Zeitschrift: IABSE surveys = Revue AIPC = IVBH Berichte

Band: 3 (1979)

Heft: S-10: Basic considerations on the design of underground openings

Artikel: Basic considerations on the design of underground openings

Autor: Kovari, Kalman

DOI: https://doi.org/10.5169/seals-44930

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

Kalman KOVARI

Dr.sc.techn.
Swiss Federal Institute of Technology
Zürich, Switzerland

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 frequency 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 virgin rock or soil due to the creation of the opening. Each step in the excavation process involves a redistribution of stresses in the ground, thus transforming 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 deformations 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.

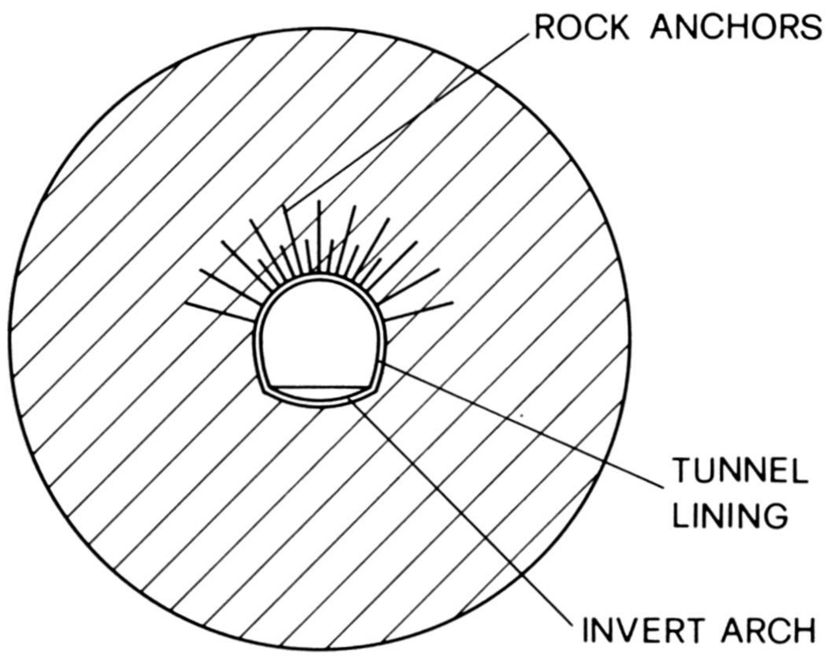


Fig. 1

Tunnel support and rock forming a structural unit

Keeping the rock pressure under control and limiting the deformations respectively 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 mechanics 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 conditions. 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 latter 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 decreasing 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 techniques. Rocks containing clay or anhydrite give rise to special problems in tunnelling. 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 swelling 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 conditions are often, in fact, first known as the underground opening is under construction. This is specially true for deep tunnels, for which borehole explorations 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 composition 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

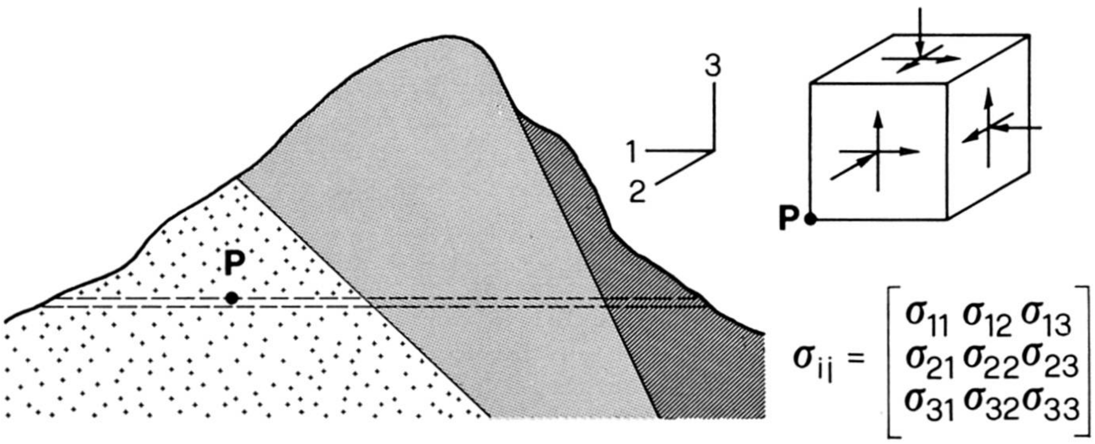


Fig. 2 Initial state of stress in the rock

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 component. 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 unfavourable 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 construction 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

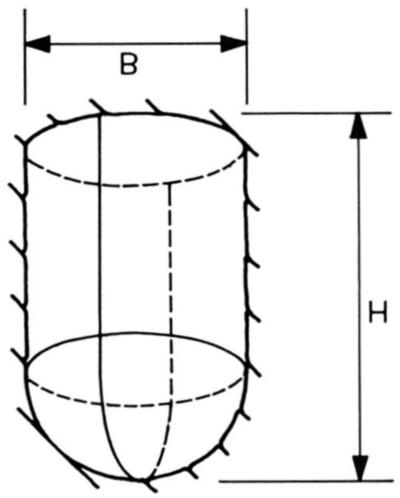


Fig. 3 Large underground chamber with statically favourable shape

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.

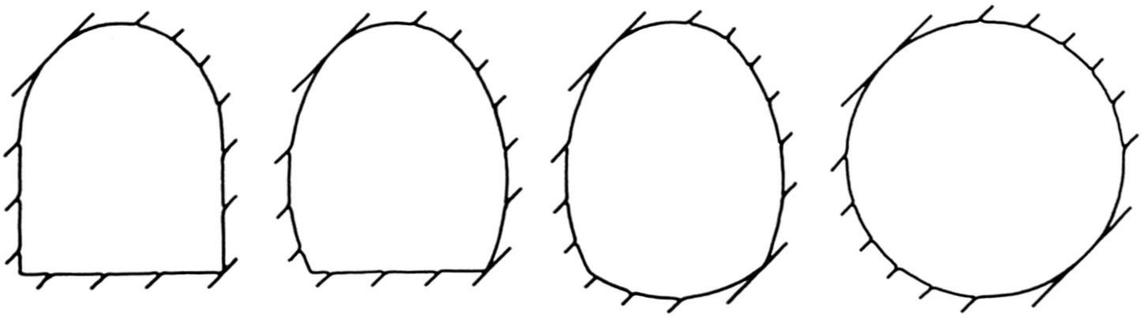


Fig. 4 Possible adaptation of tunnel profile to increasing rock pressure

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 (heading 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 section 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 invert 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

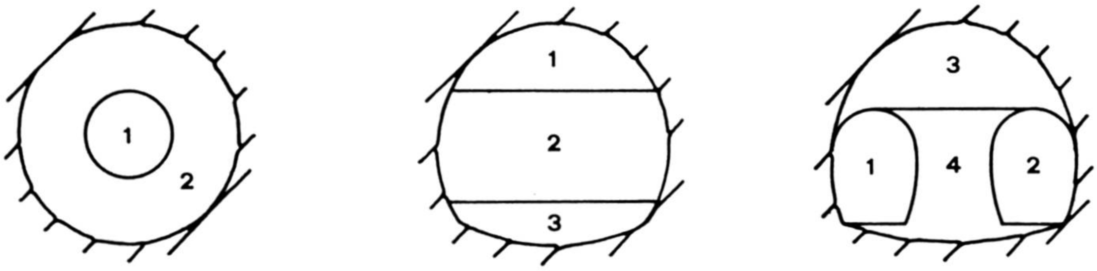


Fig. 5 Examples of multiple stage excavation in the tunnel section

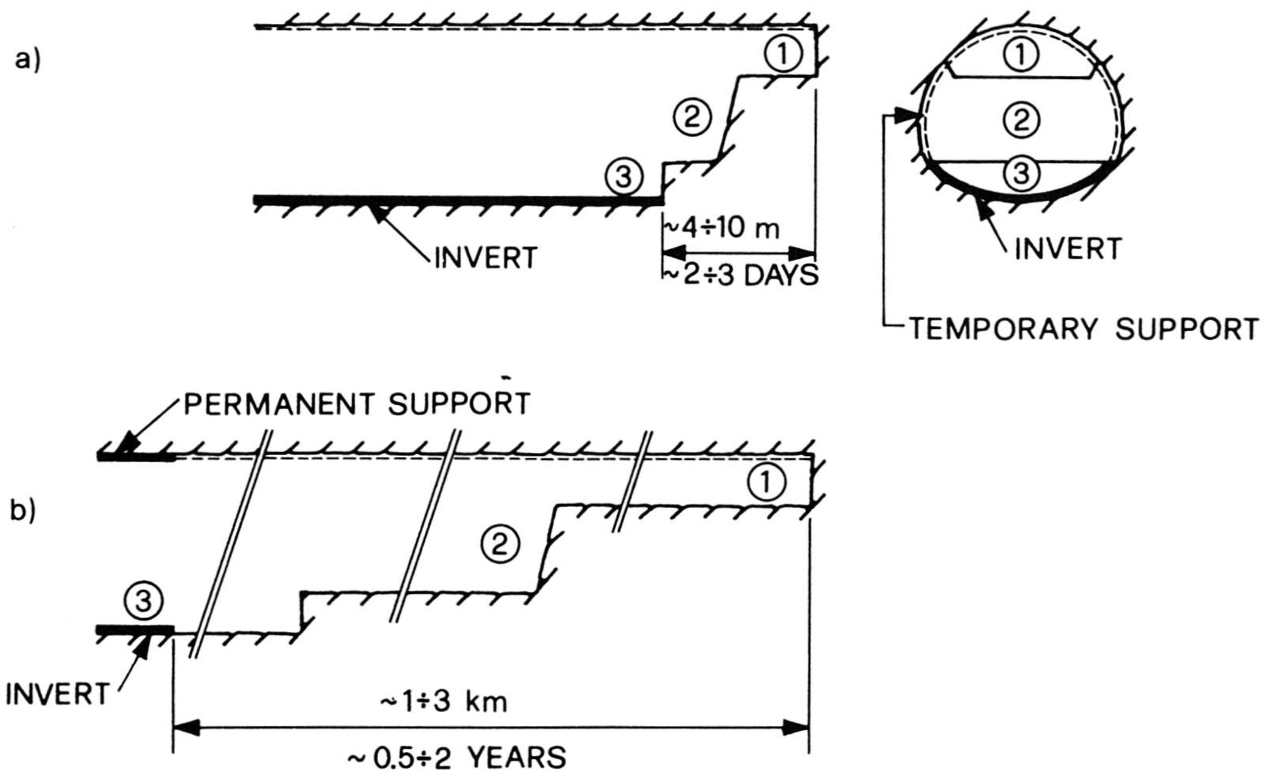


Fig. 6 Placement of an invert-arch
a) close to the face in a subway construction
b) at great distance from the face in a deep tunnel

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 deformations 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 surrounding rock is mentioned here. The rock should not be unnecessarily loosened by blasting, as this would result in a considerable loss in strength. The indisputable 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 underground 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 adequate 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 settlement and deformational criteria. The third group of problems intimately linked with the static analysis is the "interpretation of observations and measurements". 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 geotechnical 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 separately.

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 economically. 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 material 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 inhomogeneous, arbitrarily-shaped body in plane strain condition consisting of anisotropic, elasto-plastic zones of material under various loading conditions {4}.

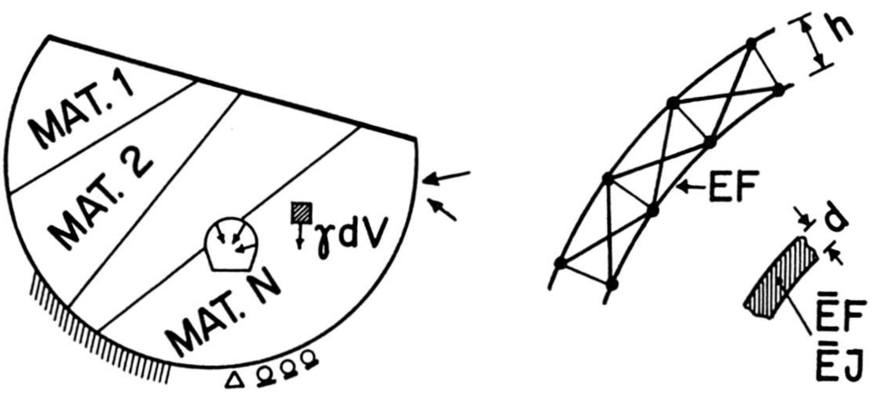


Fig. 7

Structural model with the simulation of a tunnel lining by a truss in a 2-dimensional 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

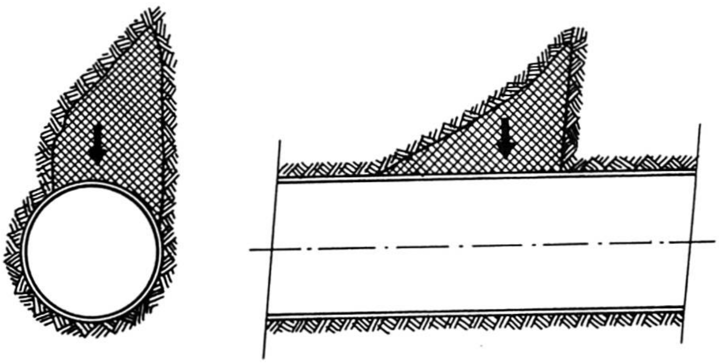
Under favourable rock conditions the opening requires neither a temporary nor a permanent support. For very difficult conditions, however, the opening can be excavated only with the immediate aid of very extensive support arrangements. In quickly changing geological conditions, both extremes are often found at the same site, and there are many situations which lie in between. Thus the computational models must be adapted to the structure section-by-section depending on the type of rock pressure anticipated. A differentiation between rock pressures 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 important 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 fractures 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 weakness. 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 computationally 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 corresponds 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 computational model is that all computed results are dependent upon the estimated loading and are thus questionable. One must consider, however, that by means of such computations useful relationships between important factors, like the shape



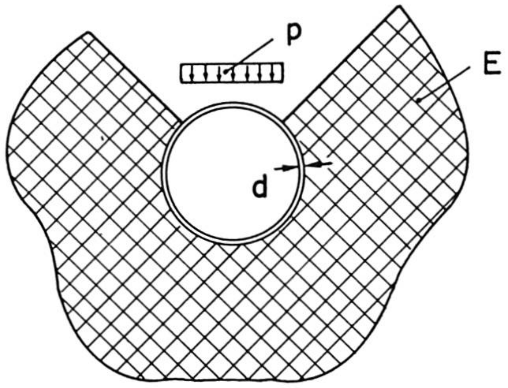


Fig. 8 Embedded lining with assumed dead load p of the loosened rock body

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 parametric 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 pressure 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 parametric study. Several loading cases, different values of E for the deformability 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 stresses 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 overburden 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

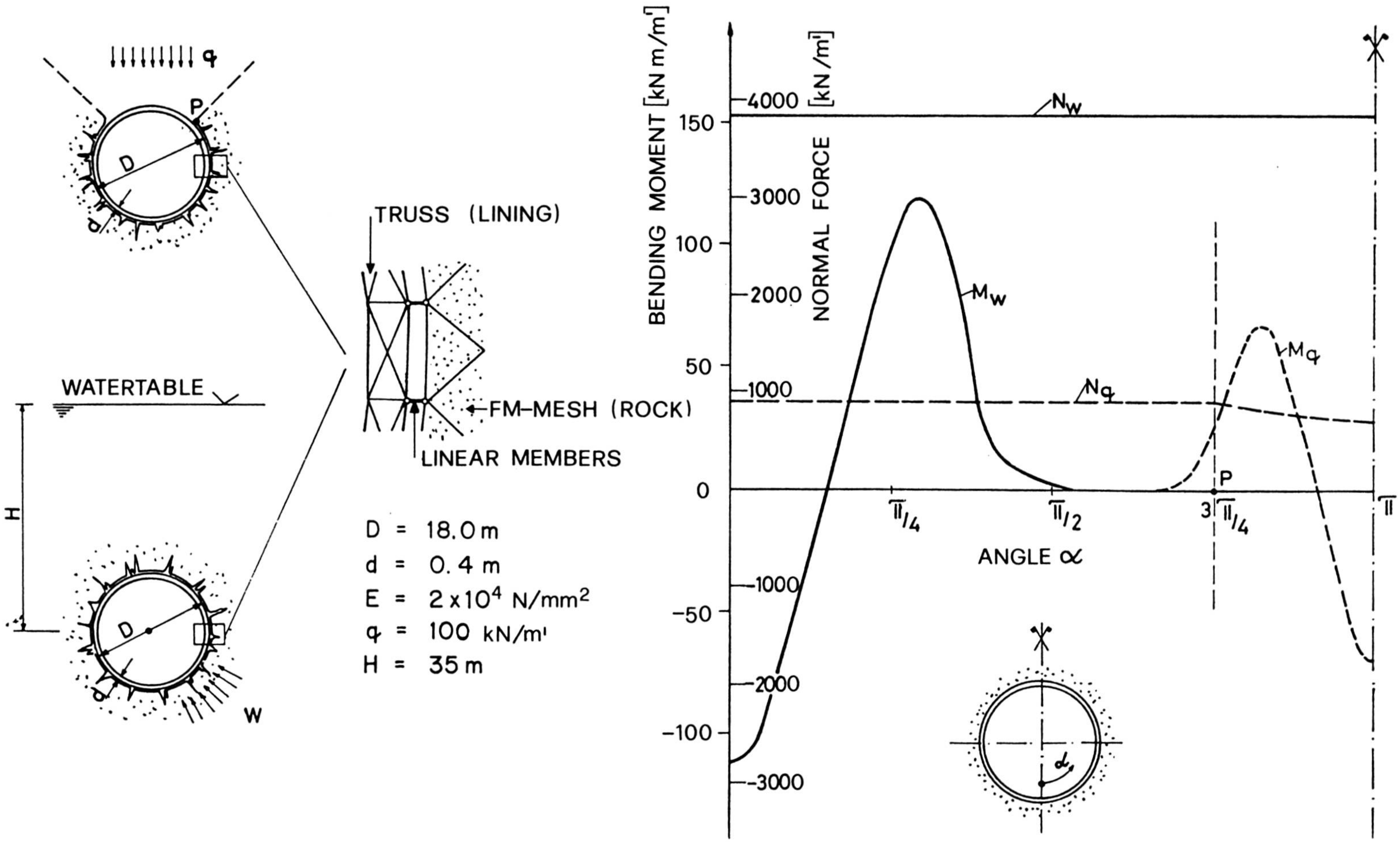


Fig. 9 Large diameter pressure tunnel : moments and normal forces in the lining due to rock load q and water pressure w

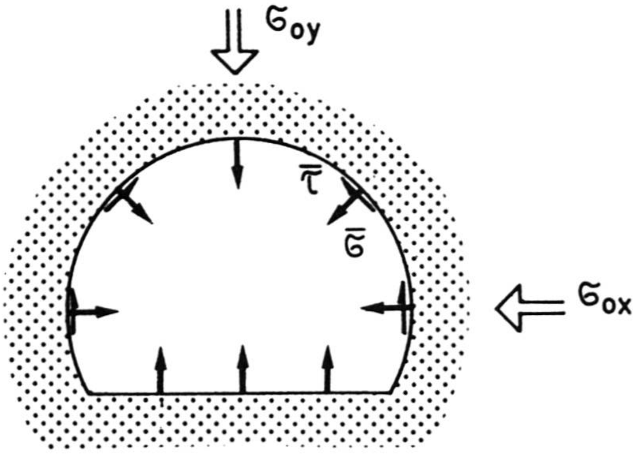


Fig. 10 Removal of boundary stresses $\bar{\sigma}$ and $\bar{\tau}$ according to the initial state of stress (σ_{ox}, σ_{oy})

the plastic zones, the redistribution of stresses and associated deformations, a continuum model is generally used as described in chapter 4.1. First we consider the stress redistribution in the case of an unlined opening. The excavation of a hole leads to a stress-free internal boundary. One proceeds, therefore, (Fig. 10) from the initial stress state (σ_{ox}, σ_{oy}), determines the stress components 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 deformations before the lining begins to function (Fig. 11). This behaviour in the immediate neighbourhood of the tunnel face has been the subject of extensive theoretical 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.

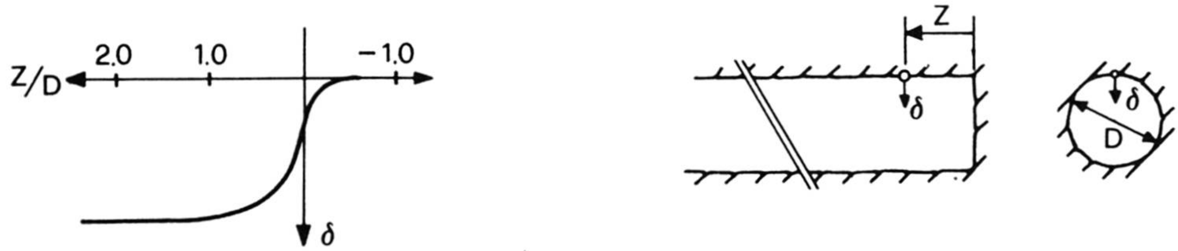
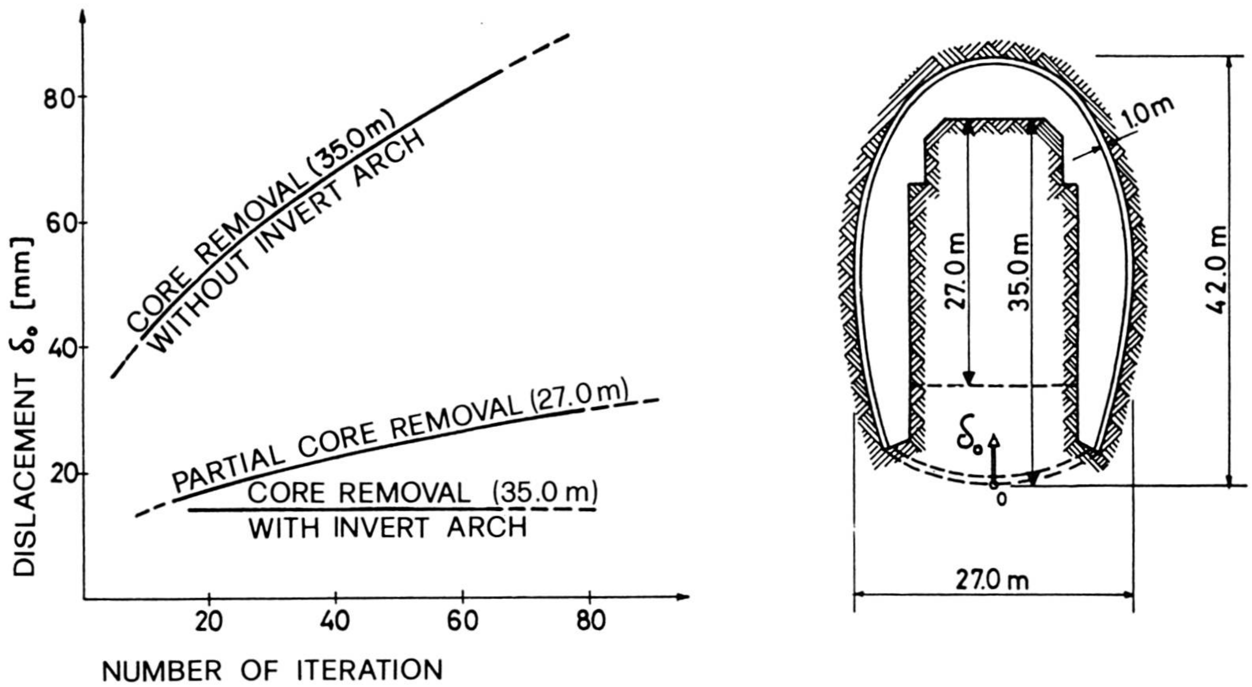


Fig. 11 Radial displacements δ in the roof of a tunnel due to the advance of the face

The difficulties associated with the rheological behaviour of the rock are obviated 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 calculated values and much more on a parametric study. In keeping with such a "sensitivity analysis" the initial rock deformations (preceding the placement of the lining) will very often be neglected and first brought to attention in the interpretation 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 results, 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 introduced 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

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

Fig. 12 Underground power station in stratifield rock. Heaving δ_0 of the floor as a function of the number of iterations

An example of a stability analysis is illustrated in Fig. 12. In the construction of a large underground opening for a power station it was necessary to investigate, amongst other things, the influence of the core removal on the displacement of the rock, which was, for the most part already lined. As the material 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 dimensions 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 interaction 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 recognized 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 internal 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 "characteristic 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

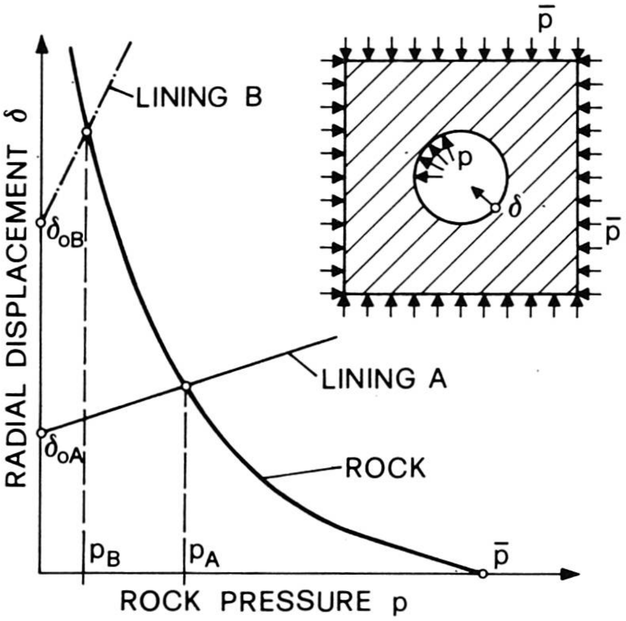


Fig. 13 Rock pressure assessment by means of the "characteristic - line method"

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 frequently determined by deformation and surface settlement criteria. The procedures used in the numerical treatment of such problems have been dealt with elsewhere {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 investigations 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.

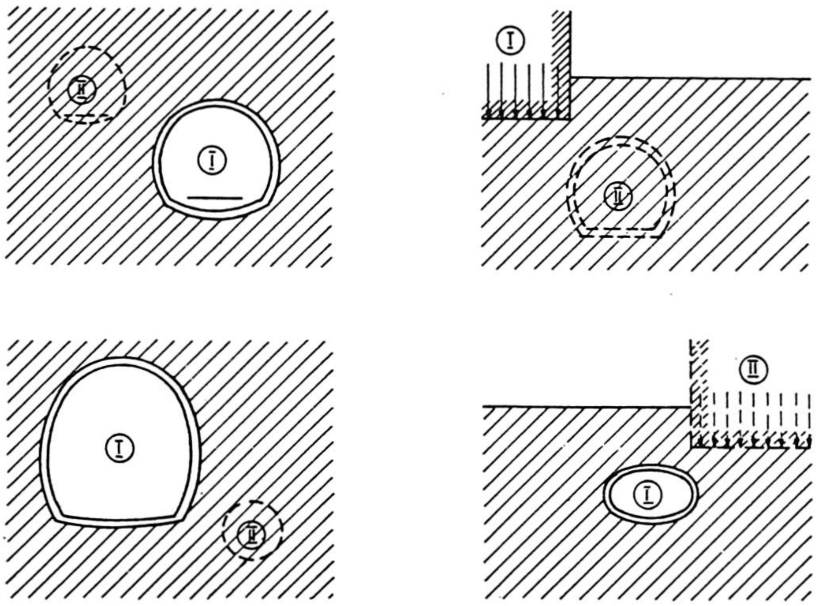


Fig. 14

Typical cases of interaction of an underground opening with other structures

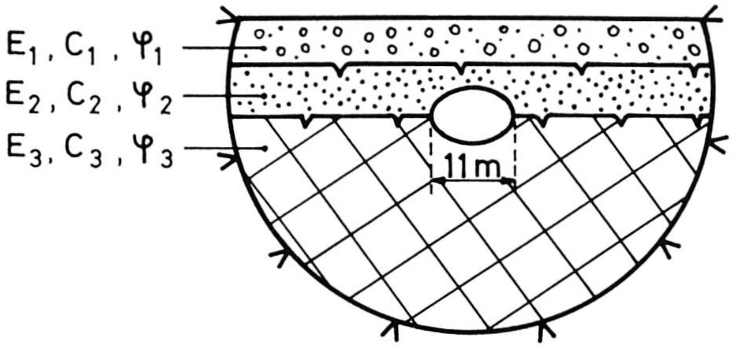


Fig. 15 Subway Munich: Computational model with three different layers of soil

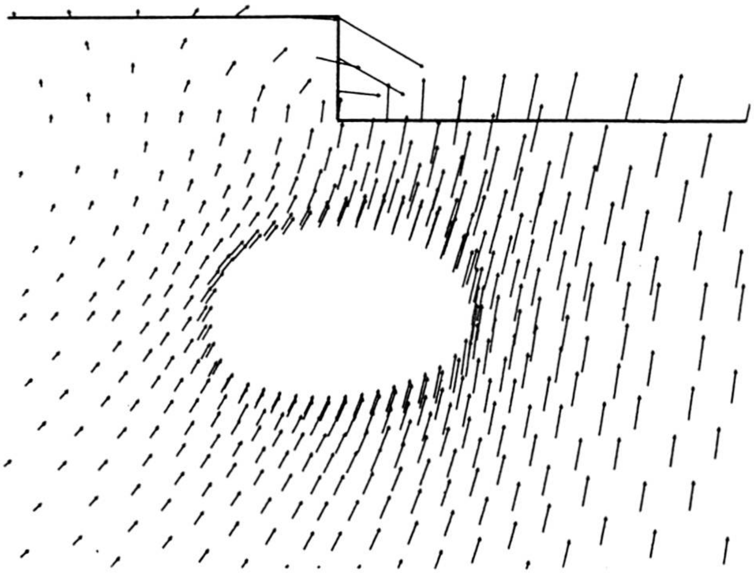
The project engineer and the responsible authorities decided therefore to investigate the whole problem thoroughly by an exhaustive statical analysis. The computations were carried out using the program STAUB in Zurich. The computational model is shown in Fig. 15. Taking into account the various phases 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

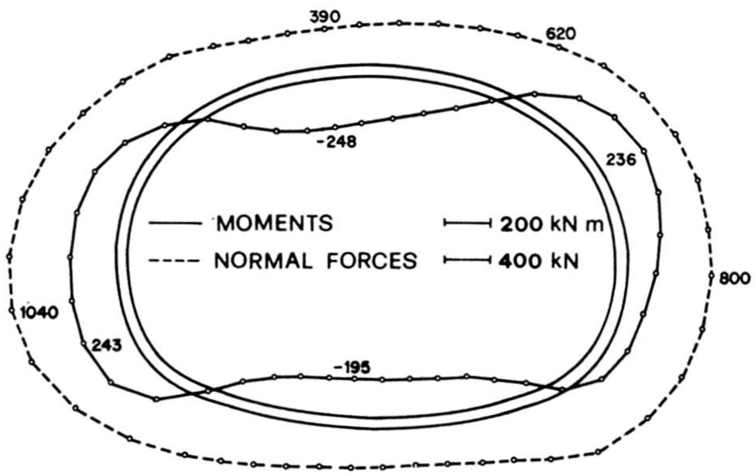
LOADING CASE	COMPUTATION NO.	WATER PRESSURE	LINING				E-MOD. GUNIT			ANCHOR	TYPE OF COMPU-TATION		LOADING CASE
			d1	d2	d3	d4	EB1	EB2	EB3		EL	PL	
I	1			●				●			●		GW
	2			●				●		●	●		
	3			●				●				●	
	4			●				●		●		●	
	5		●								●		
	6			●			●				●		
	7				●			●				●	
	8	●			●			●				●	
	9				●				●			●	
	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.



a)



b)

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 depends 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 anchoring as a support), which may in many cases also be applied in subway construction, continuous measurements can serve as feed back signals for the constructional 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 substantial 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 possibilities 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 decided 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 influence 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 individual measurements may be concerned with quite different aspects, the most important 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:

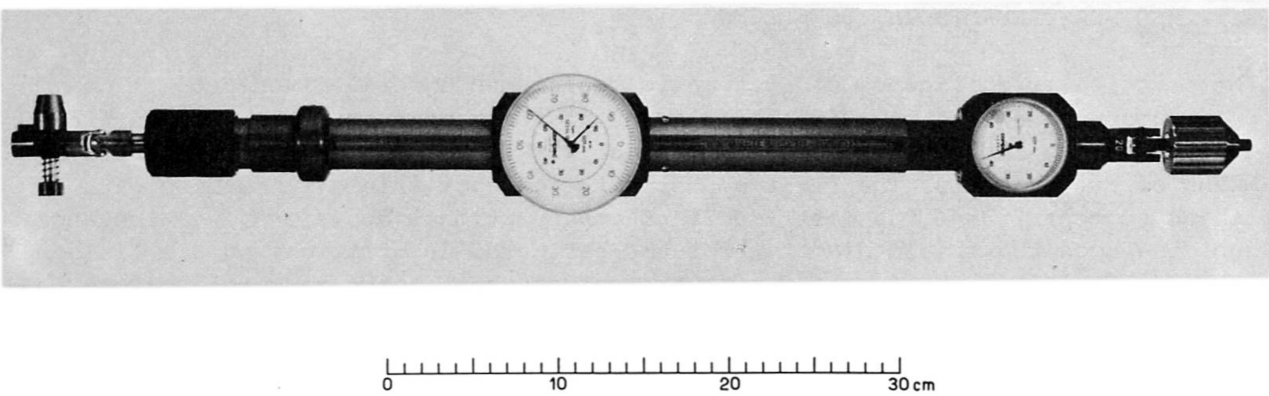


Fig. 18 The Distometer-ISETH with the displacement and the tension gauge

- 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 displacement between a point on the excavation surface with respect to one on the opposite 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 determined 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 instruments 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 displacement vector in the direction of the borehole axis are determined. The readings 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. Displacement 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 deformations 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 changing 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 dependent 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 subsequent 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

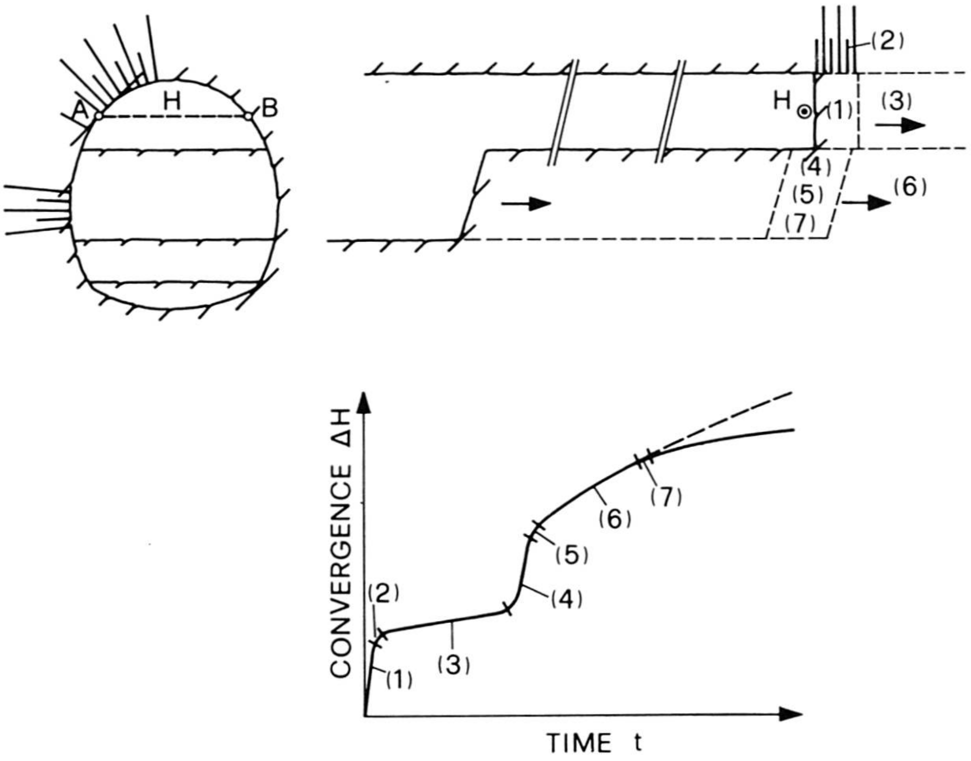


Fig. 19 Influence of construction phases and time on the convergence ΔH

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 expected. 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 pressure. Extensometer measurements provide useful indications to estimate the swelling potential of the surrounding rock.

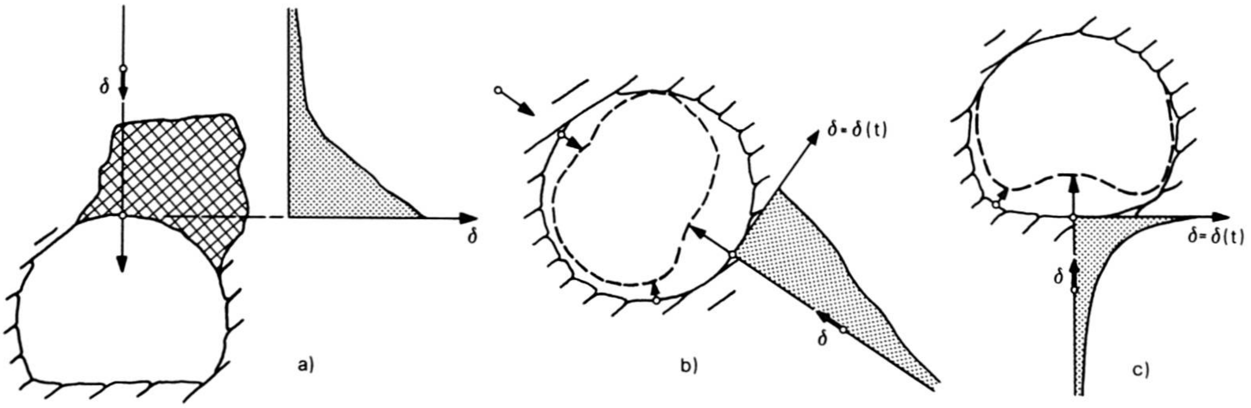


Fig. 20 Typical displacement fields associated with different types of rock pressure
a) loosening pressure, b) genuine rock pressure, c) swelling pressure

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 devices 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 precise 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 according to well established procedures in structural engineering provided the material properties of the support members can be specified with sufficient accuracy. 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 simultaneously three valuable, easily interpretable pieces of information, namely the deformations, 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 engineering experience of the designer.

REFERENCES

{1} Golta, A.: Schwellvorgänge im Planum schweizerischer Bahntunnel, Rock Mechanics, Suppl. 5, 1976

{2} Grob, H.: Schwelldruck im Belchentunnel, Proc. of the Int. Symposium on Large Underground Openings, Luzern, 1972

{3} Schweiz. Ingenieur- und Architekten-Verein: S.I.A. Norm 188, Untertagbau, Zürich, 1975

{4} Kovari, K.: The Elasto-plastic Analysis in the Design Practice of Underground Openings (Chapter 12), Finite Elements in Geomechanics, Gudehus, G. editor, John Wiley & Sons, London, 1977

{5} Zienkiewicz, O.C., Valliapan, S., King, I.P.: Elasto-plastic Solutions of Engineering Problems, Initial Stress, Finite Element Approach, Int. Journal for Numerical Methods in Engineering, 1, 1969

{6} Terzaghi, K.: Introduction to Tunnel Geology, in Rock Tunnelling with Steel Supports (Proctor, R., White, T.), Youngstown Printing Co., Ohio, 1968

{7} Lombardi, G.: Zur Bemessung der Tunnelauskleidung mit Berücksichtigung des Bauvorganges, Schweiz. Bauzeitung 32, 1971

{8} Stroh, D., Chambosse, G.: Messungen und Setzungsursachen beim Tunnelvortrieb im Frankfurter Ton, Strasse, Brücke, Tunnel, 2, 1973

{9} Zienkiewicz, O.C.: Continuum Mechanics as an Approach to Rock Mass Problems, Rock Mechanics in Engineering Practice, Stagg, K.G. and Zienkiewicz, O.C. editors, John Wiley & Sons, 1968

{10} Kovari, K.: Methoden der Dimensionierung von Untertagbauten, Proc. of the Int. Symp. on Underground Openings, Luzern, 1972

{11} Laabmeyer, F.: U-Bahn München, Los 9, Berichte der Studientagung über "Aktueller Tunnelbau in nicht standfestem Gebirge", Schweiz. Ingenieur- und Architekten-Verein, Dokumentation 12, 1975

{12} Müller, L.: Der Felsbau, Bd. III: Tunnelbau Ferdinand Enke, Stuttgart, 1978

{13} Kovari, K., Amstad, Ch., Grob, H.: Displacement Measurements of High Accuracy in Underground Openings, Proc. of the 3rd Congress of the Int. Soc. for Rock Mechanics, Denver, 1974

{14} John, M.: Adjustment of Programs of Measurements Based on the Results of Current Evaluation, Proc. of the Int. Symposium on Field Measurements in Rock Mechanics, Zürich, 1977, Balkema Publishers, Rotterdam

Leere Seite
Blank page
Page vide