Basic Considerations on the Design of Underground Openings

Problèmes lors du projet et de l'exécution de travaux souterrains

Probleme der Projektierung und Ausführung von Untertagbauten

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SUMMARY

After a brief review of the factors governing the structural behaviour of underground openings a survey is given of the use of numerical methods in tunnelling. The use of parametric studies to enable the important factors in a particular case to be singled out is illustrated. Field measurements during construction are discussed as a further design aid of equal importance.

RÉSUMÉ

Les possibilités des méthodes de calcul numérique dans la statique des tunnels sont expliquées, et une courte discussion des facteurs qui influencent le comportement des ouvrages souterrains est présentée. Ces calculs font l'objet d'étude de paramètres, ce qui permet la détermination des facteurs importants dans chaque cas particulier. L'exécution de mesures in-situ — autre moyen auxiliaire important pour le projet — est également mentionnée.

ZUSAMMENFASSUNG

Nach einer kurzen Diskussion der Faktoren, welche das Verhalten von Untertagbauwerken beeinflussen, werden die Möglichkeiten von numerischen Berechnungsmethoden in der Tunnelstatik erläutert. Solche Berechnungen führt man im Sinne parametrischer Studien durch, damit die Faktoren, welche für einen gegebenen Fall wesentlich sind, erkannt werden können. Als ein weiteres nützliches Hilfsmittel werden noch die Kontrollmessungen am Bauwerk erörtert.

1. INTRODUCTION

The design of underground openings like tunnels, subways and chambers in soil
or rock was in the past almost purely a matter of experience. In the last two
decades, however, new methods of site investigation, systematic measurements in
the field and computational methods have been introduced as powerful design aids
in order to arrive at safe and economical structures. In fact, the increasing
worldwide activity in the construction of underground openings and the frequen-
cy of large projects even under difficult geotechnical conditions call for a
continual improvement in design principles. The basic cause for the development
of displacements in the ground around the opening or for the occurrence of rock
and earth pressure phenomena is the disturbance of the stress field in the vir-
gin rock or soil due to the creation of the opening. Each step in the excava-
tion process involves a redistribution of stresses in the ground, thus trans-
forming the primary state of stress into the secondary state. Temporary and
permanent support like anchoring and tunnel lining have the task of restoring
a new state of equilibrium, firstly for the construction period, and secondly
for the service life of the structure. In many cases a new equilibrium state
is required under the rigorous condition of limited displacements around the
openings, for instance, in subway construction settlements of buildings and
traffic surfaces have to be kept to a minimum.

2. THE STRUCTURAL BEHAVIOUR OF UNDERGROUND OPENINGS

The tunnel support (lining, anchoring, etc.) and the surrounding rock form a
unit which is looked upon as the actual structure in tunnelling (Fig. 1). In
practice, the behaviour of this structure is often characterised by the nature
of the rock pressure, i.e. the effective contact stress between the ground and
the lining. The magnitude, distribution and time variation of the rock pressure
are important indicators of the sort of problem arising in tunnelling. The de-
formations of the tunnel section and the displacements in the rock together
with their time dependent characteristics, however, are also good indicators
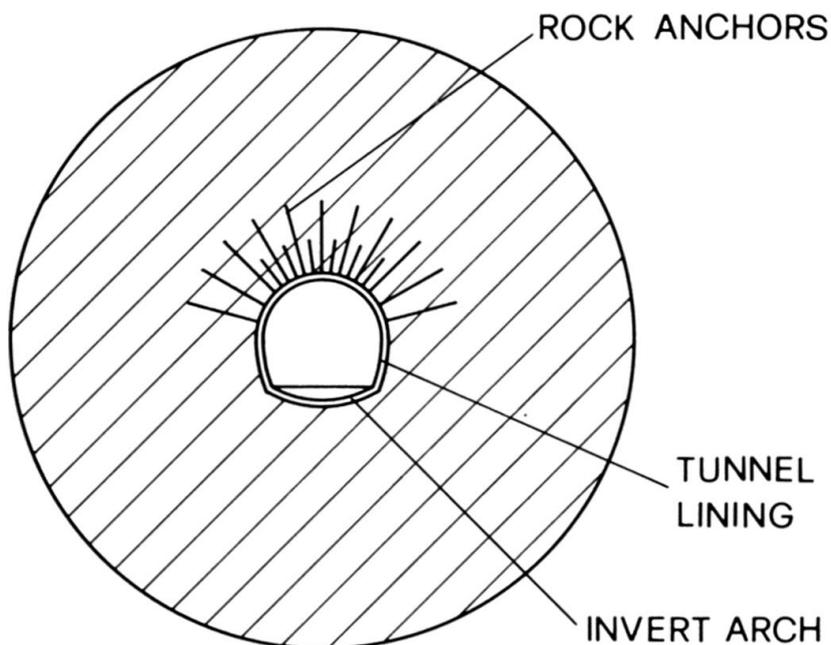for the behaviour of the structure.

Fig. 1

Tunnel support and
rock forming a
structural unit

Keeping the rock pressure under control and limiting the deformations respecti-
vely in the most economical way often presents the main problem in tunnelling.
In the solution of this problem it must be kept in mind that the behaviour of
an underground opening depends essentially on four groups of factors, namely

- material properties (soil, rock mass)
- the primary state of stress in the ground
- the size and shape of the opening and
- the method of construction

2.1 Material Properties

The materials in tunnelling - soil or rock - are not chosen as in some other
branches of structural engineering rather they are encountered. Their mechanical
properties are determined by means of geological surveys and soil and rock me-
chanics investigations. As far as possible this information should be obtained
well in advance of the construction. Generally drill holes or adits give access
to the material in the area of the planned underground opening. Often, important
information is gathered from outcrops on the surface, as well as by using the
experience gained from previous constructions under similar geotechnical condi-
tions. The rock properties on the scale of specimen size together with the
structure of the rock mass determine properties on the scale of the construction.
The rock structure is given by stratifiction, schistarity and jointing. The lat-
ter constitute actual or potential surfaces of separation or slip. Therefore,
their frequency and orientation in space are generally of great importance. The
material tests in the laboratory comprise soil mechanics investigations,uniaxial
and triaxial compression tests and frequently direct shear tests on surfaces of
weakness. Load tests in boreholes or even trial sections in tunnels or chambers
on a reduced scale or on full profile can, in certain cases, be applied with
advantage as further methods of investigation.

Of the many aspects that are important for the geological conditions only two
are given special mention here, namely the presence of water and the rock types
containing clay or anhydrite. Water inflow in even relatively small quantities
into the opening may affect substantially the progress in excavation. The water
may reduce the strength of the material by decreasing its cohesion or by decrea-
sing the effective normal stresses in the sense of Terzaghi. When tunnelling in
saturated soils special measures, often very expensive, must be taken in order to
prevent infiltration and to stabilize the ground, for example, ground water
lowering, application of air pressure, hydro shield or ground freezing techni-
ques. Rocks containing clay or anhydrite give rise to special problems in tun-
nelling. Such rocks, e.g. marlstones and anhydrite, can swell, i.e. increase
considerably their volume due to absorption of water, whereby a substantial
amount of heave in the bottom of the tunnel may occur. The tunnel lining (invert
arch), in resisting the heave, may be subjected to high swelling pressures. In
tunnelling practice, unconstrained heave of up to 70 cm may occur {1} and swel-
ling pressures of up to 3,5 MN/m^2 have been reported to act on the invert arch
{2}.

Many of the unexpected difficulties that arise in tunnelling can be traced back
to an inadequate knowledge of the material properties. The actual rock condi-
tions are often, in fact, first known as the underground opening is under con-
struction. This is specially true for deep tunnels, for which borehole explora-
tions either for technical or economic reasons are out of question, or else can
only be carried out to a very limited extent. Also, one only has to think of the
possible variability of the material with respect to its petrographic composi-

tion and its structure (jointing etc.), then it becomes evident that it is especially important to determine the ranges in which the rock mass behaviour may be expected to vary. Here, not only statical but also purely constructional considerations can be important. The greater the degree of mechanisation in the method of construction, the more important possible extreme cases in the material occurrence become. For instance, when using the shield tunnelling method in soils, if the cutting edge comes up against occasional boulders a big time delay in construction may result, which leads to increased costs. Turning to another example, the economical application of a full-face boring machine is not only limited by poor rock quality (too short stand-up time of the rock, insufficient thrust for the advance of the machine) but also in certain circumstances by a very hard, massive rock. The more uncertain the geotechnical predictions or variable the rock conditions, the more adaptable the constructional method has to be. The adaptation of the method of construction to the rock conditions is so important, that in tunnelling a rock classification may be made on the basis of the required constructional measures {3}. Such a classification is then fundamental for the contract between the owner and contractor.

2.2 The Initial State of Stress in the Ground

Due to gravitational forces and possible tectonic influences, the rock is already stressed before the underground opening is excavated. Thus, one speaks of an initial or primary state of stress, which, of course, is different from location to location(Fig. 2). There are two ways in which the initial stresses may give rise to difficulties in tunnelling. Firstly, the material in the vicinity of the opening often reacts to the changes in the stress field by failure and creep processes, which may lead either to radial movement or, if it is hindered, to the development of rock pressure. Secondly, in hard rock at great depths the much feared phenomenon of rock burst may occur. This is characterized by the explosive-like separation of plate-shaped pieces of rock often of considerable size, which may endanger the lives of the people working in the tunnel. The mechanism of rock burst has not, as yet, been adequately investigated. All that is known with certainty is that the orientation of the tunnel axis in relation to the directions of the principal stresses of the initial state of stress plays an important role.

The stress tensor in the rock cannot be determined theoretically because of the changing topographical conditions, the generally complex structure of the rock mass and its nonlinear stress-strain relationship, and the tectonic forces which

$$\sigma_{ij} = \begin{bmatrix} \sigma_{11} & \sigma_{12} & \sigma_{13} \\ \sigma_{21} & \sigma_{22} & \sigma_{23} \\ \sigma_{31} & \sigma_{32} & \sigma_{33} \end{bmatrix}$$
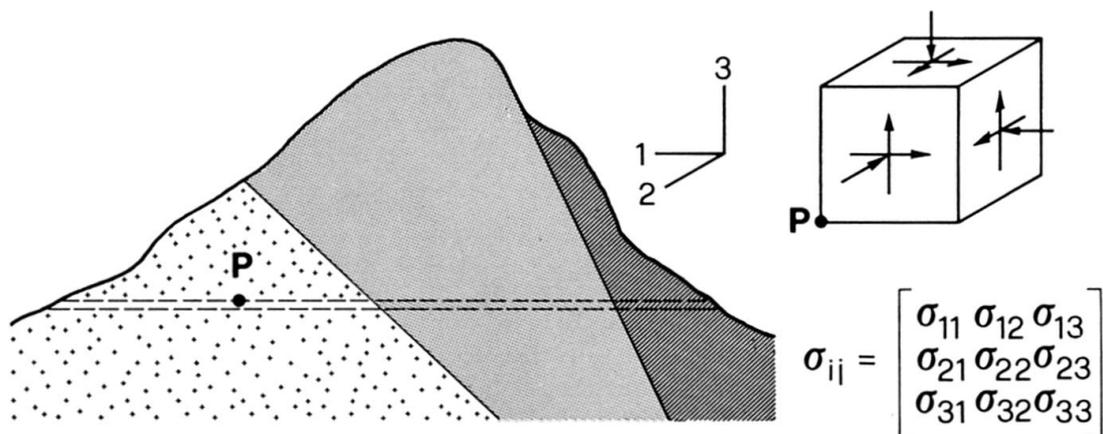
Fig. 2 Initial state of stress in the rock

may still be active today. Stress measurements in situ are only successful if
the rock in the immediate vicinity of the measuring point can be assumed to be
elastic, isotropic and homogeneous. Unfortunately, these conditions are often
only fulfilled in those cases in which knowledge of the initial stresses due to
excellent rock strength is only of secondary importance. Thus, with regard to
the magnitude and direction of the principal stresses, we are left with little
more than suppositions. For a more or less horizontal surface terrain it is
justifiable to assume that the vertical normal stress in the initial state is
approximately equal to the overburden stress of the overlying rock or soil. No
generally valid statement can be made about the horizontal normal stress compo-
nent. It can vary from a small fraction to a multiple of the vertical stress.
The lower and upper limits for the relationship between the horizontal and
the vertical normal stresses may be assessed by the failure condition of the
material in the sense of the active and passive earth pressures. It may be noted
that the greater the tendency for the material to creep and the greater the
overburden pressure, the closer the initial stresses approach a hydrostatic
stress condition. Tunnels located in slopes or beneath the bottom of a deep
valley require special attention with regard to the initial state of stress.

2.3 Dimensions and Shape of Underground Openings

The relationship between the span of the opening and the average joint spacing
is decisive in many cases for stability considerations. With increasing span
the influence of the jointing becomes more marked and the probability of an un-
favourable joint combination, which could give rise to a rock fall, increases.
In the special case of soil with no cohesion the vertical pressure on the tunnel
lining in the roof increases with increasing span of the tunnel. The ratio of
span of tunnel to height of overburden is also an important factor. If this
ratio is less than one it is not possible to develop an arching effect in the
soil, not even in heavily-jointed rock. Especially large dimensions in the con-
struction of tunnels or rock chambers are only possible, from the point of view
of safety and economy, by imparting a special shape to the profile. A good
example illustrating this point is a chamber in the form of a vertical cylinder
with a spherical closure (Fig. 3). Statically this shape is very favourable,for
horizontally we have the effect of a closed ring and a double arching action
exists at the roof closure. Cavities of this form and with dimensions of about
H = 80 m, D = 45 m are at present planned for underground nuclear power plants.
The shape of a section is also important in the case of a tunnel. However, as
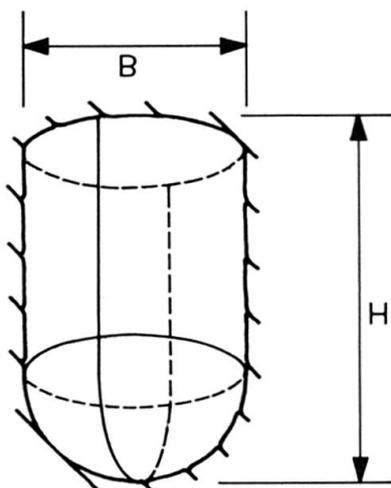

Fig. 3 Large underground chamber
with statically favourable
shape

a design parameter it is, in many instances, not given the attention it deserves.
Should rock conditions be encountered in which high rock pressure is expected,
the shape of the profile should be selected in such a way that an arching action
in the rock and tunnel lining may be developed. In railway tunnels, for instance,
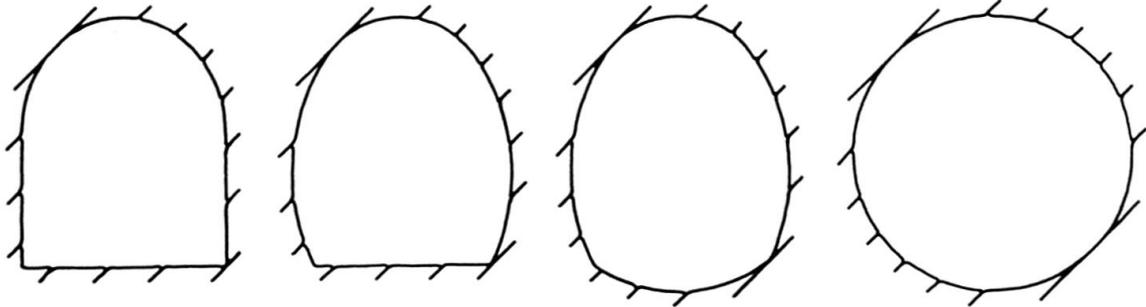this can be achieved by choosing shapes as shown in Fig. 4.

Fig. 4 Possible adaptation of tunnel profile to increasing rock pressure

2.4 Method of Construction and Support Measures

The method by which the opening is excavated along its longitudinal direction ·
and in the cross section can have a significant influence on the development of
the rock pressure and the displacements in the surrounding rock. In the case of
a tunnel the profile can be excavated in a full face operation or by dividing
the section in different parts and excavating it in different sequences (hea-
ding and bench method, multiple drift method etc.) Difficulties of various kinds
can be overcome more easily when working in smaller cross sections. When the
rock conditions require it, therefore, the profile must be excavated in two or
more stages (Fig. 5), whereby staging is also employed in the direction of the
tunnel axis. The first stage of excavation is in many cases well in advance of
the works on enlarging the section to the full profile thus providing an useful
means of rock exploration. Depending on whether the problem is to control the
rock pressure or to limit the displacements in the neighbourhood of the tunnel,
various constructional procedures may be chosen along the axis of the tunnel.
This is illustrated by practical examples, one for a subway construction and
the other for a deep tunnel, both driven through a soft rock. For the cross sec-
tion one can in both cases proceed according to Fig. 6. For the subway tunnel,
in order to avoid undesirable settlements of buildings in its vicinity, the in-
vert arch of the permanent lining should be placed as quickly as possible. The
time required to complete a full ring may be only a matter of days or a couple
of weeks. Thus, in this way, at a distance of less than one tunnel diameter a
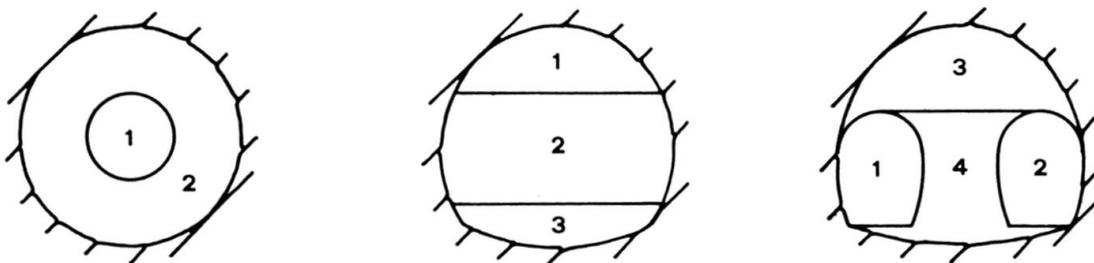closed ring is formed, which is statically extremely efficient. In a tunnel

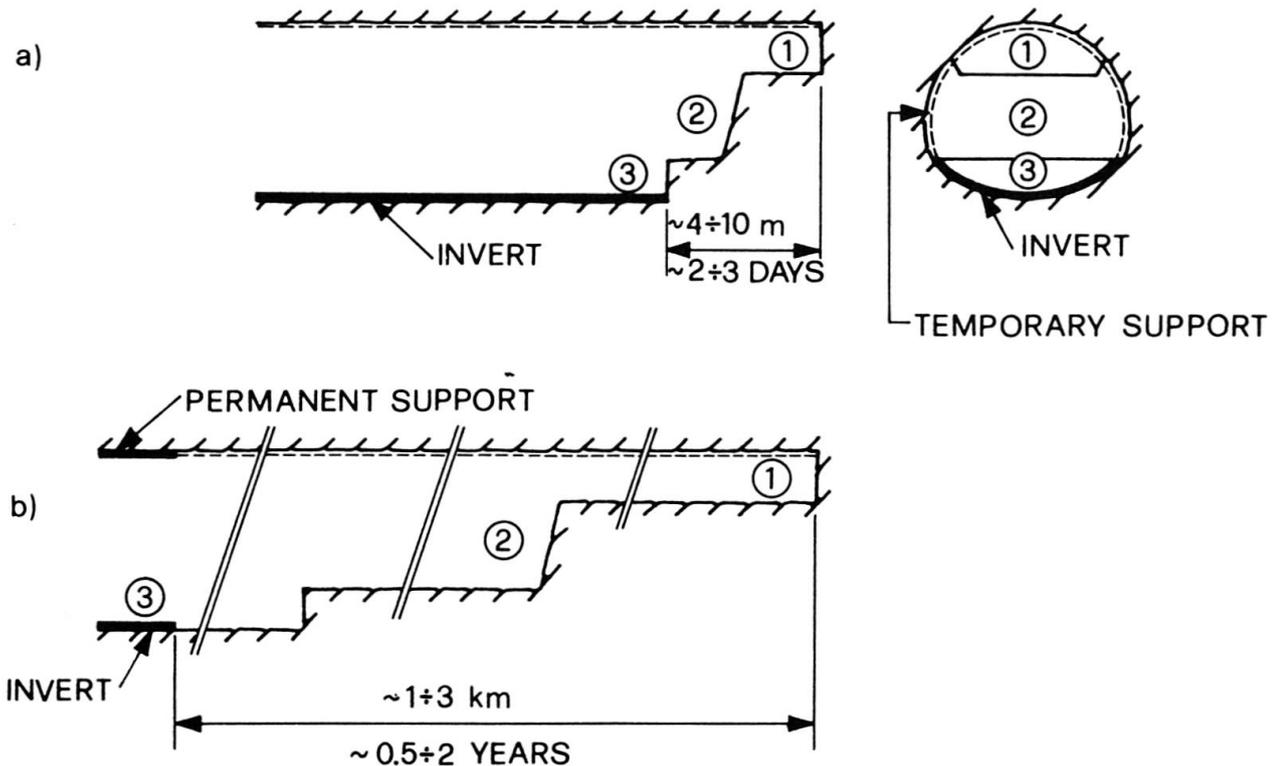Fig. 5 Examples of multiple stage excavation in the tunnel section

Fig. 6 Placement of an invert-arch
       a) close to the face in a subway construction
       b) at great distance from the face in a deep tunnel

situated at great depth, where high rock pressures can be developed, considerable
deformations may be deliberately permitted using a flexible temporary support
to keep them under control. In any case, it is impossible to prevent the defor-
mations completely even when using a stiff lining, since the pressures that
would occur may be of the order of magnitude of the initial stresses (in a depth
of 1000 m there would be an overburden pressure of about 30 MN/m^2 in the rock).
Thus, with the protection of a flexible, temporary support, one allows radial
displacements of the sides of the opening of up to 50 cm or more, which in some
cases may take years to develop. Any further deformation that might occur can
then be safely prevented using a suitable ring-shaped permanent lining. This may
follow the working face of the heading in a distance of a few kilometers. With
regard to the conventional methods of excavation only the elementary requirement
of carefully controlled blasting which causes the least disturbance of the sur-
rounding rock is mentioned here. The rock should not be unnecessarily loosened
by blasting, as this would result in a considerable loss in strength. The in-
disputable advantage of blast-free mechanical excavation methods is that they
do not effect the in situ rock quality around the opening.

In summarising the above it holds true generally that the method of excavation
and the type of support system (rigid or flexible) as well as the time and place
of its installation have a profound influence on the behaviour of the under-
ground opening.

3. DESIGN AIDS IN TUNNELLING

A clear understanding of the above factors and their importance in influencing the behaviour of an underground opening under specific conditions can only grow out of the engineer's own experience and out of his theoretical knowledge. Experience manifests itself in good structural judgement. Together with statical computations and systematic field measurements it forms the basis for decision - making both at the planning and construction stage. To what extent such modern aids should be applied on a given project depends solely on the nature of the problems arising. In the following, an attempt will be made to give an up-to-date survey of the possibilities and limitations of statical analysis and field observation techniques. By means of statical computations an attempt is made to predict the structural behaviour of the opening in an analytical way. The interrelationship between the various factors, for instance rock properties, shape and dimensions of the opening, initial state of stress etc. may be clearly seen in the calculated results. But although these results are available at the design stage, they are subject to great uncertainities. Measurements carried out on the structure enable its behaviour to be observed directly, without the actual mechanism, which gives rise to its behaviour, necessarily being illuminated. The measurements are usually carried out during the constructional phase and if carefully planned and executed they can give a true picture of the behaviour of the structure. From these considerations it is obvious that computations and measurements complement each other and only when combined are they capable of leading to a correct explanation of the structural performance in complex geotechnical situations.

4 STATICAL COMPUTATIONS

Statical computations are concerned with the numerical investigation of the external and internal forces acting on the structure as well as with the associated deformations. Computations in tunnelling are carried out mainly for the following reasons:

i   so that a particular design, based on experience, can be investigated numerically. In this way orders of magnitude can be estimated and critical points in the design recognized.

ii   so that important decisions concerning a design solution can be substantiated before professional colleagues, responsible authorities and last but not least for the designer himself.

Static analysis, since it is of a quantitative nature, offers the best framework for theoretical considerations. Experience shows that even the efforts to find a suitable structural model for computations brings to light the actual problems. We should not expect too much, however, from the static analysis of underground structures. There is a difference between computations for a tunnel and, say, for a bridge. In the case of underground structures computations are a tool for studying the problem, and they only give hints with regard to understanding the behaviour, whereas for the bridge they provide a numerical check on the design in terms of stress, deformation and stability. This could be one of the reasons for the fact that in tunnelling the use of numerical methods is sometimes either rejected completely or excessively over-rated.

The typical problem areas for the static analysis of tunnels are:

i   stability of openings
ii   interaction between structures
iii   interpretation of field measurement data

These problems are met with generally both in the planning phase and during
construction. Occasionally they are also encountered after the structure has
been completed, for example, in the cases of reconstruction, extensions or when
disputes arise. Under the heading "stability of openings" those problems are
included which occur with respect to the safety of an opening against rock fall,
major break downs or inadmissible deformations. These problems concern the ade-
quate selection of profile-shape, the method of construction and the type of
temporary and permanent support. The "interaction of structures" can apply to
adjacent tunnels (twin-tubes, tunnel underpass) and underground openings within
the zone of influence of structures at the ground surface. In such cases - above
all in subway constructions - all measures must frequently be taken for settle-
ment and deformational criteria. The third group of problems intimately linked
with the static analysis is the "interpretation of observations and measure-
ments". A problem here is to cope with the large amount of data often obtained
from a program of field measurements, and to evaluate this data such that it
is useful for the case in hand as well as for structures in similar geotechni-
cal situations. Although, in some cases, all three groups of problems can be
present on the same project, for the sake of clarity we shall treat them sepa-
rately.

4.1 The Computer as a Tool of the Designer

Tunnel design is still dominated by the use of a two-dimensional statical system
under the condition of plane strain. In some cases, however, the consideration
of three-dimensional systems is not only desirable but also justified economi-
cally. Boundary conditions must be considered at the opening and on the exterior
surface, whereby displacement and load conditions have to be taken in account.
The system is rendered inhomogeneous by the inclusion of various material zones.
The tunnel lining is best simulated by a truss built up of pin-ended bars which
also may be used for rock anchors (Fig. 7). The material property of the rock is
often assumed to be described by an ideal elasto-plastic body using the Mohr-
Coulomb failure criterium. Due to the presence of regular sets of joints, layers
or foliation one would require in the general case a transverse-anisotropic ma-
terial behaviour. The corresponding constituitive law needs five elastic and
four strength parameters in order to be described completely. With the help of
the computer, it is now possible to analyse easily and economically an inho-
mogeneous, arbitrarily-shaped body in plane strain condition consisting of ani-
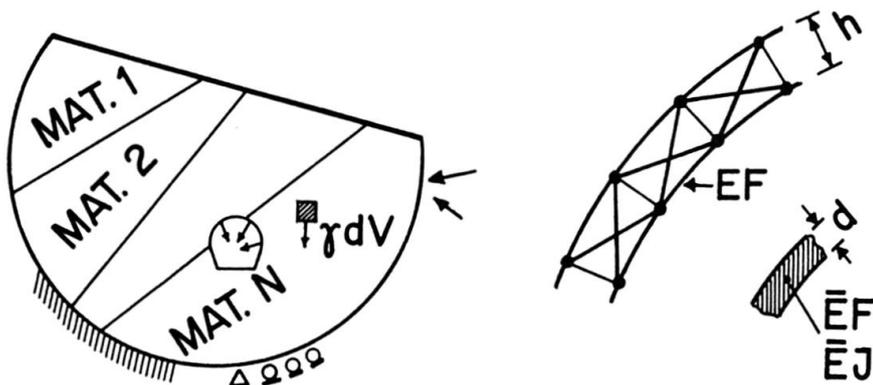sotropic, elasto-plastic zones of material under various loading conditions {4}.

Fig. 7

Structural model with
the simulation of a
tunnel lining by a
truss in a 2-dimen-
sional analysis

A point of great practical importance is that the computation can be carried
out in steps, which gives the engineer the possibility of modifying the input
data at various stages of the calculations. In this way boundary conditions and
material properties can be changed to follow as best as possible the various
constructional stages. The basic numerical method for practically all cases is
the finite element method {5}.

4.2 The Stability of Underground Openings

Under favourable rock conditions the opening requires neither a temporary nor a
permanent support. For very difficult conditions, however, the opening can be
excavated only with the immediate aid of very extensive support arrangements.
In quickly changing geological conditions, both extremes are often found at the
same site, and there are many situations which lie in between. Thus the compu-
tational models must be adapted to the structure section-by-section depending
on the type of rock pressure anticipated. A differentiation between rock pres-
sures in terms of the following three phenomena, namely

- loosening pressure
- genuine rock pressure
- swelling pressure

corresponds to traditional engineering practice and also gives the first impor-
tant indications of the kind of statical problems to be expected. The variations
in model concepts are also brought about by the different ways in simulating
the interaction between tunnel and surrounding ground.

4.2.1 Computational models for rock masses with loosening pressure

Such a rock mass is basically self-supporting around the excavated opening. The
rock samples have a uniaxial compressive strength which is many times higher
than the overburden pressure. Thus the excavation might not induce new fractu-
res in the rock. Its behaviour is characterized mainly by the original joint
pattern and by the shear strength along those joints or other surfaces of weak-
ness. The overall deformations of the rock take place essentially before the
introduction of the supports. The corresponding reduction in the cross section
is negligible, and generally hardly noticeable. The danger of loosening and
rock fall due to the presence of local defects, however, call for permanent and,
in many cases, for temporary support as well. Unfavourable combinations of
layers and joints, shattering effects of blasting and percolation of ground
water can all contribute to the settling of a certain volume of material on to
the lining, which exerts a pressure according to its deadweight. Because the
process of loosening occurs in parts of the tunnel which cannot be determined
in advance and is influenced by uncontrollable factors, its prediction computa-
tionally appears to be out of question {6}. In this type of rock a reasonable
approach is to assume a loading in the roof-zone of the tunnel which corresponds
to the estimated volume of the loosened material. The interaction of the lining
with the rock mass is taken into account by the partial embedment of the tunnel
lining in an elastic continuum (Fig. 8). The modulus E, of the continuum corres-
ponds to the deformability of the rock mass. In the computation the condition is
applied that no tensile stress is transmitted between the rock and the lining.
Generally it necessitates an iterative procedure. A criticism of this computa-
tional model is that all computed results are dependent upon the estimated loa-
ding and are thus questionable. One must consider, however, that by means of
such computations useful relationships between important factors, like the shape
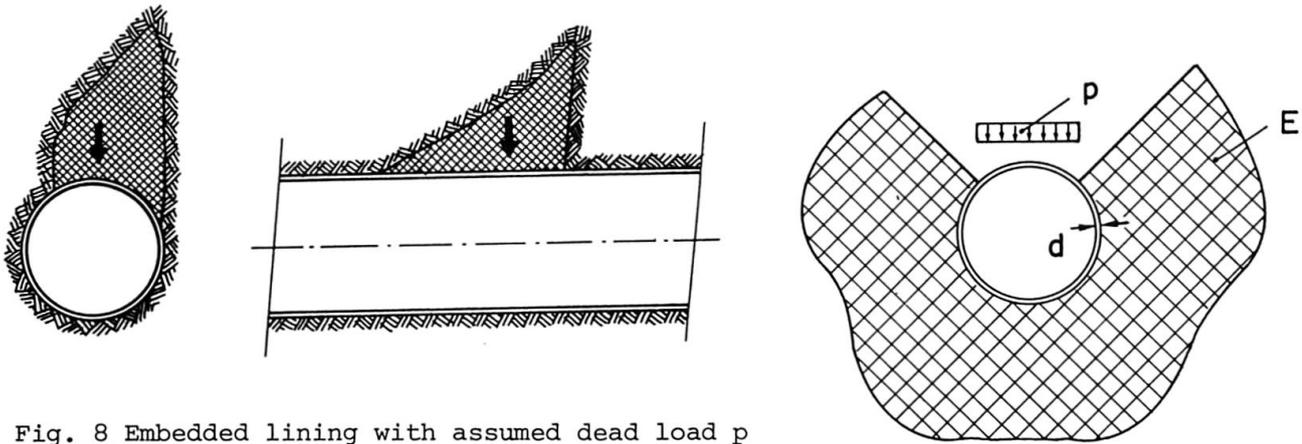
Fig. 8 Embedded lining with assumed dead load p
        of the loosened rock body

and size of the cross-section, the stiffness of the lining and the deformability
of the rock, may be established. In this way a reasonable and rewarding para-
metric study can be carried out. Optimistic and pessimistic estimates of the
loading can be made by considering the constructional method (protective or
destructive excavation), the rock structure (jointing etc.) and the size of the
opening. Similar considerations are valid for the deformability modulus, E, of
the rock mass.

Such a computation is now illustrated with an example from tunnelling practice.
In connection with the construction of a major hydroelectric scheme parametric
studies had to be carried out for the design of several, parallel-running pres-
sure tunnels of unusually large diameter. The rock – a compact gneiss – was
well-suited to the excavation of tunnels of up to 300 m^2 cross-sectional area.
The basis for the determination of the diameter of the opening as well as for
the determination of the depth of the concrete lining were provided by a para-
metric study. Several loading cases, different values of E for the deformabi-
lity of the surrounding rock and different lining depths were considered. In
Fig. 9 the computational models for an assumed rock pressure q due to loosening
and for the loading case of an external ground water pressure w (for sudden
emptying of the tunnel) is represented. The bending moments M and normal forces
N of the lining were computed by an iterative procedure, in which tensile stres-
ses acting between lining and rock were eliminated.

4.2.2 Computational models for rock masses with genuine rock pressure

In contrast to the rock masses characterized by loosening pressure, in this kind
of rock it is not the occurrence of accidental details such as jointing that is
important, but the properties of the basic material surrounding the tunnel. Its
low strength, its susceptibility to creep, the presence of water and high over-
burden stresses favour the development of genuine rock pressure. The deformations
occur in all directions and make their presence felt in the form of a reduction
in cross section. The ground exerts pressure on the tunnel support from all
sides even from the bottom. The movements are strongly time-dependent but reach
a standstill as a consequence of the support measures. In order to follow, by
means of computations, the occurrence of failure in the rock, i.e. the spread of

D = 18.0 m

d = 0.4 m

E = 2 x 10⁴ N/mm²

q = 100 kN/m'

H = 35 m

TRUSS (LINING)

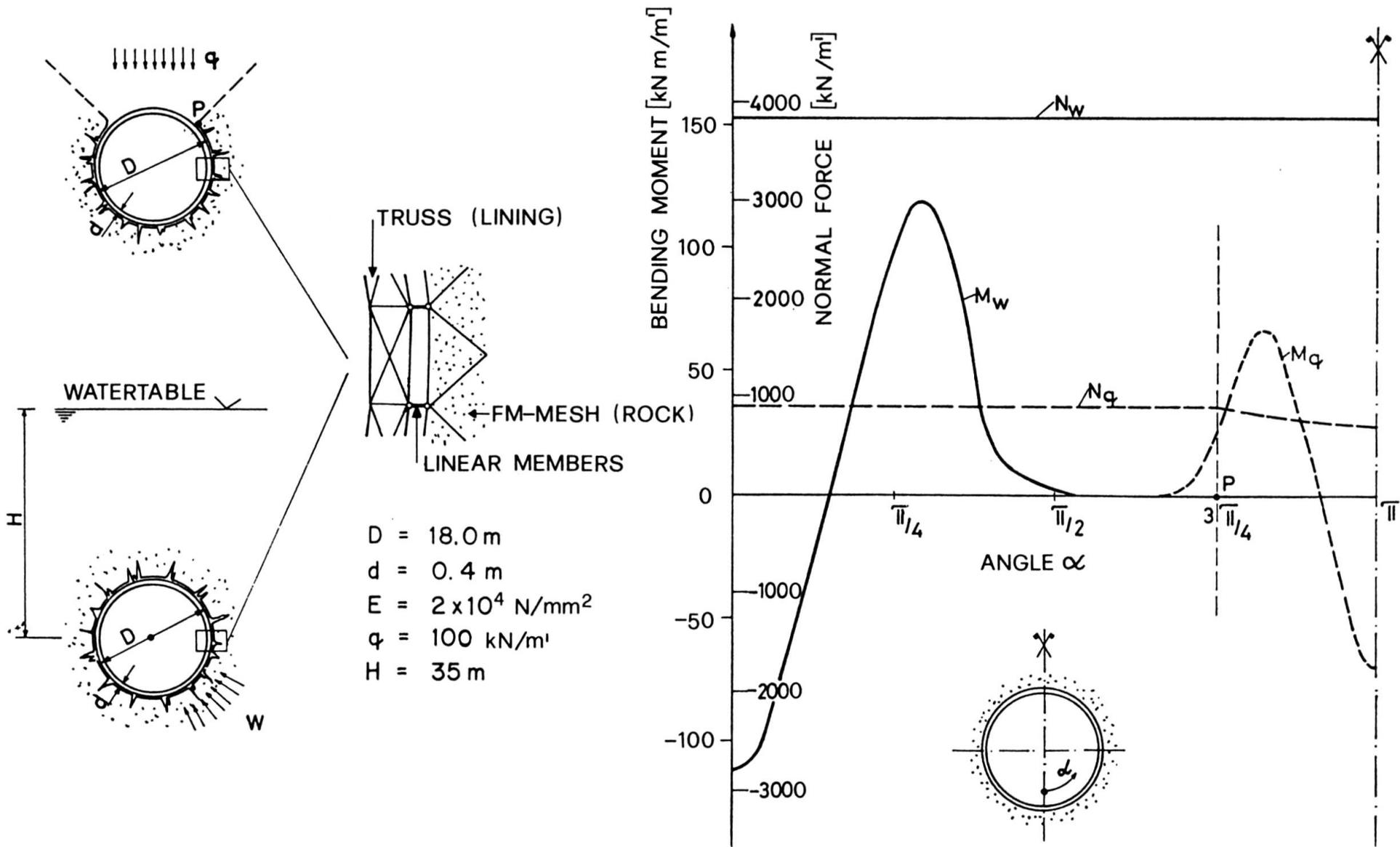FM-MESH (ROCK)

LINEAR MEMBERS

WATERTABLE

Fig. 9 Large diameter pressure tunnel : moments and normal forces in the
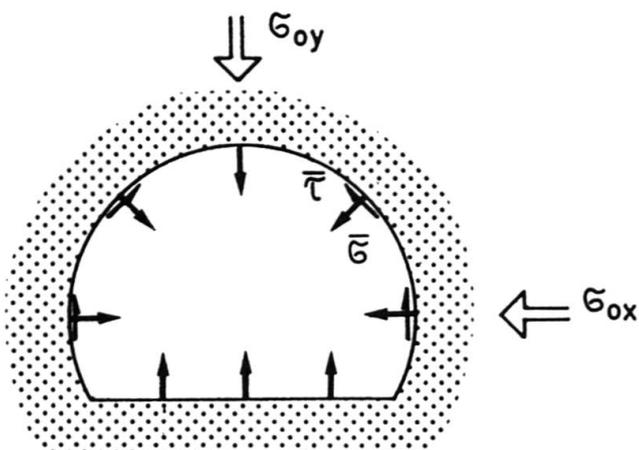lining due to rock load q and water pressure w

Fig. 10 Removal of boundary stresses
$\bar{\sigma}$ and $\bar{\tau}$ according to the
initial state of stress
$(\sigma_{ox}, \sigma_{oy})$

the plastic zones, the redistribution of stresses and associated deformations,
a continuum model is generally used as described in chapter 4.1. First we con-
sider the stress redistribution in the case of an unlined opening. The excava-
tion of a hole leads to a stress-free internal boundary. One proceeds, therefore,
(Fig. 10) from the initial stress state $(\sigma_{ox}, \sigma_{oy})$, determines the stress com-
ponents at the boundary $(\bar{\sigma}, \bar{\tau})$, and then applies these with reversed sign. For
elasto-plastic analysis this procedure of unloading at the boundary is carried
out in small steps using load increments. If, in the iterative calculation, no
convergence is obtained, this may indicate an instability of the underground
opening.

Next, the possibilities for considering the lining in the statical system are
looked at. The tunnel lining is introduced into the opening as a new and foreign
structural element, using a particular constructional method. The lining is
stressed as a result of those movements that occur after the lining has been
placed. These rock movements may be traced back to two basic causes:

i   to a new stage of excavation in the vicinity of the lining already
     introduced
ii   to creep of the rock material

Even if the tunnel is excavated in full face operation and the lining is placed
immediately at the face, the rock will still undergo stress changes and deforma-
tions before the lining begins to function (Fig. 11). This behaviour in the imme-
diate neighbourhood of the tunnel face has been the subject of extensive theore-
tical research {7}, experiments, and in-situ observations {8}. How is it possible
for a two-dimensional model with time-independent material properties to describe
this complicated process? We can do it, evidently, only by making extremely
simplifying assumptions. These assumptions relate to deformation of the rock
preceding placement of lining and to the rheological properties of material. The
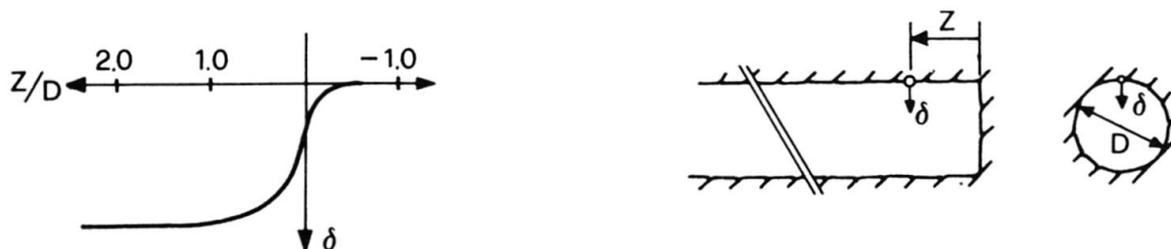deformation of the rock before the functioning the lining has to be estimated.

Fig. 11 Radial displacements δ in the roof of a tunnel
        due to the advance of the face

The difficulties associated with the rheological behaviour of the rock are ob-
viated sometimes by means of the use of a "long term modulus of elasticity" {9}.
It should be realized that here also the computed results are dependent upon the
somewhat over-simplified assumptions made in the analysis. These assumptions make
the results as uncertain as was the case in the behaviour of the loosened rock
mass. In the static computations we shall concentrate less on the actual calcu-
lated values and much more on a parametric study. In keeping with such a "sensi-
tivity analysis" the initial rock deformations (preceding the placement of the
lining) will very often be neglected and first brought to attention in the inter-
pretation of the results. Therefore, the stressing of the lining turns out to be
excessively large. Corresponding to a given E-modulus and strength parameters of
the rock one obtains beyond a certain overburden depth greatly exaggerated re-
sults, which contradict the observations made in tunnelling practice.

The formal procedure in the computations is as follows. First of all - as in the
case of an unlined tunnel - the stress components at the future boundary of the
opening are calculated from the initial state of stress. These are then intro-
duced with negative sign at the boundary (incrementally for plastic material
behaviour) so that the rock and the lining are simultaneously acted upon by
these forces. As a result, re-adjustment of the stresses and deformations in the
rock mass and the extent of possible plastic zones are obtained, as well as the
bending moments and normal forces in the lining. Thus a functional relationship
between the stresses, deformations, initial state of stress, deformation and
strength properties of the rock, shape and size of the underground opening, as
well as the stiffness of the lining is established {4}.
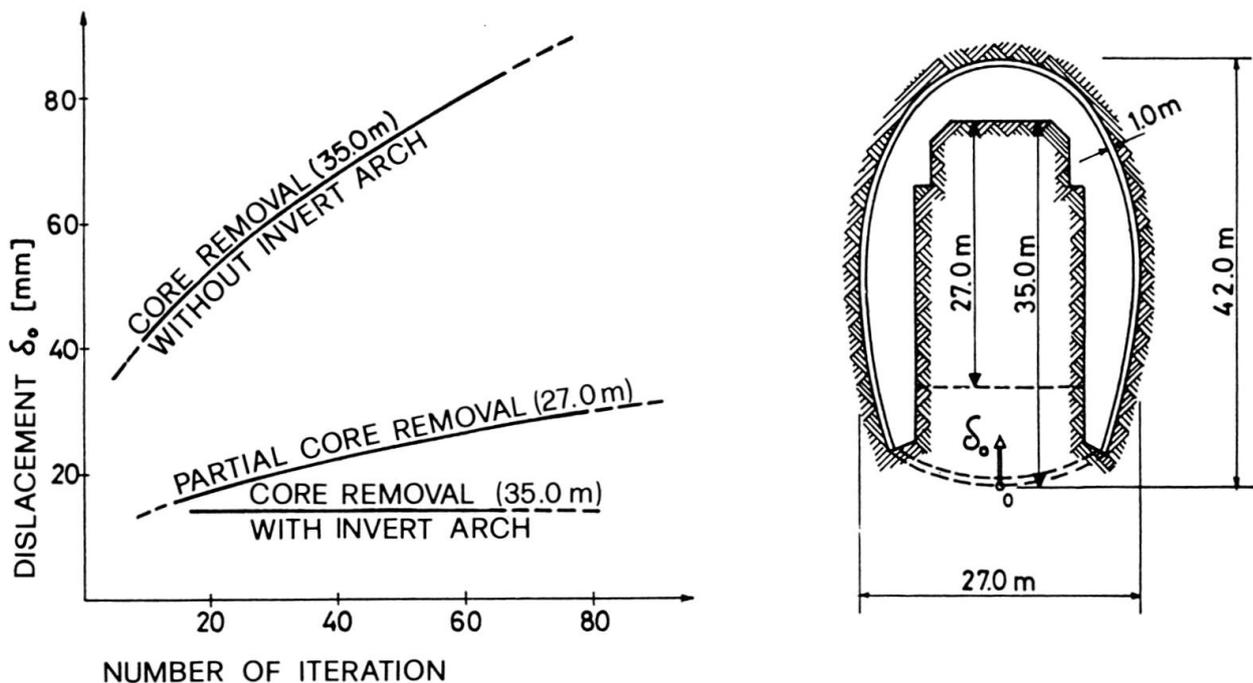
ROCK WITH HORIZONTAL STRATIFICATION

$E_1 = 12000 \text{ N/mm}^2$, $E_2 = 3000 \text{ N/mm}^2$, $G = 2900 \text{ N/mm}^2$
($\nu_1 = 0.3$, $\nu_2 = 0.1$)
$\varphi = 40°$, $c = 0.01 \text{ N/mm}^2$, $\bar{\varphi} = 17°$, $\bar{c} = 0.18 \text{ N/mm}^2$

Fig. 12 Underground power station in stratifield rock. Heaving δ_O of the
         floor as a function of the number of iterations

An example of a stability analysis is illustrated in Fig. 12. In the construc-
tion of a large underground opening for a power station it was necessary to in-
vestigate, amongst other things, the influence of the core removal on the dis-
placement of the rock, which was, for the most part already lined. As the mate-
rial was layered both deformation and strength anisotropy had to be considered.
The unloading due to the removal of the core caused plastic zones to be formed.
In Fig. 12 the convergence of the heaving of the base δ_o for the third and last
unloading increment is shown as a function of the number of iterations. The
figure shows clearly that only a quick placement of the invert arch guarantees
the desired stability. The convergence of deformations has been interpreted as
an indication of the structural stability of the system. Non-convergence was
assumed to indicate a failure mechanism.

For tunnels with approximately circular cross section and under an approximately
hydrostatic state of initial stress the so-called "characteristic-line method"
has rendered worthwhile service {7}. The associated computational model assumes
- in the strict sense - a homogeneous, isotropic, elasto-plastic continuum in
which the initial stresses are in a hydrostatic state of magnitude $\bar{p}$. The model
(Fig. 13) further assumes a circular hole of diameter corresponding to the dimen-
sions of the tunnel. In this way only the simple case of an axi-symmetric stress
analysis has to be dealt with. In order now to be able to calculate the inter-
action between the rock and the tunnel lining a uniformly distributed internal
pressure p is assumed to act on the boundary of the opening. It is quickly re-
cognized that for a value of $p = \bar{p}$ the undisturbed state of initial stress is
maintained and thus no deformations result. A successive reduction of the inter-
nal pressure leads to radial rock deformations. The relationship between the
rock displacement δ at the excavation boundary and the internal pressure p yields
a curve, which represents the characteristic line for the rock. In the case of
genuine rock pressure, after falling below a certain threshhold value of p, failure
occurs in the rock, so that the characteristic line is no longer linear. The
tunnel lining (in the form of a closed ring) also exhibits a certain deformation
response to an all-round pressure. Assuming elastic behaviour the "characte-
ristic line" of the concrete lining is linear in form, as shown in Fig. 13. If
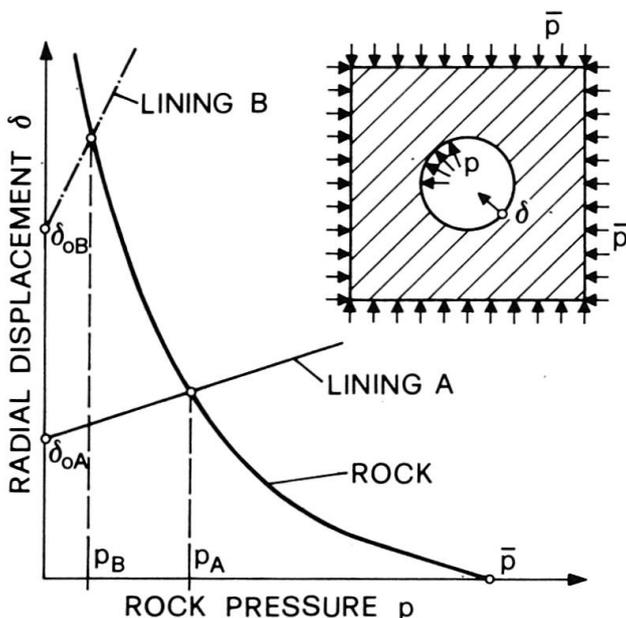the rock has already undergone a radial deformation before placing the permanent

Fig. 13 Rock pressure assessment
by means of the "characte-
ristic - line method"

lining, then the corresponding straight line will begin at that value on the
δ-axis. The intersection point of the straight line with the curve yields the
theoretical value of the rock pressure and the corresponding value of the total
rock deformation. In Fig. 13 we recognize further, that the characteristic line
of the tunnel lining B leads to a lower value of rock pressure (p_B) than that of
A. The reason for this is, firstly, the higher value of permitted deformation of
the rock δ_{OB} and, secondly, the greater deformability of the lining (B) compared
to A. Even with this greatly simplified representation the basic aspects of the
interaction of lining and rock under the condition of genuine rock pressure are
brought out. A better approach to problems in practice may be achieved by
the introduction of visco-elastic and visco-plastic material behaviour into the
mathematical model.

4.3 The interaction of structures

Structural interaction may occur in the construction of parallel tunnels, if one
tunnel crosses another one or if underground openings lie within the zone of
influence of surface structures (Fig. 14). In such cases - especially in subway
design - the method of construction as well as the support measures are fre-
quently determined by deformation and surface settlement criteria. The procedu-
res used in the numerical treatment of such problems have been dealt with else-
where {10}.

A typical example, for which a parametric analysis was necessary, is the stretch
9 of the Munich subway. The particular section discussed here is situated in
the centre of the city {11}. From the point of view of the statical investiga-
tions the following features had to be considered:

- the ground consists of soil with three distinct layers
- during the construction the ground water level is temporarily lowered
- despite the unfavourable ground conditions, the small overburden and the
  size of the underground opening, the tunnel had to be constructed by the
  New Austrian Tunnelling Method. The temporary support consists of gunite
  and systematic anchoring
- permanent support is provided by a 40 cm thick concrete lining. Between the
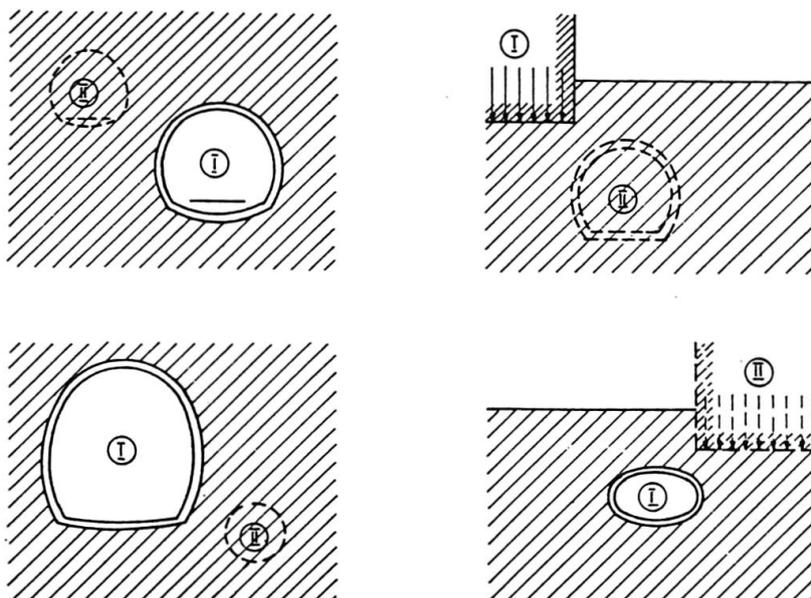 gunite and the permanent lining there is a water-proofing layer.

Fig. 14

Typical cases of inter-
action of an underground
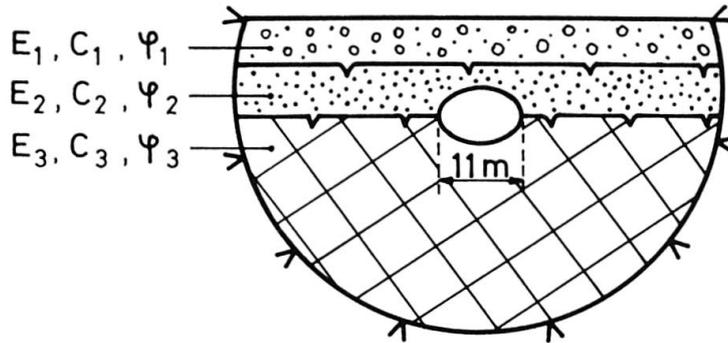opening with other
structures

Fig. 15 Subway Munich: Computational model with three different layers of soil

The project engineer and the responsible authorities decided therefore to investigate the whole problem thoroughly by an exhaustive statical analysis. The computations were carried out using the program STAUB in Zurich. The computational model is shown in Fig. 15. Taking into account the various phases of the construction of the support – gunite plus anchors, gunite anchors plus lining and lining alone (gunite and anchors assumed to be destroyed by aggressive

d1 = no support
d2 = only gunit   EB1 = 20'000   EB2 = 150'000   EB3 = 30 000 N/mm^2
d3 = gunit with concrete vault   EB concrete vault   = 30 000 N/mm^2
d4 = only concrete vault
TYPE OF COMPUTATION : EL = elastic   PL = elastic-plastic

LOADING CASE	COMPUTATION NO.	WATER PRESSURE	LINING d1	LINING d2	LINING d3	LINING d4	E-MOD. GUNIT EB1	E-MOD. GUNIT EB2	E-MOD. GUNIT EB3	ANCHOR	EL	PL	LOADING CASE
I	1			•				•			•		
	2			•				•		•	•		
	3			•				•				•	▼GW
	4			•				•		•		•	
	5		•					•			•		
	6			•			•				•		
	7				•			•				•	
	8	•			•			•				•	
	9				•				•			•	
	10	•			•				•			•	
	11					•						•	
	12	•				•						•	
II	13				•			•				•	EXCAVATION
	14	•			•			•				•	
	15					•						•	
	16	•				•						•	
III	17				•				•			•	FOUNDAMENT LOAD
	18	•			•				•			•	
	19					•						•	
	20	•				•						•	

Fig. 16 Subway Munich: Parametric study

ground water) - two separate linings must be incorporated into the model. Since the layer of water-proofing material greatly reduces the friction acting between the two linings, the connection between the gunite and the concrete lining is simulated by using "pinned" linear members. In this way it was also possible to take into account the hydrostatic groundwater pressure acting on the permanent lining more correctly. In the table in Fig. 16 all the computations are summarized under three loading cases. One recognizes how the influence of the different factors - statical system, loading case, thickness of the lining, elastic moduli of the gunite - have been investigated for elastic and elasto-plastic analyses. The computations for this load case (deep excavation for a building in the vicinity of the completed subway) use for the initial state of stress the stress field obtained in the previous calculations. The excavation is simulated by applying in increments equivalent nodal forces at the boundary of the excavation . For each increment the well-known "initial Stress" approach {5}is used. The displacements caused by the deep excavation are represented in Fig. 17/a. In Fig. 17/b the final moments and the normal forces in the lining are shown. They are obtained by superposition of the results of a previous load case and that of the deep excavation.
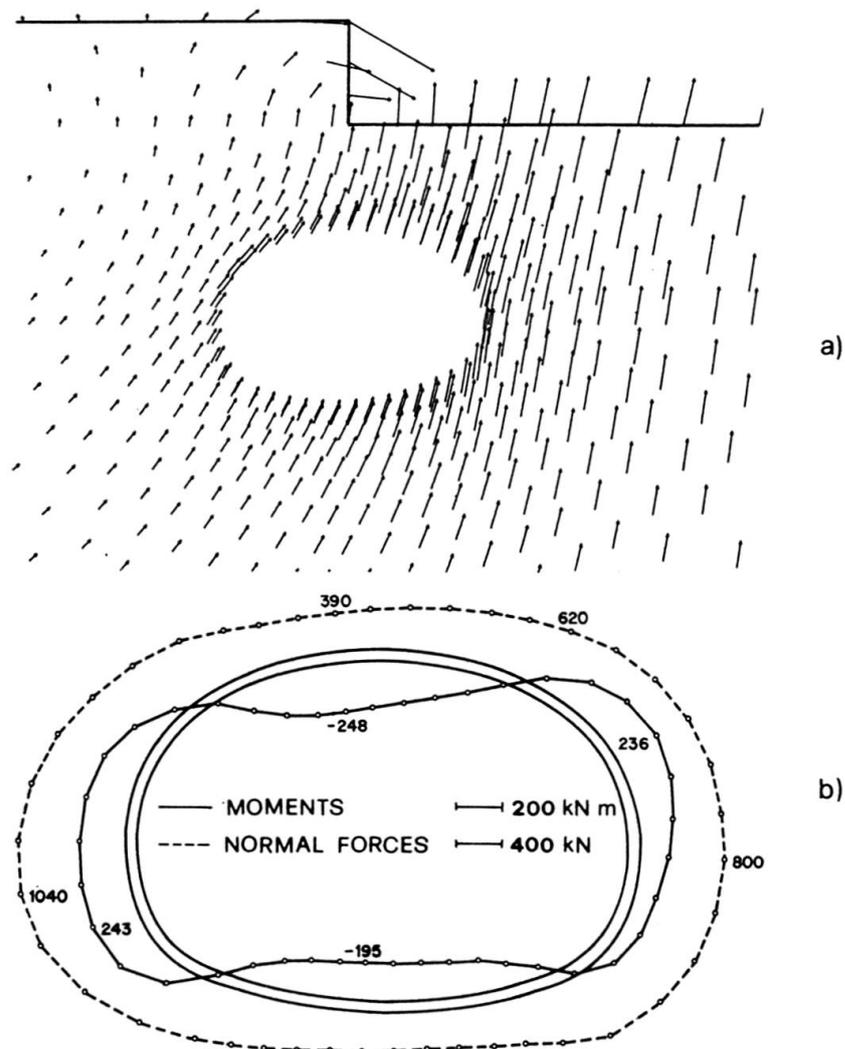

Fig. 17 Subway Munich: Computation No 15, ·(a) displacement field, (b) bending moments and normal forces

5. FIELD MEASUREMENTS IN TUNNELLING

The practical significance of systematic measurements for a given project de-
pends upon the extent to which the results of the continuous observations are
able at all to influence the constructional work. This point is illuminated by
means of two examples. The first concerns the case of shield tunnelling with
lining segments. Here the most important constructional decisions, for instance
deciding upon the shield diameter for the estimated deformations of the lining
ring, or designing the ring segments themselves, have to be made well before the
start of the construction. The observation of the actual deformations of the
tunnel profile, the movements of the surrounding ground or settlements at the
ground surface have the function, mainly, of checking the structural behaviour
with regard to a satisfactory design and proper execution of the works. In this
way shortcomings arising in backfilling the space between the rings and the
ground or concerning insufficient support of the tunnel face can be detected. In
contrast, using the New Austrian Tunnelling Method {13} (with shotcrete and an-
choring as a support), which may in many cases also be applied in subway con-
struction, continuous measurements can serve as feed back signals for the con-
structional process. On the basis of careful statical computations a concept is
worked out for the excavation sequences both in the cross section and along the
axis with the corresponding support measures. If the measurements indicate a sub-
stantial deviation from the anticipated behaviour of the structure, then the most
important corrective measures in the construction can still be applied. The above
comparison of the two methods of construction restricted itself to the possibili-
ties of influencing the tunnelling process by a proper use of measurements and
should in no way be regarded as a general critique of the two methods. Which of
the two methods of construction should be applied in a particular case is deci-
ded of course by safety and economic considerations.

The basic idea of field measurements lies in the optimization of the design and
construction of the underground structures. In other words, the aim is to obtain
adequate safety for a minimum of cost expenditure, whereby the manifold influ-
ence of the construction time is also included in the expenditure. This does not
exclude, to be sure, the conscious decision to accept a calculated risk. Since
the problem of optimization is very varied, the immediate objective of the indi-
vidual measurements may be concerned with quite different aspects, the most im-
portant of which are

- the investigation of material properties and possibly the determination of
  the initial state of stress
- safety control
- the verification of structural response to a specific method of construction
- the comparison of theoretical predictions with the actual structural behaviour.

As a general rule, the above classification of the objectives of measurements
is not rigid. It is intended to indicate the main emphases. It should be noted,
that mostly with the same program of measurements several aims are sought. The
most important thing is that the concept, the execution and interpretation of
the measurements are adjusted to suit the needs of the problem in hand.

5.1 Measured Quantities

The most important quantities to be measured when observing the behaviour of an
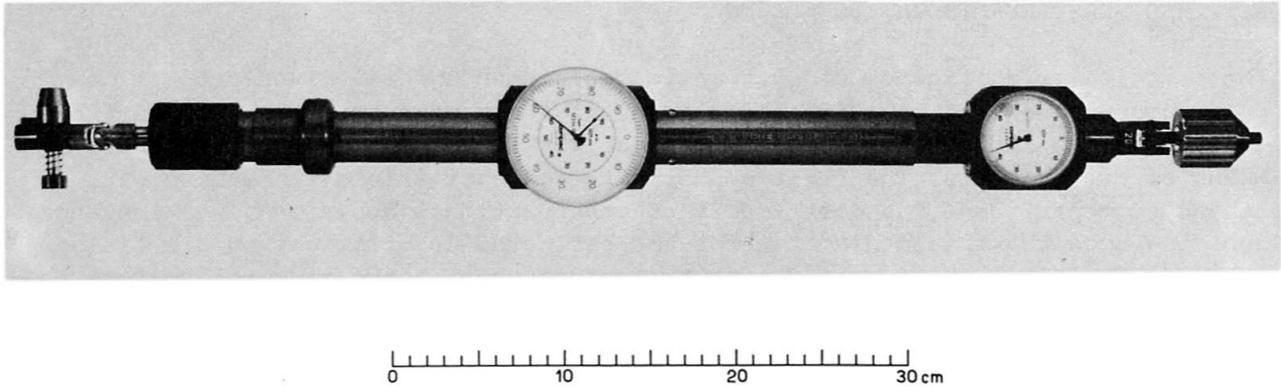underground structure are as follows:

Fig. 18 The Distometer-ISETH with the displacement and the tension gauge

- Convergence of the opening
- Displacements in the rock mass or ground
- Rock or earth pressure
- Settlements of the surface terrain.

By convergence measurements is meant the determination of the relative displace-
ment between a point on the excavation surface with respect to one on the oppo-
site side of the opening. The two points are precisely defined by measuring
bolts. The instruments work on the well-known principle of the tensioned invar
steel wire or steel tape. As such measurements can be carried out quickly they
do not affect the construction activities. An example for a device especially
developed for the needs of underground constructions is shown in Fig. 18. It works
with a tensioned invar wire {13} and permits a high accuracy of the measurements,
which is particularly important when the rate of displacement has to be deter-
mined in a short time. If the wire length is designated by L the mean error m on
a reading is less than $m = 5.10^{-6}$ L. Displacements in the rock mass or in the
ground are generally measured by Borehole Extensometers. By means of such instru-
ments the displacements of various well-defined points along a borehole with
respect to a fixed point are measured. In fact, only the component of the dis-
placement vector in the direction of the borehole axis are determined. The rea-
dings are carried out on the measuring head situated at the mouth of the borehole
and the displacement of the marked points are transferred by steel rods. Displa-
cement measurements of the types described above can provide a great help in
checking if the structure, or its parts, are reaching or have already reached a
condition of stable equilibrium, or if instabilities or inadmissibly large de-
formations are to be expected. Measurements can serve, therefore, as a possible
warning system enabling preventive measures to be introduced in proper time. The
correct interpretation of the observations, i.e. the establishment of warning
levels, however, may present a difficult problem when the displacements increase
steadily in time although with decreasing rate. During the construction of the
Arlberg Tunnel West (Austria) under the difficult conditions of genuine rock
pressure the rate of the measured deformations was found to be the most important
information for the continuous adaptation of the temporary support to the chan-
ging rock conditions {14}. Let us now consider typical results of a convergence
monitoring system revealing the pronounced effects of the excavation sequences
in the vicinity of the measuring section as well as the influence of time de-
pendent deformations (Fig. 19). An initial change in length H is caused by the
attack (1) thus removing the supporting effect of the tunnel face. Due to the sub-
sequent placement of rock anchors (2) there will be a stabilisation of the rock
wall, however, without hindering further movements during the time of the advance
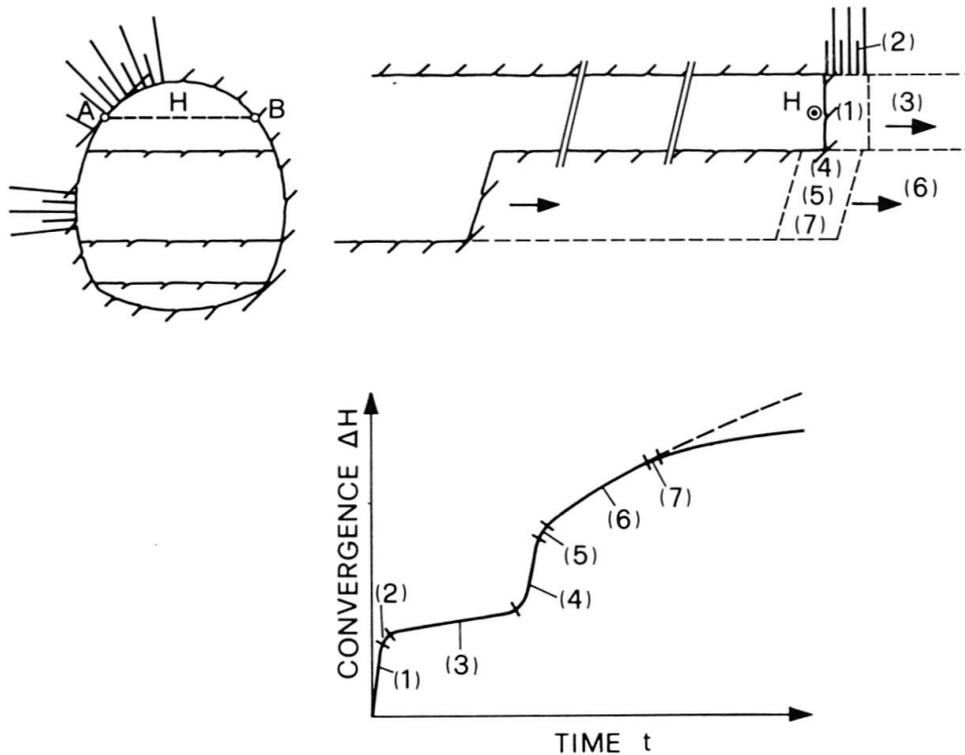
Fig. 19   Influence of construction phases and time on the convergence ΔH

of the tunnel face (3). The bench excavation (4) at the area of the measuring
section will trigger new displacements and an immediate support of the walls by
anchoring (6) will result in stabilisation. This anchoring may, however, turn out
to be unsufficient. As a preventive measure additional anchoring may be necessary
(7) with the result of slowing down considerably the rate of deformation.

Typical examples for extensometer measurements are given in Fig. 20. The curves
show the radial rock displacements in a borehole as a function of the depth.
Based on such observations one can, for instance, determine the nature of rock
pressure, which under given conditions (properties of the rock, dimensions of
the opening, height of overburden, method of construction etc.) is to be expec-
ted. For this purpose measurements in access tunnels, drifts, trial headings etc.
are advisable. From the amount, time variation and spacial distribution of the
measured displacements in the rock an idea of the nature of the present or
future rock pressure can be gained. As already been pointed out, in a
situation with loosening pressure large deformations are generally observed in
the area of the roof, which usually can be brought to a standstill in a short
time with just temporary support measures (Fig. 20/a). In the case of genuine
rock pressure the displacement field is fairly uniform around the opening and
stretches far into the surrounding rock (Fig. 20/b). The deformations continue
to increase over a long period of time and in many cases do not stop until a
permanent lining has been constructed. The third type of rock pressure, namely
swelling pressure is confined to the area of the bottom of the tunnel (Fig. 20/c)
and the resulting deformations exhibit the same character as genuine rock pres-
sure. Extensometer measurements provide useful indications to estimate the
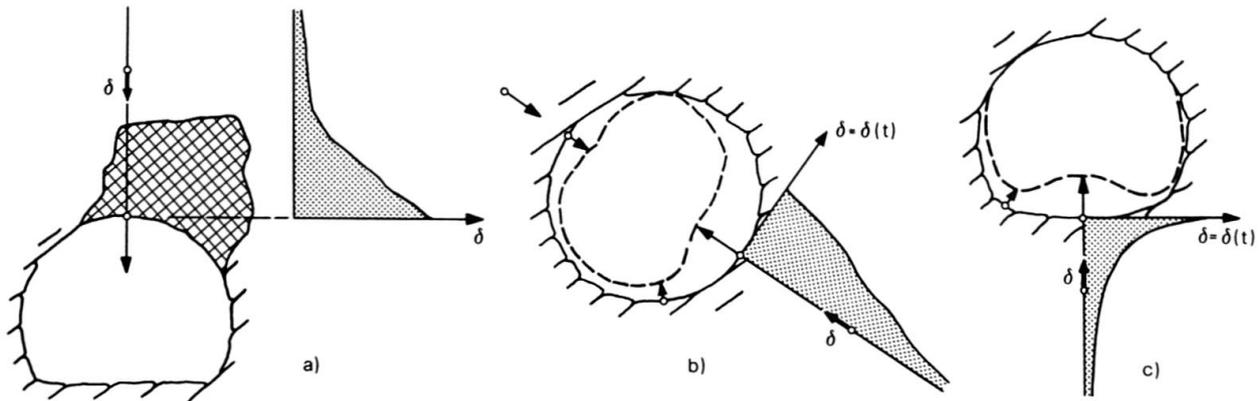swelling potential of the surrounding rock.

Fig. 20 Typical displacement fields associated with different types
of rock pressure
a) loosening pressure, b) genuine rock pressure, c) swelling pressure

Rock or earth pressure exerted on steel supports or on the tunnel lining may be
determined by load cells. The rock pressure is then obtained pointwise along the
circumference of the support structures. One of the most frequently employed de-
vices is the Glötzel-type hydraulic cell. It comprises a flatjack (fluid filled
cushion) and the rock load is measured by balancing the fluid pressure in the
cell.

Another possibility of rock or earth pressure determination is based on the pre-
cise measurement of the deformations of the tunnel lining. As the rock pressure
causes the lining of a tunnel to deform the loads may be back-calculated accor-
ding to well established procedures in structural engineering provided the ma-
terial properties of the support members can be specified with sufficient accu-
racy. This is obviously the case with steel arches. Measurements of the strains
and curvatures at an adequate number of consecutive points along the intrados of
steel arches permit the determination of the bending moments and normal forces
and, in turn, the magnitude of the rock pressure. Thus one obtains simultaneous-
ly three valuable, easily interpretable pieces of information, namely the defor-
mations, the state of stress in the tunnel lining and the rock pressure.

6. CONCLUDING REMARKS

The successful design of large underground openings is based on different sources
of information. The most important among them are geological explorations, soil-
and rock mechanics investigations, statical computations and field measurements
during construction. The way to make use of computer programs and the criteria
for the interpretation of the results obtained are still the subject of some
discussion. This is the main reason for the lack of standard design procedures
in tunnelling. The inherent weak elements in purely theoretical considerations
can, however, be compensated by direct field observations and the sound enginee-
ring experience of the designer.

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