts as additional protection against radiations.

The two shafts are connected through a concrete structure where a removable protection wall of concrete blocks can be placed.
This wall allows work to proced in the access shaft while the SPS is in operation.
An experimental machine of 1000 t weight has to be moved from one shaft to the other as required by the experiments. Only very small movements of the entire structure can be accepted. This means that a stiff structure must be designed.

The geological log shows (Figure 17) that different layers are susceptible to swelling. The upper layers, at elevation -10 and -24 m, when swelling will press in a horizontal direction on both shafts and the connecting structure. These are strong enough to withstand easily the acting pressure which is relatively small since the depths of the layers are not great.

On the other hand, various swelling layers between -32 and -50 m with a total thickness of 12 m could produce a heave of the floor of about 150 mm if they were allowed to swell freely.

The swelling capacities of the different layers were carefuly tested. Some results are reported in Figure 18.
To avoid the heave completely a pressure equal to the weight of the excaveted rock should be applied to the bottom of the structure. Obviously the weight of the structure itself is not large enough even if some weight of the surrounding rock mass is added by a protruding edge of the foundation slab (Figure 19).

Once again, a solution was found by concentrating the entire available load on a narrow strip so that the pressure exceeds the former natural stress in order to avoid any heave at this point and to avoid any movement of the entire structure. Below the center of the foundation slab some swelling is allowed to occur.

The foam will start to be crushed and to react opposing a pressure to the swelling, which will slow down until an equilibrium is reached.

Figure 20 shows how the equilibrium point is defined by mean of the characteristic lines.
At the equilibrium point F the total heave of the rock is 50 mm which is compensated by a crushing of the foam of 40 mm and a deflection of the slab of 10 mm. The acting pressure at that moment is about 22 t/m^2.
The final swelling of the rock is only 27% of the free swelling (40 mm instead of 150 mm). The load is only 44% (22 t/m^2 instead of 50 t/m^2) of the value for the case of a slab with no foams (Point S) or 22% (22 t/m^2 instead of 100 t/m^2) of the value which would be occur for the case of an absolutely stiff foundation plate (Point T).

It seems that the compromise obtained between a very stiff and heavily loaded structure and a very flexible structure with large heave movements and very small acting load, is an adequate solution of this specific problem.*

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* Presently a new 26,5 km long accelerator ring called LEP is under construction at the CERN mostly under the same geological conditions.

5. CONCLUSIONS

The swelling of some rock formations may be the cause of difficult problems and sometimes of costly accidents in underground works. (See Bibliography 1)).

Knowledge of the laws of swelling, accurate investigations and testing, some design ingenuity and adequate computations make it possible to find technically and economically interesting solutions.

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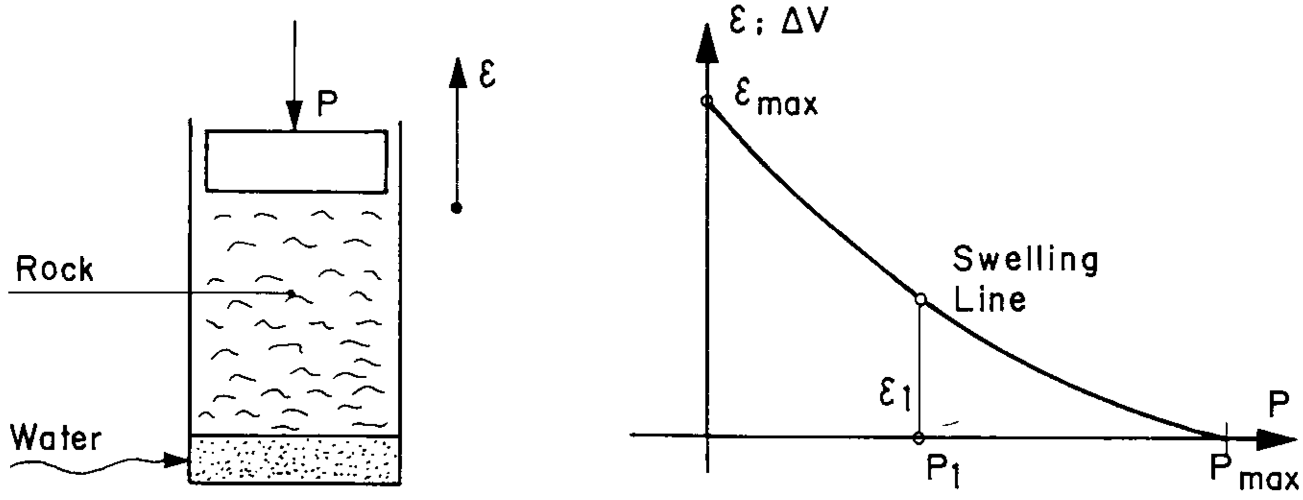
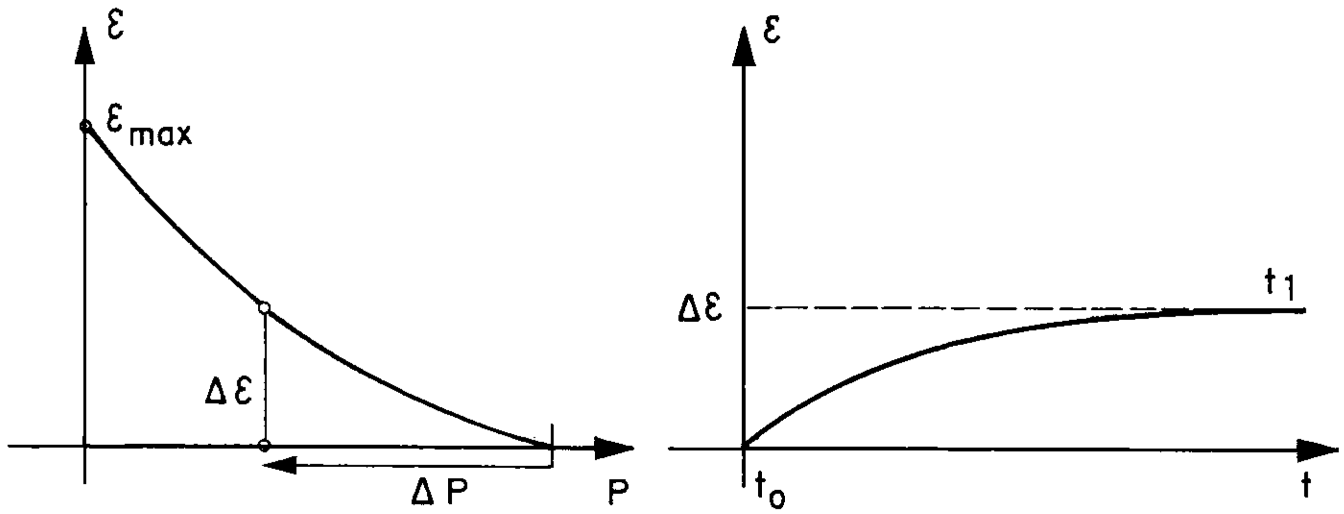
Locarno, December 1984/May 1985 G. Lombardi Ph.D.



a) Total volume increase "ΔV" or expansion "ε" vs confinement pressure "P"



b) Time effect

Figure 1

Swelling of a rock sample in an oedometer like apparatus.

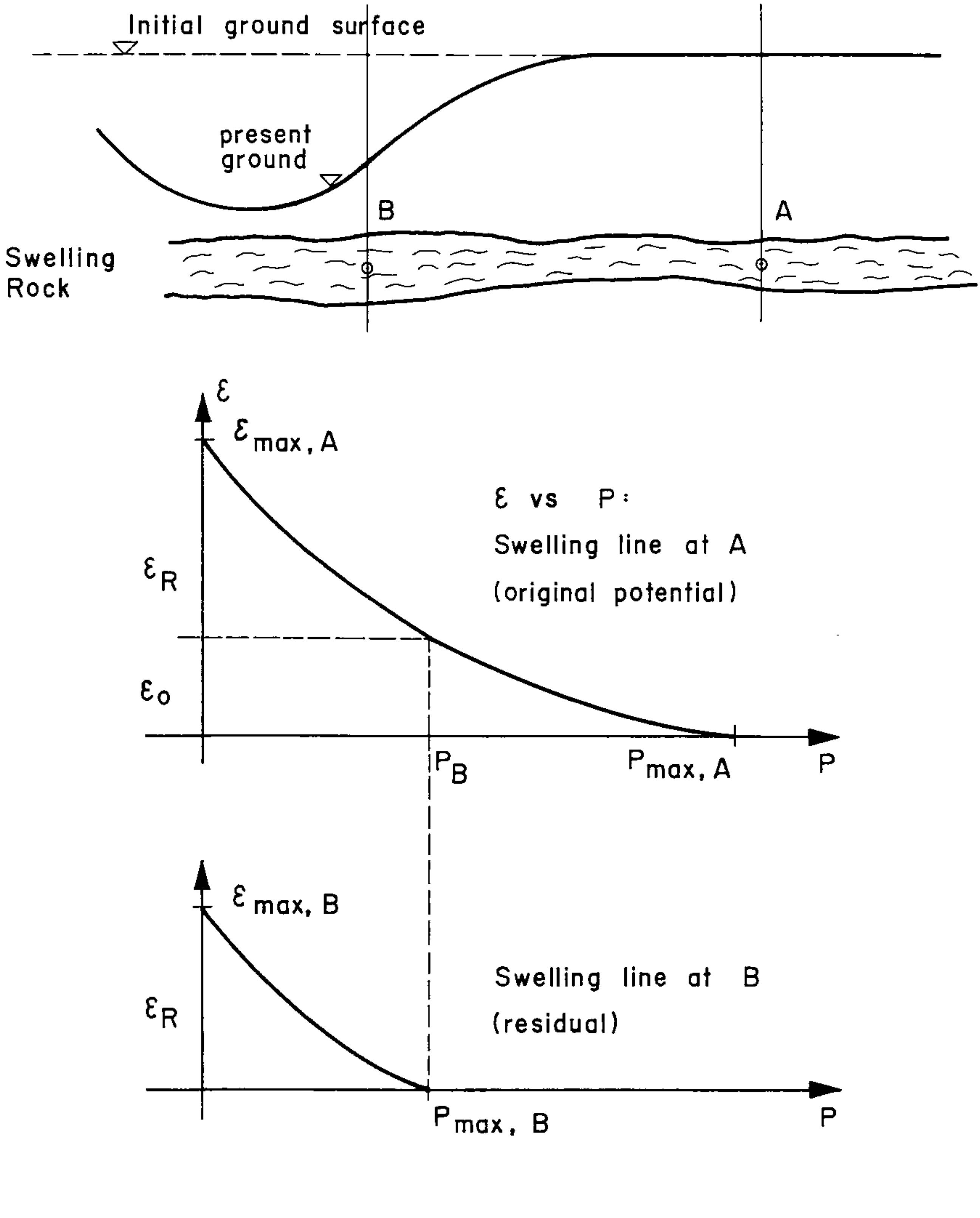
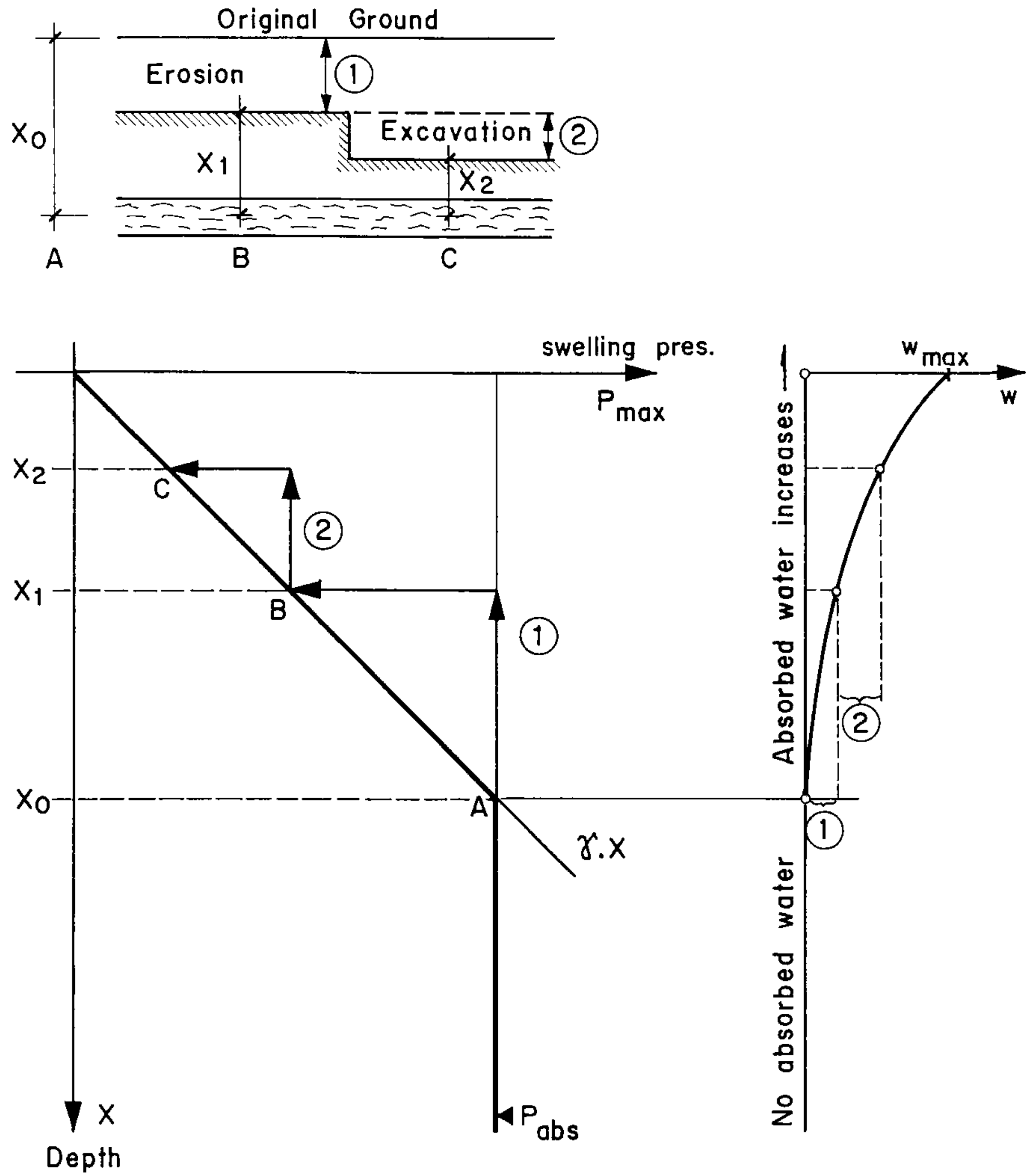


Figure 2

Geological Swelling



1 Geological destressing and swelling

2 Man-made destressing and swelling

Figure 3

P max vs depth x

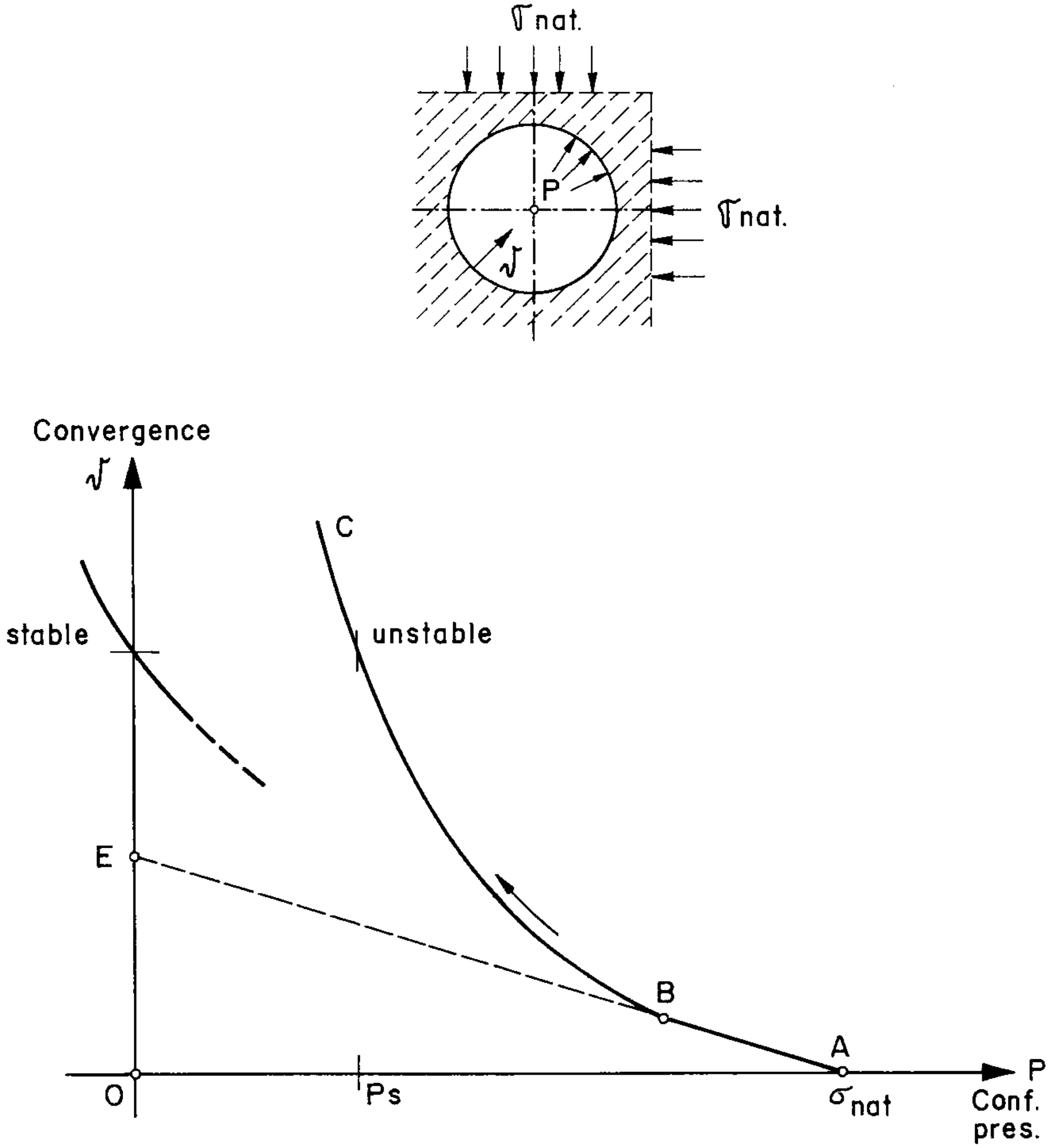


Figure 4

Convergence vs Confining pressure
= Characteristic line

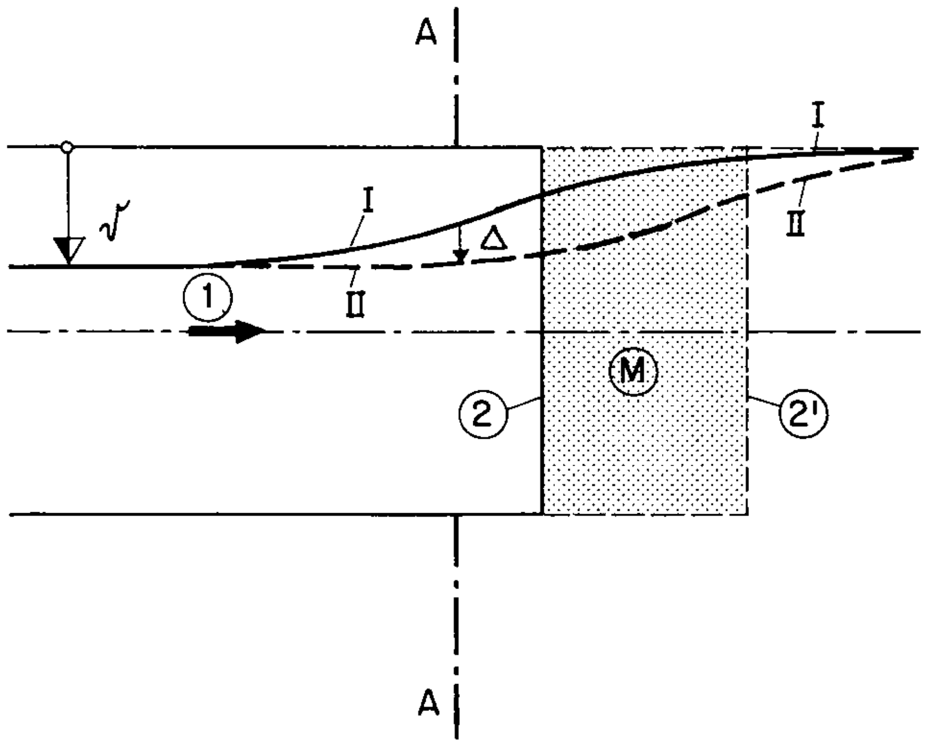


Figure 5

Deformations during the excavation process

1 tunnel axis

2 face

2' face after the excavation of M

A-A fix section

√ convergence

I-I convergence for face 2

II-II convergence for face 2'

Δ change in √ due to the advance from 2 to 2'

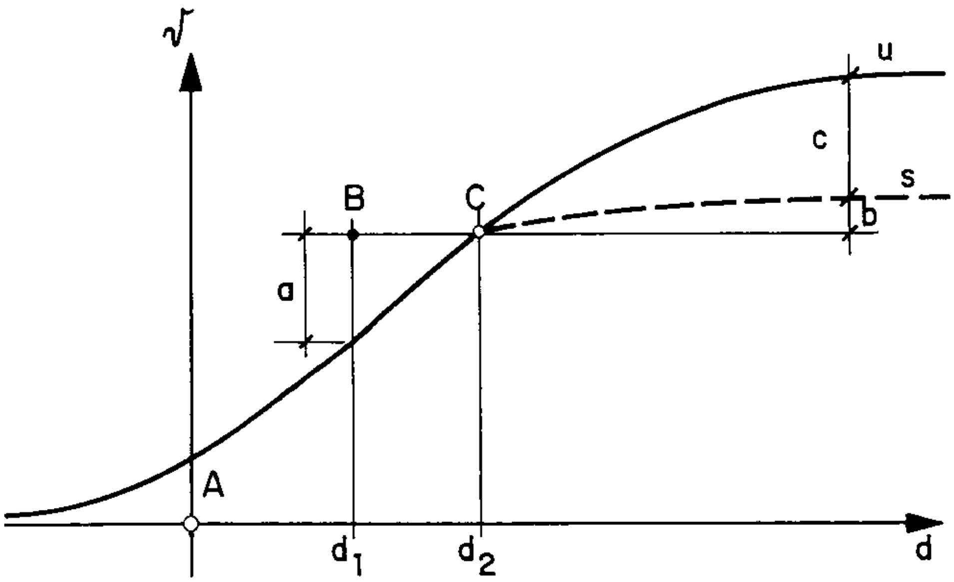


Figure 6

Convergence as function of the distance from the face

d distance from the face

√ convergence

B support placed at d_1 from the face with a gap "a") support effective at d_2 (Point C)

u convergence of the unsupported tunnel

s same for the supported tunnel

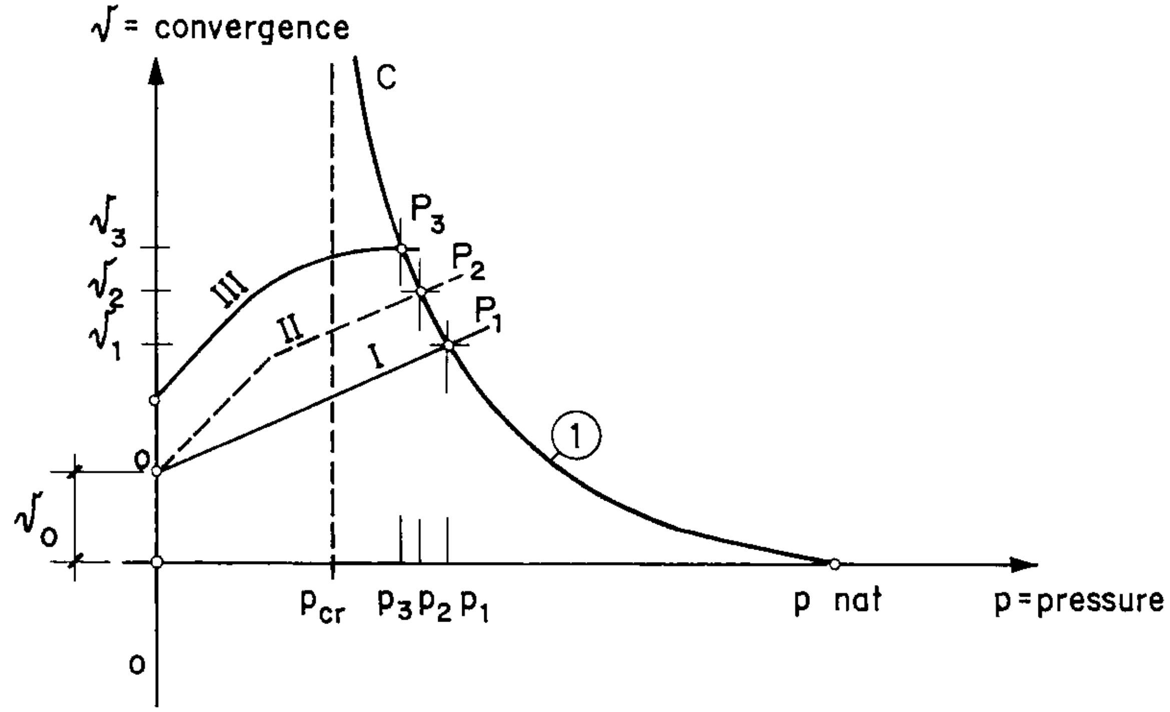


Figure 7

Characteristic lines of the cavity and of various supports.

1	char. line of the cavity
I, II, III	char. line of the supports
P_1, P_2, P_3	points of equilibrium with final loads p_1, p_2, p_3 and final convergence $\surd_1, \surd_2, \surd_3$.
$\surd_0$	convergence of the cavity until the support is placed
p_{cr}	critical pressure ; collapse of the cavity

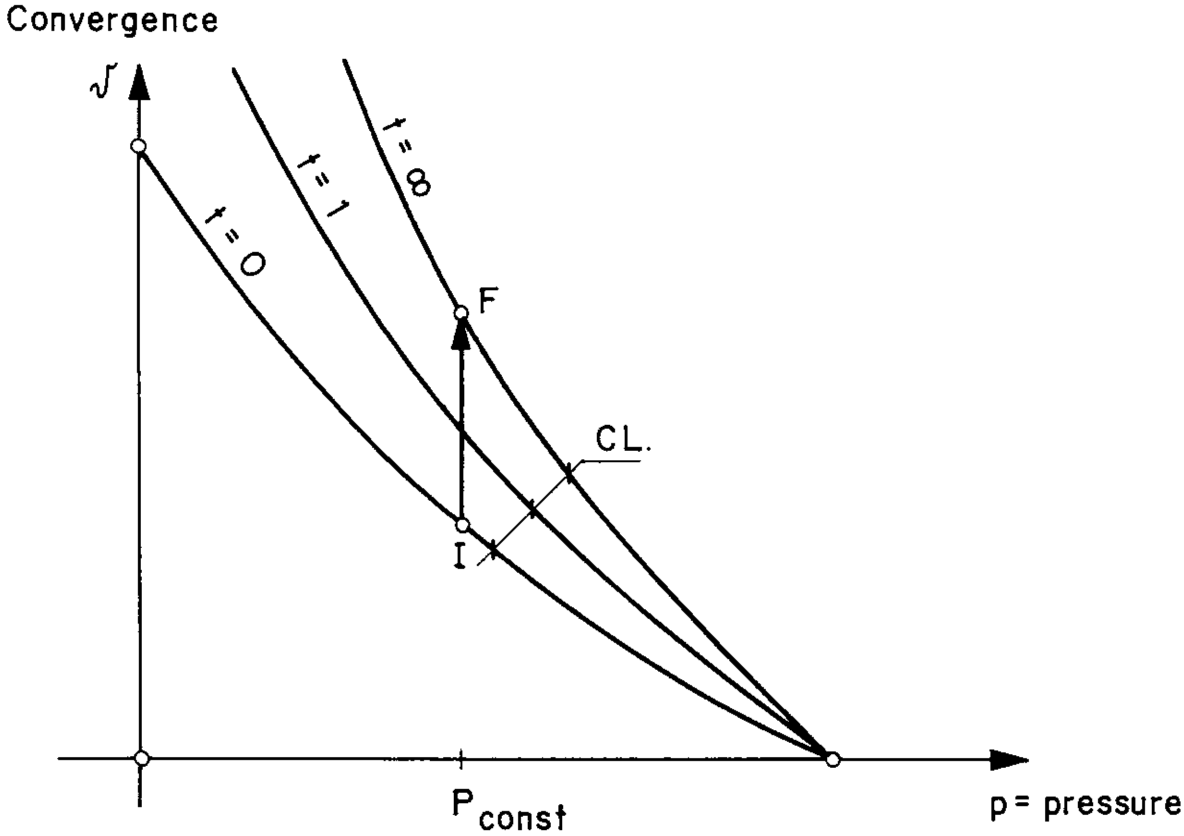


Figure 8

Viscosity or Swelling and characteristic lines (C.L.) at different moments of time.

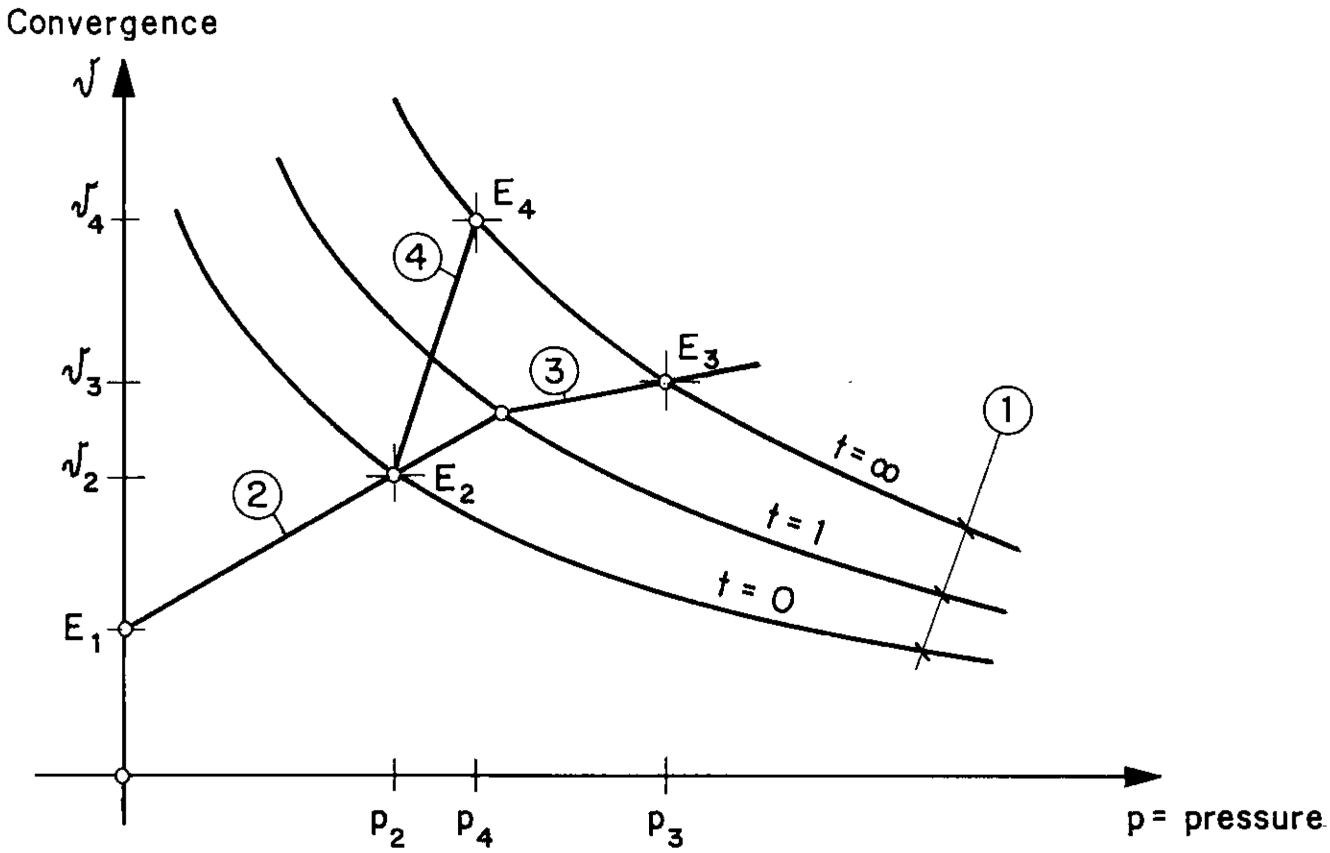


Figure 9

Time effect and Characteristic Lines

(1) characteristic lines of the cavity at various moments

(2) characteristic line of the support at the beginning

(3) case of the support getting stiffer with time (e.g. NATM) or with deformation

(4) case of the support softening with increasing deformation or time

E_2 equilibrium shortly after excavation

E_3, E_4 final equilibrium depending on the type of support

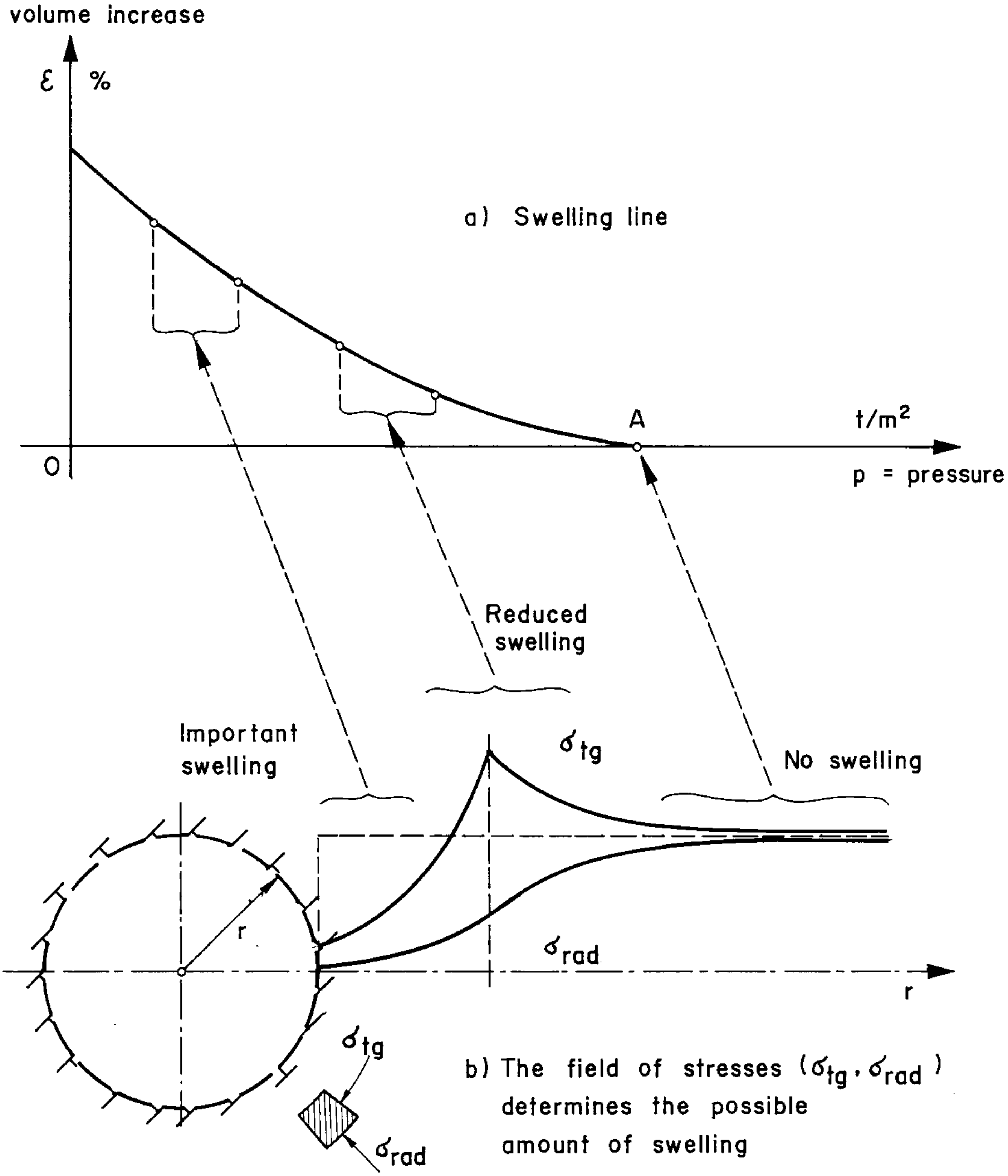


Figure 10

Swelling at various points

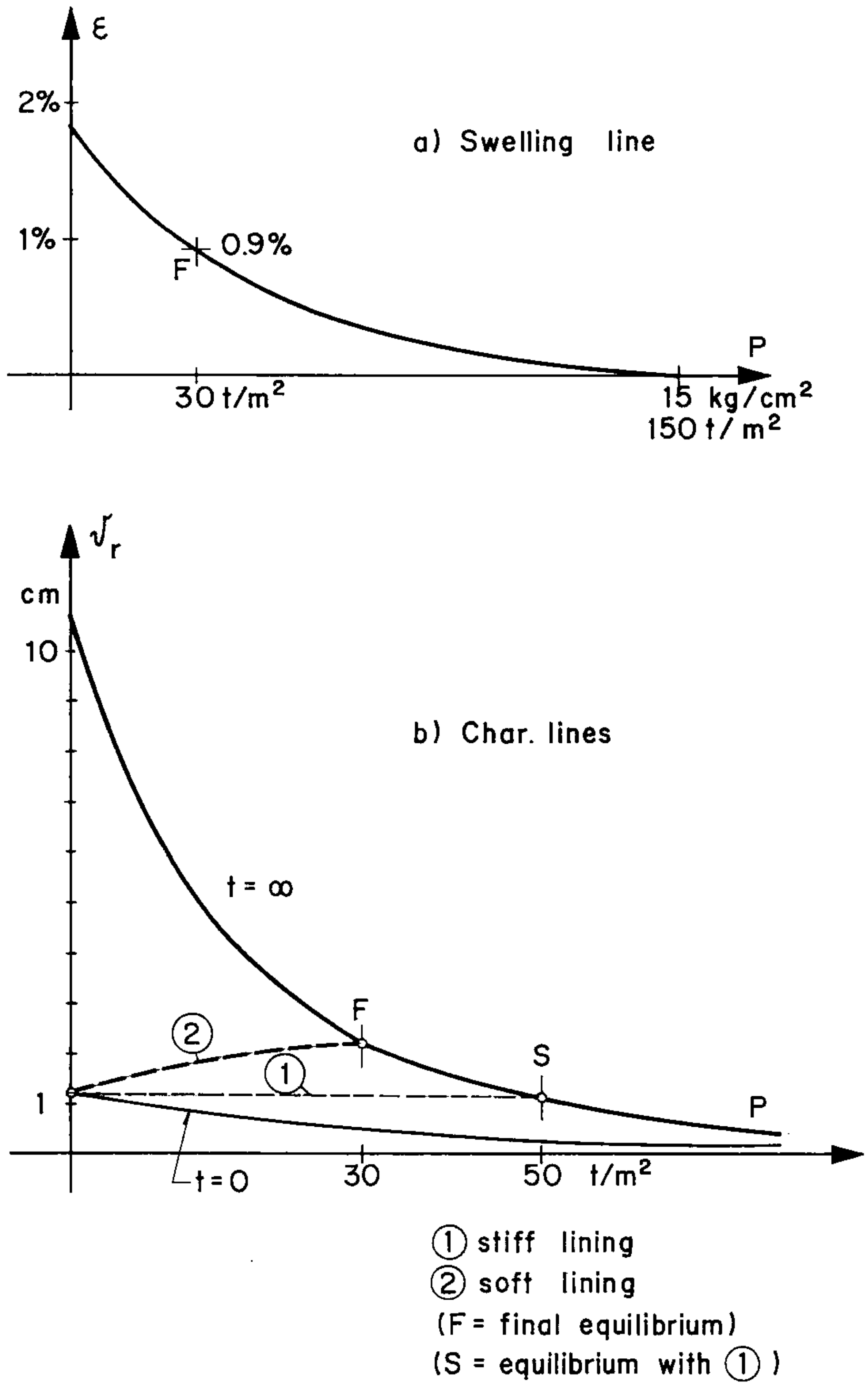


Figure 11

Buechberg tunnel

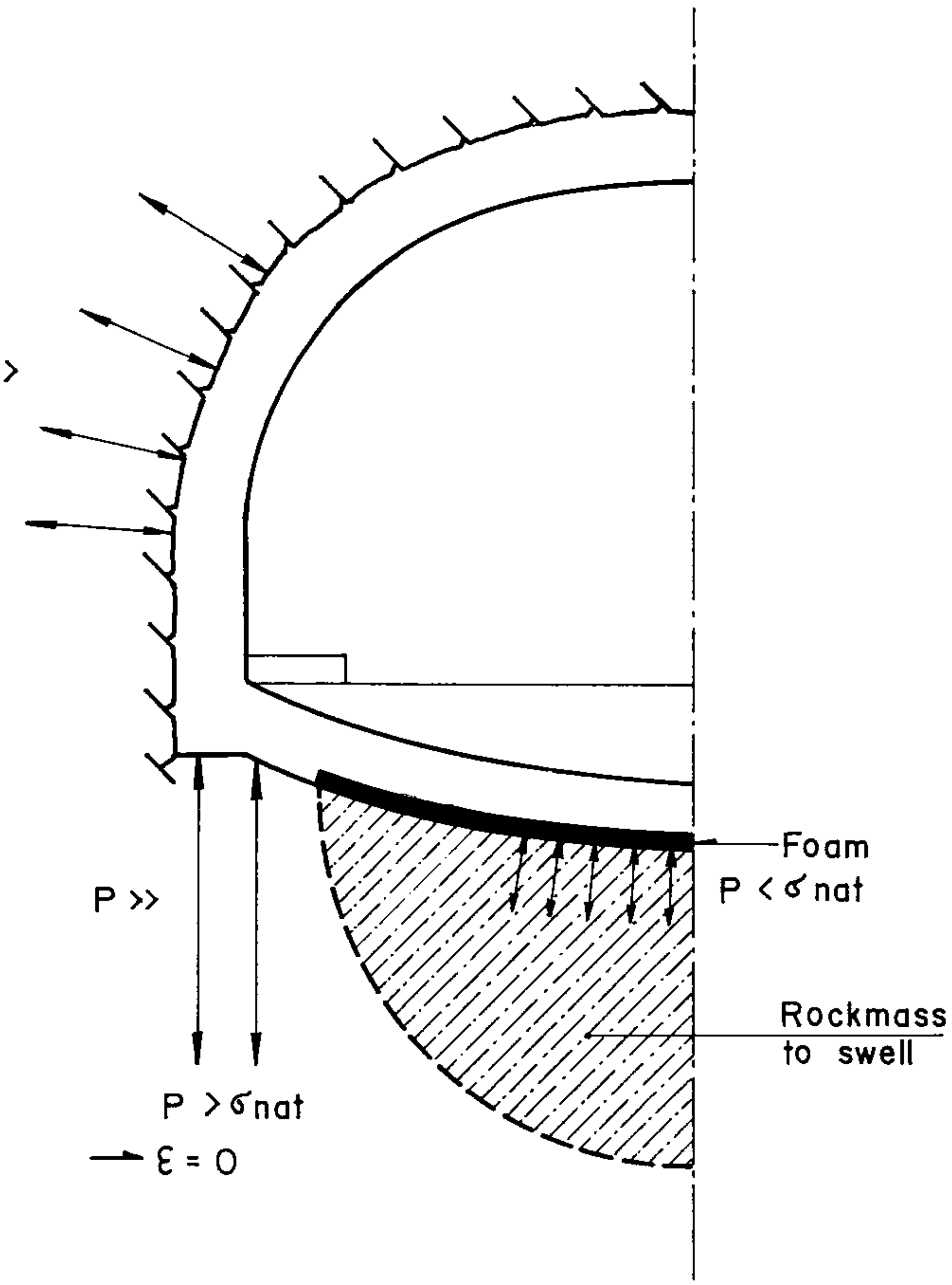


Figure 12

Buechberg tunnel 1974

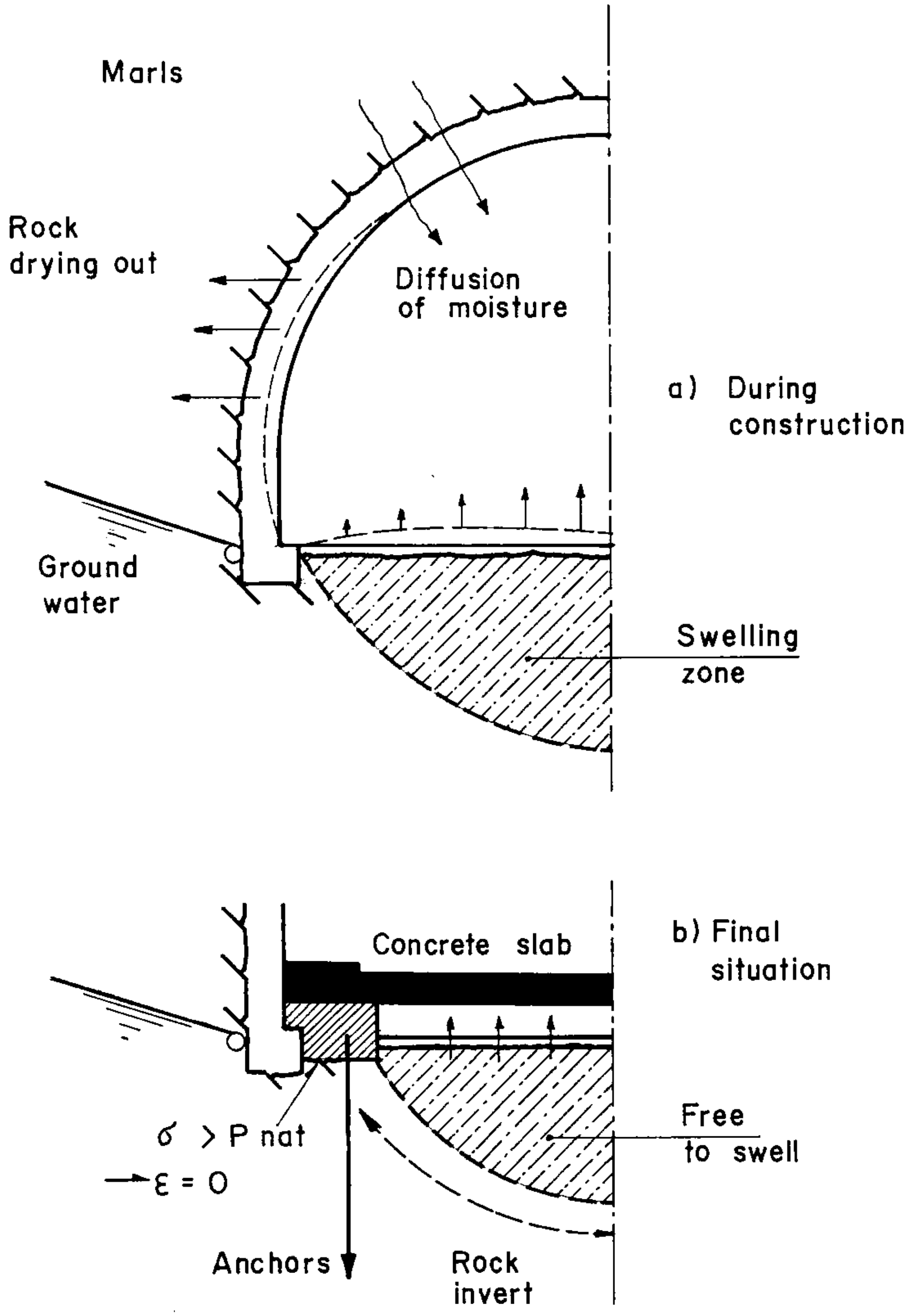


Figure 13

Seelisberg tunnel 1977

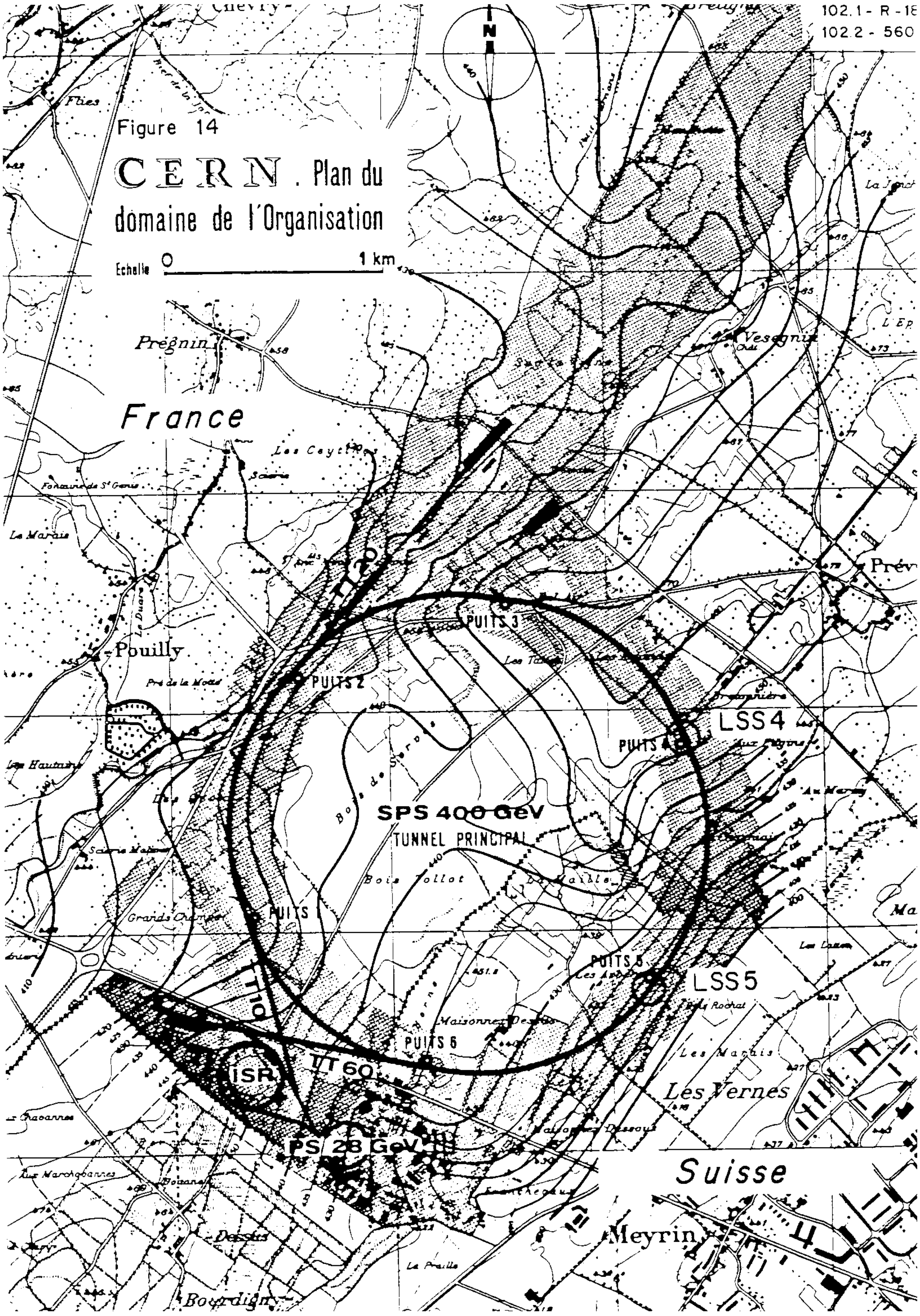


Figure 14

CERN . Plan du domaine de l'Organisation

Echelle 0 — 1 km

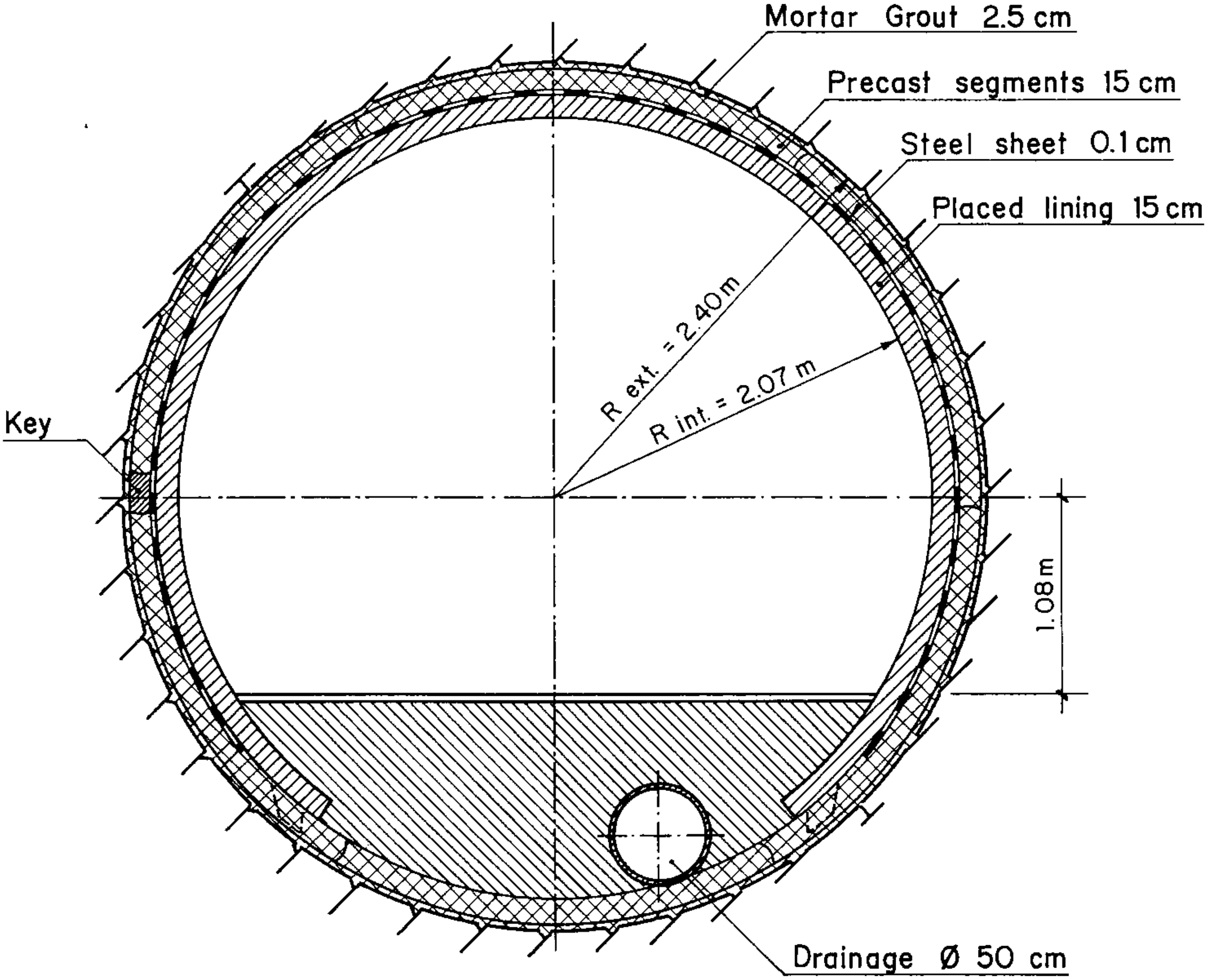
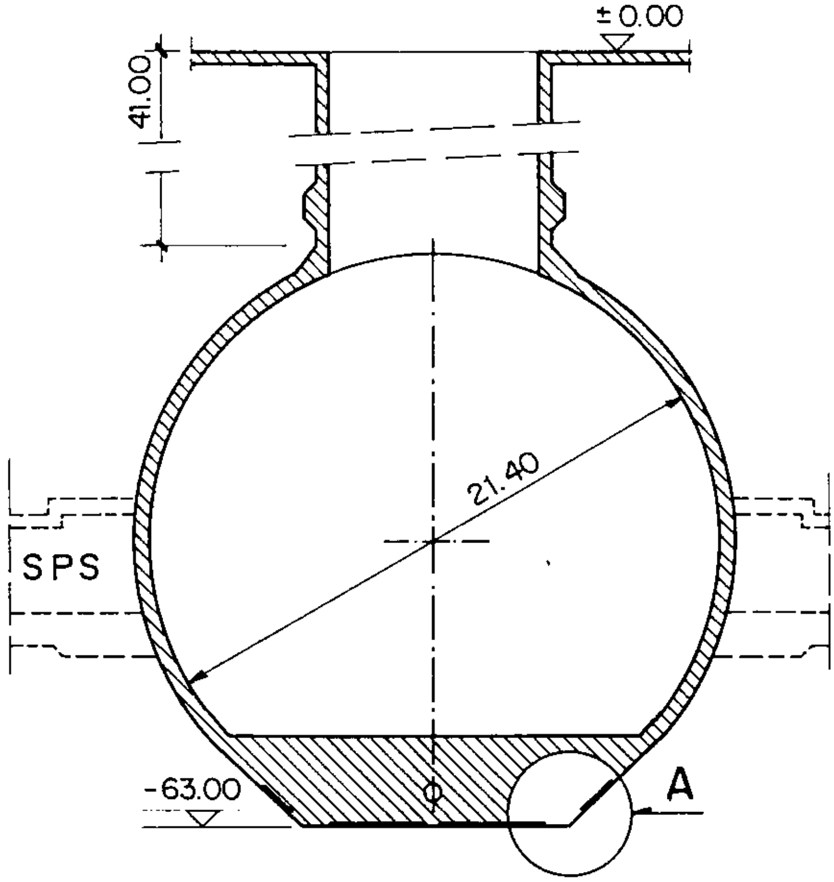


Figure 15

CERN

Main section of the SPS

Cross section (length 43.5 m)



Detail A

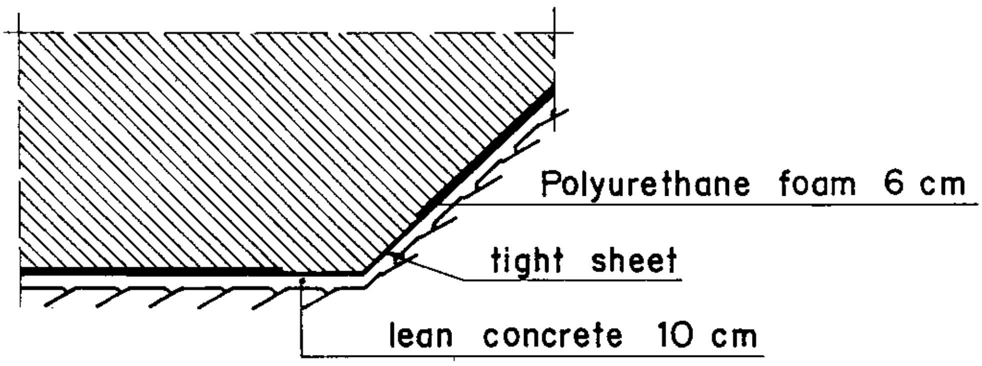
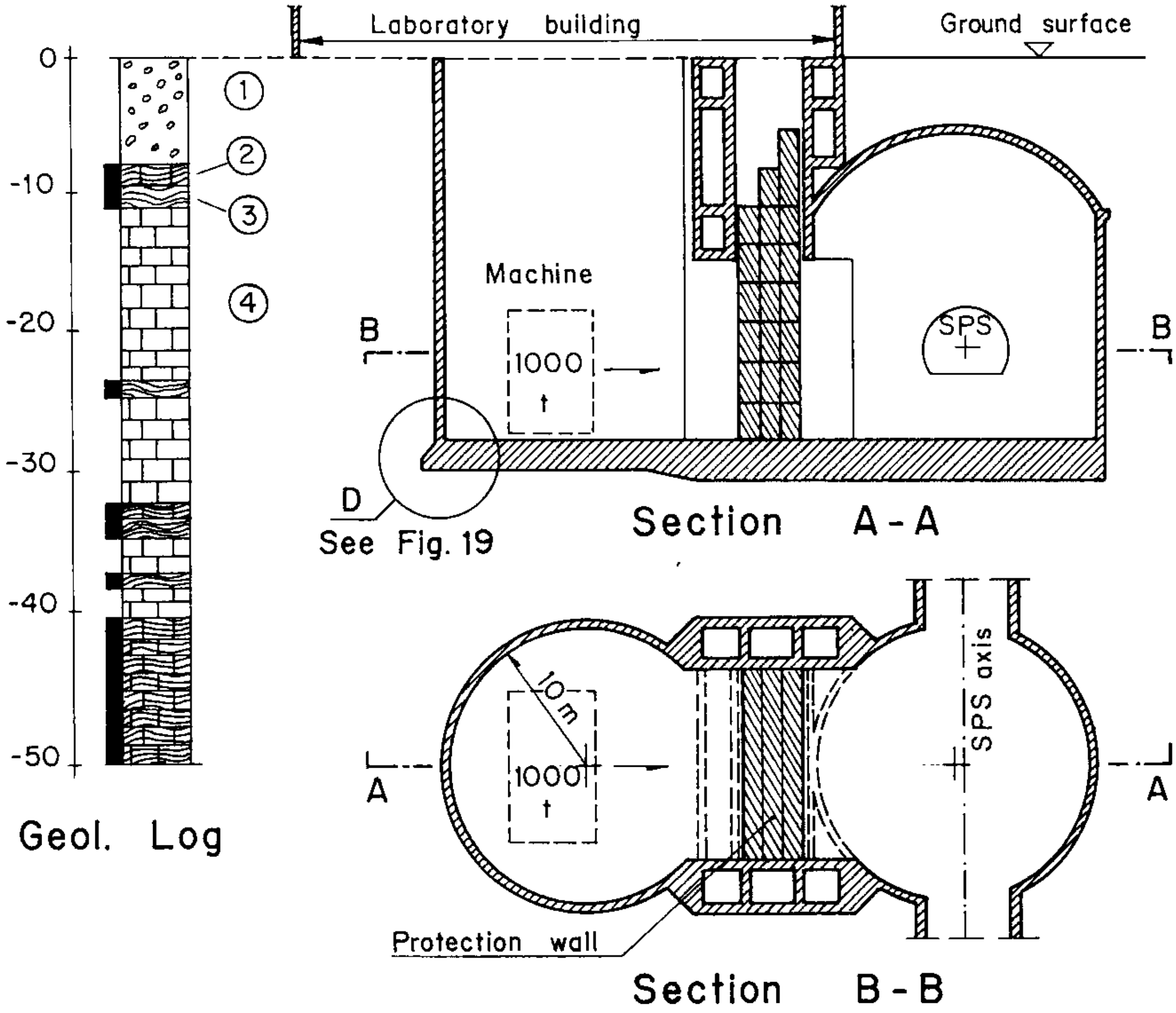


Figure 16

CERN

LSS 4 Underground chamber for proton - antiproton experiments.



Geology

① Moraine

② Marly sandstone

③ Marl

④ Sandstone

▮ Swelling rock layers

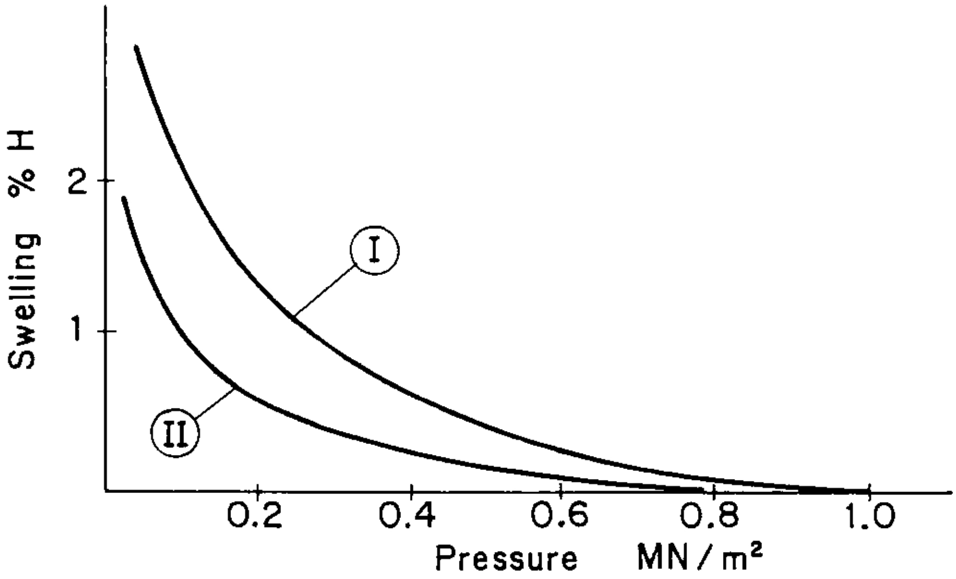
Main dimensions

Total	length	47.8 m
Total	width	21.2 m
Total	depth	30.0 m

Figure 17

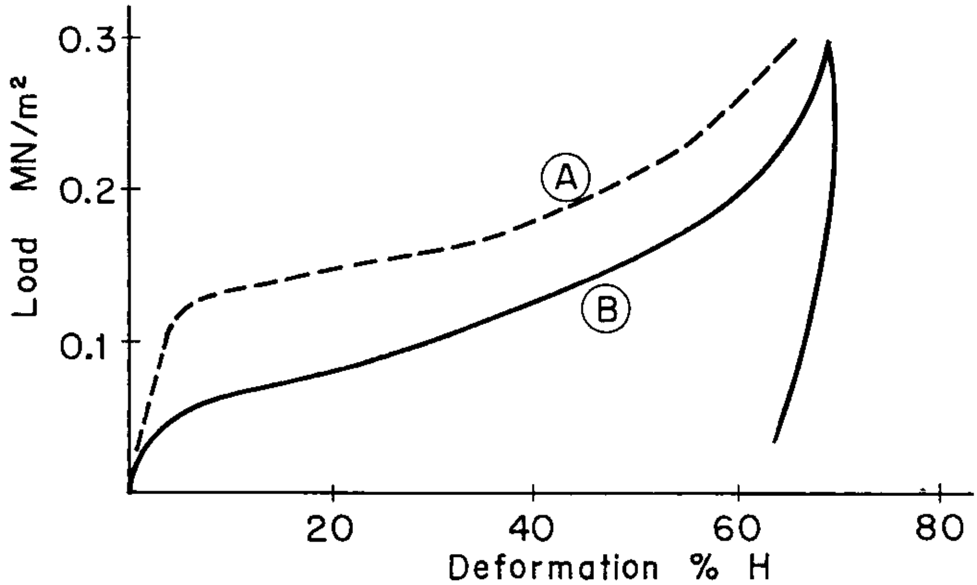
CERN

LSS 5 Underground chamber for proton - antiproton experiments.



Examples : of the swelling lines of the rock masses

(I) Marl (II) Marly sandstone



Compression lines of the foam

(A) : fast loading (1mm/min.) (B) : slow loading (30 days)

Figure 18

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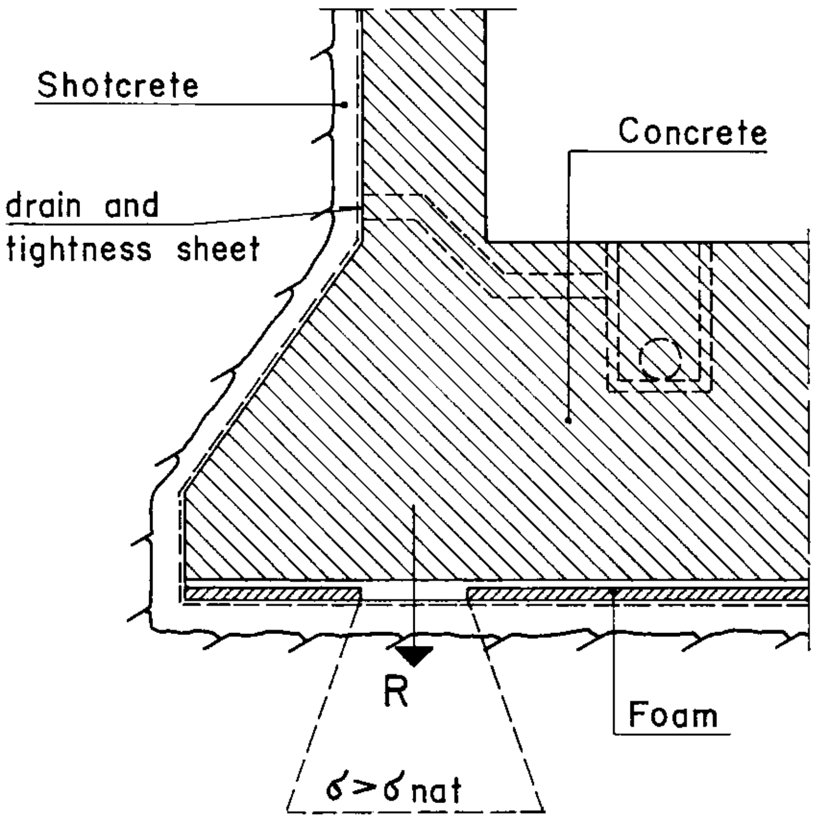
LSS 5 Underground chamber



Detail D of the edge of the foundation slab.
(See Fig. 17)

Figure 19

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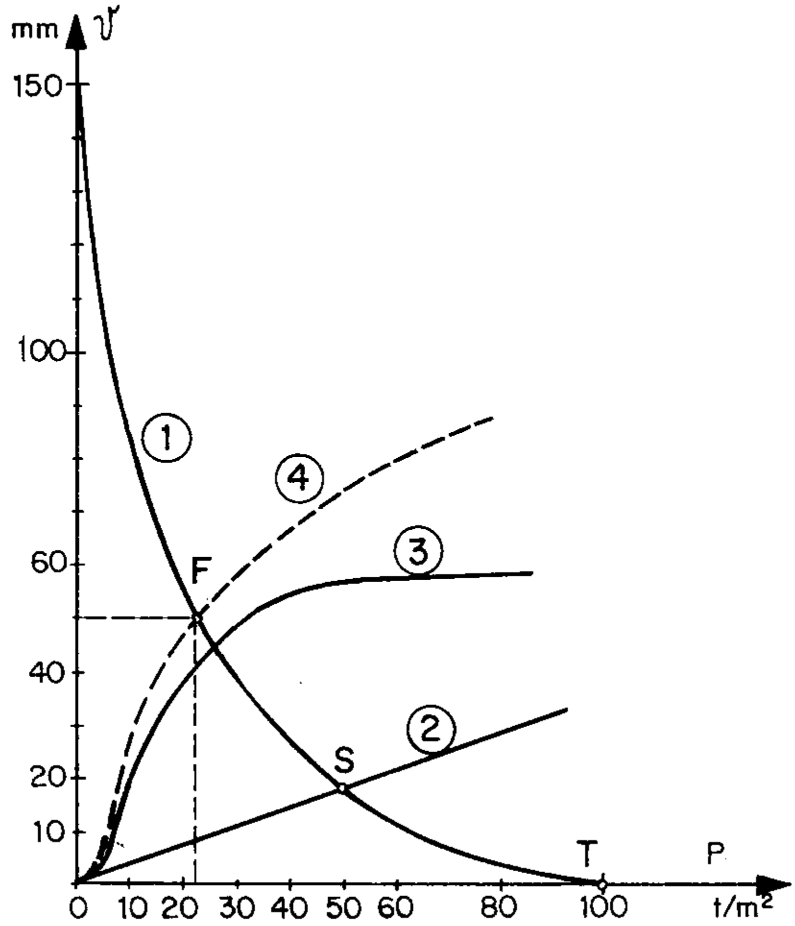
LSS 5 Underground chamber for proton - antiproton experiments.



ϑ = heave

P = load at the bottom

(1) Characteristic line of the rock mass

(2) characteristic line of the slab

(3) characteristic line of the foam

(4) characteristic line of the slab plus foam

F = final equilibrium

S = equilibrium without foam

T = equilibrium with abs. stiff slab

Figure 20

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Heave versus load at the bottom