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Decision Making in Tunneling Based on Field Measurements

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20.1 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 and strains in the ground, thus transforming the primary state of stress and strain 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.

20.2 THE STRUCTURAL BEHAVIOR OF UNDERGROUND OPENINGS

The tunnel support (lining, anchoring, *etc.*) and the surrounding rock form a unit (Figure 1) which is looked upon as the actual structure in tunneling [1]. In practice, the behavior of this structure is often characterized 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 tunneling. The deformations of the tunnel section and the displacements in the rock together with their time-dependent characteristics, however, are also good indicators and in many cases are practically the only indicators for the behavior of the structure. The protection of the opening against rockfall, keeping the rock pressure under control and limiting the deformations in the most economical way often present the main problems in tunneling. For the solutions of these problems, it must be kept in mind that the behavior of an underground opening depends essentially on the groups of factors shown in Figure 2.

20.2.1 Rock Conditions

The scope of the problems which may arise in tunneling is best illustrated by the fact that tunnels may have to be driven through completely cohesionless soil, hard rock mass or through any intermediate type between these two extremes. The materials in tunneling 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 construction. Generally drill holes or

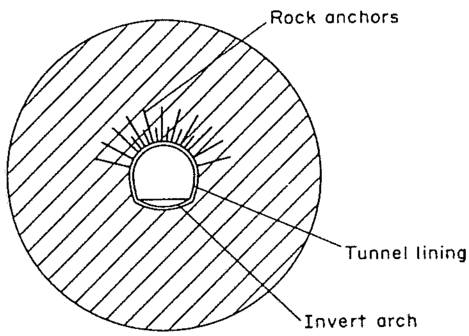


Figure 1 Tunnel support and rock, forming a structural unit

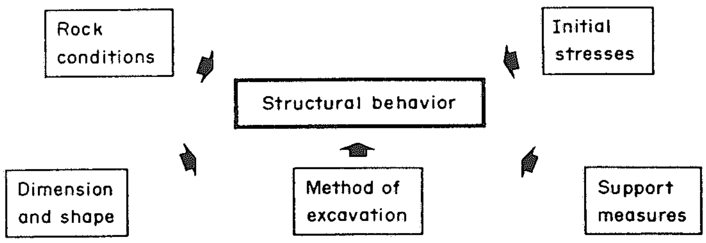


Figure 2 Factors influencing the structural behavior of a tunnel

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 stratification, schistosity 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 or on full scale 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 substantially affect the progress of excavation. The water may reduce the strength of the material by decreasing its cohesion or by the development of pore pressure decreasing the effective normal stresses. When tunneling in saturated soils, special measures, often very expensive, must be taken in order to prevent infiltration and to stabilize the ground, for example grouting, jet grouting, groundwater lowering, utilization of compressed air and hydroshield or ground freezing techniques. Rocks containing clay or anhydrite give rise to special problems in tunneling. 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 tunneling practice, unconstrained heave of up to 70 cm may occur [2] and swelling pressures of up to 3.5 MN m^{-2} have been reported to act on the invert arch [3].

Many of the unexpected difficulties that arise in tunneling 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 the 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 behavior may be expected to vary. Here, not only statical but also purely constructional considerations can be important. The greater the degree of mechanization in the method of construction, the more important possible extreme cases in the material occurrence become. For instance, when using the shield tunneling 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 with anchors and shotcrete support is not only limited by poor rock quality (too short a 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.

20.2.2 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 (Figure 3). There are two ways in which the initial stresses may give rise to difficulties in tunneling. 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 the closure of the opening 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

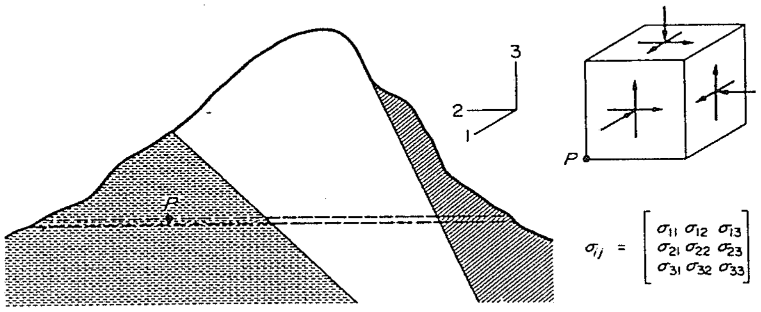


Figure 3 Initial state of stress in the rock

stress–strain relationship and the tectonic forces which 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 [4]. Unfortunately, these conditions are as a rule 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.

20.2.3 Dimensions and Shapes of Underground Openings

The relationship between the span of the opening and the average joint spacing is in many cases decisive for stability considerations (Figure 4). With increasing span D, or D/d respectively, the influence of the jointing becomes more marked and the probability of an unfavorable joint combination, which could give rise to a rockfall, increases. Thus, in the case of a subway through jointed rock the construction of stations generally requires special considerations, even when the single track tubes might be left completely unsupported. In the particular 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 the span of the tunnel to the height of the overburden being also an important factor. If this ratio is less than one it is not possible to develop a noticeable arching effect in the soil, not even in heavily jointed rock. Especially large dimensions in the construction of tunnels or rock chambers are, from the point of view of safety and economy, only possible 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 (Figure 5). Statically, this shape is very favorable, 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

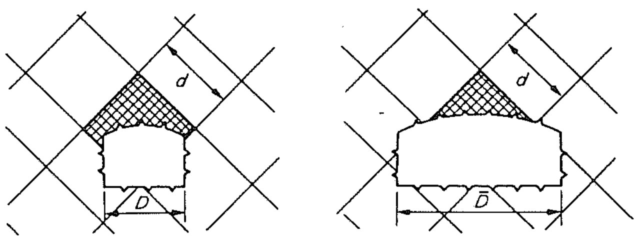


Figure 4 Influence of the span D on the stability in jointed rock

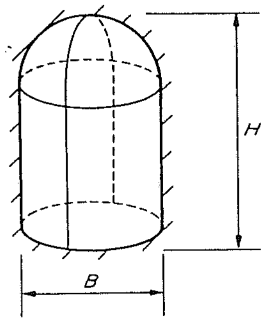


Figure 5 Large underground chamber with statically favorable shape

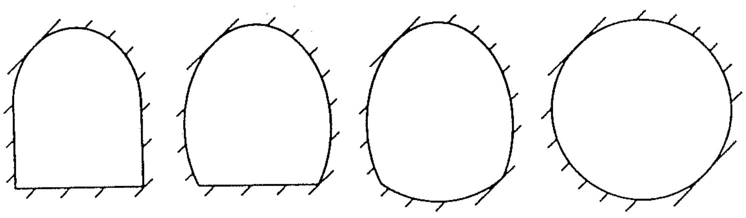


Figure 6 Possible adaptation of shape to increasing rock pressure

nuclear power plants. The shape of a section is also important in the case of a tunnel. However, as 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 Figure 6.

20.2.4 Method of Construction and Support Measures

The method by which the opening is excavated along its longitudinal direction and in its 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 into 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, the profile must be excavated in two or more stages (Figure 7), 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 for enlarging the section to the full profile, thus providing a useful means of rock exploration. Depending on whether the problem is to control the rock pressure or to limit the displacements in the neighborhood 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 Figure 8. For the subway tunnel in Figure 8(a), in order to avoid undesirable settlements of buildings in its vicinity, the invert arch 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, at a distance of less than one tunnel diameter a closed ring is formed which is statically extremely efficient. In a tunnel situated at great depth (Figure 8b), 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 a year to develop. Any further deformation that might occur can then be safely prevented using a suitable closed ring shaped

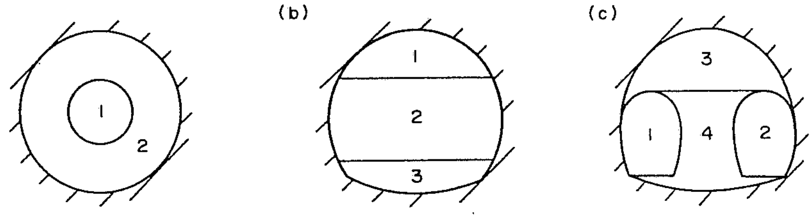


Figure 7 Examples of multiple stage excavation in the tunnel section: (a) pilot tunnel with boring machine, (b) head and bench, and (c) side drift tunneling method

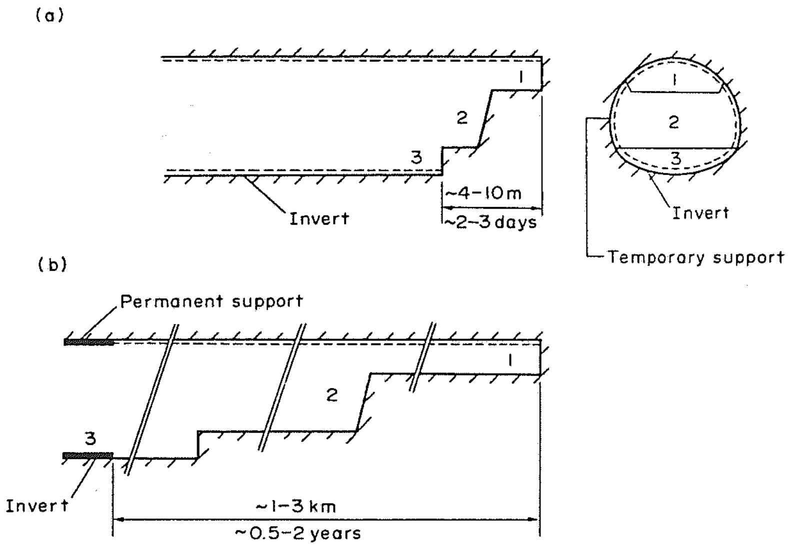


Figure 8 Placement of the invert arch: (a) subway close to the face and (b) deep tunnel at great distance from the face

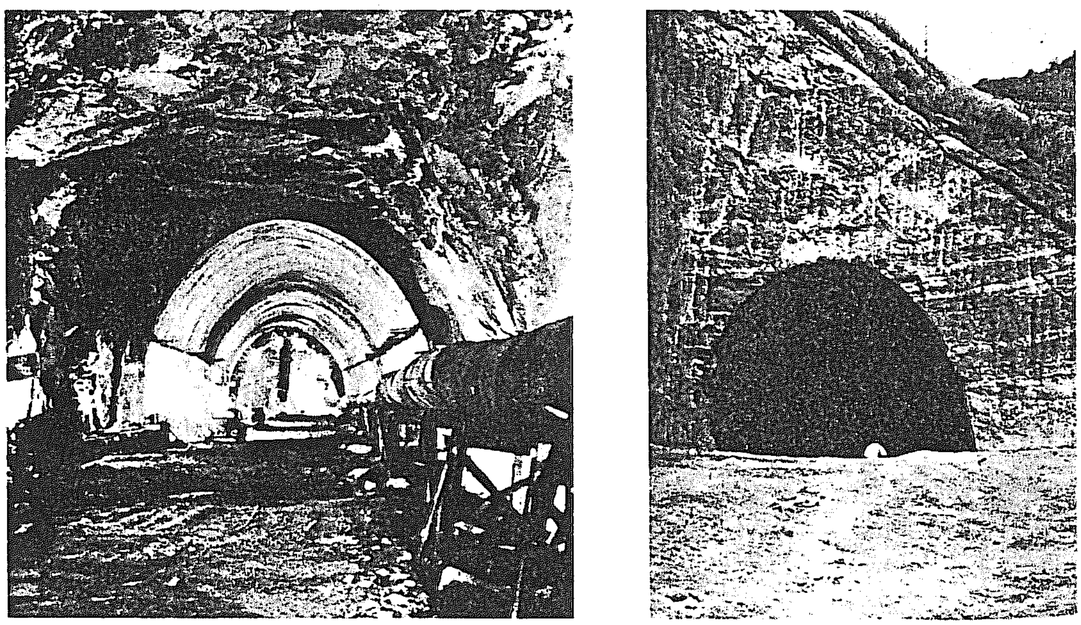


Figure 9 Two different blasting techniques applied in the same rock at the construction of adjacent roadway tunnels

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 of strength. In many instances, heavy lining is necessary only because of poor blasting work. Such a case is shown in Figure 9 together with a tunnel in the same rock but with a smooth rock surface and with no support at all. The indisputable advantage of blast-free mechanical excavation methods is that they do not affect the *in situ* rock quality around the opening.

In summarizing the above, it generally holds true 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 behavior of the underground opening.

20.3 THE PROCESS OF DECISION MAKING IN TUNNELING

In order to obtain a safe and economical structure the engineer has to make decisions on the following items: (i) location, alignment, shape and size of the opening; (ii) method of excavation, both in the section and in the longitudinal direction; (iii) support measures, temporary and permanent; and (iv) dewatering, ground improvement, *etc.*

Decisions are required prior to, during and, in exceptional cases, also after construction. It must be emphasized that the decisions are not only a matter of purely theoretical consideration but in many cases they are somewhat restricted by contractual aspects.

The technical criteria (Figure 10) for correct decisions may basically originate from the safety of the opening during construction and during its service life or from displacement restrictions and in some cases from both.

The sources of information for the structural decisions are: (i) geological explorations, field tests; (ii) laboratory investigations; (iii) statical computations; (iv) field measurement; and (v) the engineer's own experience.

Again the flow of information generally extends from the initiation of the project up to its completion. Modern tunneling is characterized by the systematic use of all sources of information in a balanced manner. A clear understanding of the factors influencing the behavior of an underground opening under specific conditions can only derive from the engineer's own experience and from his theoretical knowledge.

Experience manifests itself in good structural judgement. Together with laboratory investigations, statical computations and field measurements it forms the basis for decision making, both at the planning and the construction stage. To what extent such modern aids should be applied on a given project depends solely on the nature of the problems that arise. In the following, an attempt will be made to give an up to date survey of the possibilities and limitations of field observation techniques. Computational methods as a design aid in tunneling have been discussed elsewhere [5]. Here we only want to point out that by means of statical computations an analytical prediction of the structural behavior of the opening is obtained. 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 uncertainties. Measurements, on the other hand, enable its behavior to be observed directly, without the actual mechanism which gives rise to its behavior necessarily being completely illuminated. The measurements are usually carried out during the constructional phase and if carefully planned and executed they give a true picture of the behavior of the structure. From

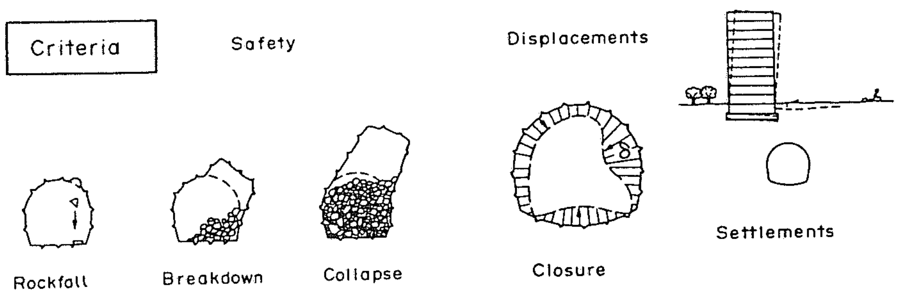


Figure 10 Criteria for decision making

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.

20.4 FUNDAMENTALS OF FIELD INSTRUMENTATION

The practical significance of systematic measurements for a given project depends upon the extent to which the results of the continuous observations are able to influence the constructional work. This point is well illustrated by means of two examples. The first one concerns the case of shield tunneling with lining segments. Here, the most important constructional decisions, for instance deciding upon the shield diameter based on the anticipated soil movements and lining deformations, or designing the 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 mainly have the function of checking the structural behavior with regards 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. Using a tunneling method with shotcrete and anchoring as a support, which may in cases of favorable ground conditions also be envisaged in subway construction, extensive measurements can really serve as feedback signals for the constructional process. Here, 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, and for the corresponding support measures. If the measurements indicate a substantial deviation from the anticipated behavior 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 tunneling process by a proper use of measurements and should in no way be regarded as a general evaluation of the two methods. Which of the two methods of construction should be applied in a particular case is decided, of course, by economy and the attainable progress in advancing the tunnel.

20.4.1 The Purpose of Field Measurements

In general the real purpose of field measurements lies in the optimization of the design and execution of 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 costs. This does not exclude, however, the conscious decision to accept a calculated risk. Since the problem of optimization is very varied, the immediate objective of the measurements themselves may be concerned with quite different aspects, the most important of which are: (i) the safety control; (ii) the investigation of material properties and possibly the determination of the initial state of stress; (iii) the verification of structural response to a specific method of construction; and (iv) the comparison of theoretical predictions with the actual structural behavior.

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

20.4.1.1 Check on the safety

As a rule, completed underground structures exhibit an excessive safety. On the other hand during construction a variety of tunnel hazards may occur which emphasize the importance of safety considerations. Since it is very difficult, however, to quantify a safety concept, the tendency in tunneling is often to speak of safety simply in a qualitative sense. Systematic measurements can provide a great deal of help here too, since, for example, using observed deformations it can be estimated 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, may, however, present a difficult problem when the displacements increase steadily in time but with a decreasing rate.

If only small deformations are permitted in the vicinity of a tunnel, as is often the case in subway construction, then not only the safety of the underground opening itself but above all that of the neighboring structures is of prime interest. Systematic displacement measurements are most frequently employed for safety checks.

20.4.1.2 *The investigation of material behavior*

The deformational properties of the material on a small scale can be estimated using tests such as a borehole dilatometer or the loading plate of a flat jack. In the tests an active loading is applied to the rock and the resulting deformation is measured. From the observed load–deformation diagram and with the aid of the theory of elasticity (with very simplified assumptions) a so-called deformation modulus of the surrounding rock is estimated. An essentially different concept of measurement is based on the realization that by excavating underground openings, such as galleries, tunnels or caverns, the rock mass is unloaded on the scale of the structure itself. To be more exact, it is a question of changing from the initial to the secondary state of stress, which is accompanied, of course, by deformations. By measuring these deformations and with an assumption regarding the initial state of stress it is possible with the aid of a suitable computational model to calculate the 'global deformability modulus' for the rock, which may yield an important indication of the overall rock quality. Although this method of back analysis has its limitations, it gives useful information about the *in situ* deformation characteristics of the material on the scale of the structure itself.

In many cases not only a quantitative assessment of the deformation properties is sought, but also a technological characterization based on measurements. One might, for instance, want to find out the nature of rock pressure, which is to be expected in a particular formation and under given conditions (dimension of opening, height of overburden, method of construction, *etc.*). For this purpose measurements in access tunnels, drifts, trial headings, *etc.* are advisable. From the amount, time variation and spatial distribution of the measured displacements at the boundary of the tunnel excavation and in the rock some clues for the nature of the present or anticipated rock pressure can be gained. 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 (Figure 11a). In the case of genuine rock pressure the displacement field is fairly uniform around the opening and stretches far into the surrounding rock (Figure 11b). The deformations continue to increase over a long period of time (years) and in many cases do not stop until a permanent lining has been constructed. The third type of rock pressure, namely swelling pressure, only occurs in rock containing clay minerals (illite, montmorillonite) or anhydrite. The volume increase (swelling) due to absorption of water might reach such proportions as to render the structure inoperative, if no special precautions are taken. Experience shows that swelling is confined to the area of the bottom of the tunnel (Figure 11c), and the resulting deformations exhibit the character of genuine rock pressure. Field measurements also provide useful indications here to estimate the swelling potential of the surrounding rock or the swelling pressure, if the deformations are prevented by an invert arch construction.

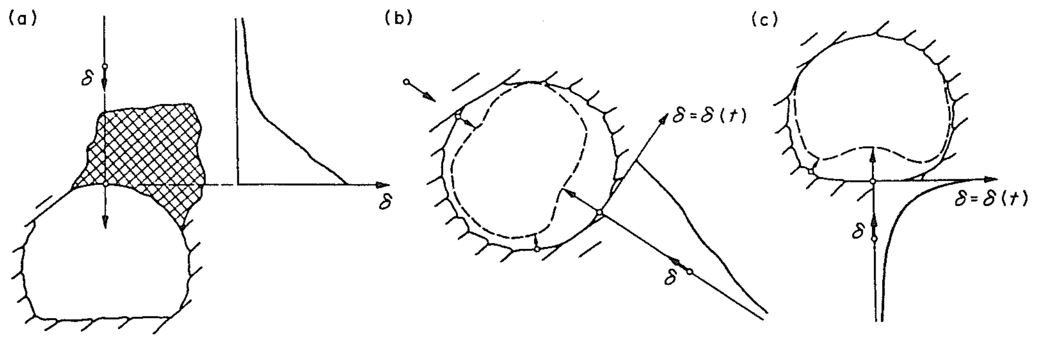


Figure 11 Typical displacement fields associated with different types of rock pressure: (a) loosening pressure, (b) genuine rock pressure, and (c) swelling pressure

20.4.1.3 Verification of the effectiveness of a particular constructional method

The optimum execution of a particular constructional concept can only be achieved, in many cases, if individual aspects like the span of the unsupported roof section, the enlargement of the cross section, the arrangement and the capacity of temporary supports, the time for introducing the permanent lining, *etc.* are determined on the basis of *in situ* measurements. The greater the uncertainty of the geotechnical prediction, whether it is due to inadequate site investigation or to the absence of sufficient experience in working in the given rock, the greater the flexibility that one should have to be able to make correct engineering decisions during construction. By means of a suitable monitoring program and statical considerations one can then check the effectiveness of the specific constructional measures decided upon, and thus, while preserving adequate safety, the object can be more economically constructed.

20.4.1.4 Comparison of theoretical studies with observed behavior

Here, primarily the verification of the theoretically assumed behavior mechanisms is implied. The selection of the physical quantities to be measured and the arrangement of the instruments are based on careful preliminary investigations of a theoretical nature. The computational results do not agree numerically, as a rule, with the measured values, but by varying the parameters and with the aid of several computer runs a better agreement can be achieved. However, if fundamental deviations between theory and reality occur this indicates that some factors, which because of too great a simplification of the model were left out of consideration, are in fact of greater significance than was originally assumed. One only has to think of the time effect, for instance, which is neglected in the usual assumption of an elasto-plastic continuum for the rock mass, or the influence of a complex three-dimensional state of stress, which cannot be considered in a conventional plane strain analysis, which is generally used in statical analysis.

The above classification of the objectives of measurement is only intended to point out the most essential features, as in many cases several aims are envisaged with the same measurement program.

20.4.2 The Measured Physical Quantities

Depending on the particular problems, the observations most frequently refer to one or to a group of the following physical quantities: (i) strains; (ii) relative displacements; (iii) absolute displacements; (iv) changes in curvature (in tunnel lining); (v) stresses in lining and in rock mass; (vi) rock or earth pressures on tunnel lining, forces in rock anchors; and (vii) piezometric heads. When planning a measuring program some sound principles have to be followed in order to obtain useful results for practical purposes with a minimum of cost expenditure.

20.4.3 Principles for Field Measurements

The main principles for field instrumentations and field measurements are as follows.

(i) Correct formulation of the structural problem, the solution of which requires observations.

(ii) Selection of the most sensitive physical quantities.

(iii) Assessment of the order of magnitude of the measured quantities; conclusions with regard to required accuracy.

(iv) Selection of measuring techniques, instruments, location of measuring sections, reading program.

(v) Assessment of possible sources of error in the readings well in advance.

(vi) Application of monitoring with overlapping results for complex situations.

(vii) Employment of reliable instruments and competent personnel only.

(viii) Continuous data processing, establishment of tentative emergency levels, correct flow of information.

Experience shows that when observing these principles, field monitoring really turns out to be an invaluable aid in the design and execution of underground works.

20.5 DECISION MAKING IN TUNNELING BASED ON FIELD MEASUREMENTS: CASE HISTORIES

With the help of examples chosen from tunneling practice, the basic considerations given above will be further discussed in the following sections. When dealing with case histories the actual problems arising in the various projects will be briefly formulated, the applied monitoring technique referred to and the relevance of the obtained results to the constructional problems discussed. The measuring techniques used and the associated instruments are described in the literature [9, 13–17].

As a rule, the authors give preference to displacement measurements (convergence of the opening or movements in the rock), since in a mathematical sense they represent integrated quantities and are basically not subject to local effects. Stresses, strains or changes in curvature, on the other hand, are differential quantities, whose validity is limited to local regions. When being measured, therefore, they should be observed at several successive points, so as to obtain their distribution over a sufficiently great area. In this way the predictive value even of differential quantities can be substantially improved.

20.5.1 Decision Making in Tunneling Based on Convergence Measurements

The measurement of convergences, *i.e.* of the changes in distance between two points of the excavation or lining surface, is one of the simplest and least expensive operations. In Figure 12 typical applications are shown for a tunnel with different construction sequences and for a circular tunnel section. Figure 13 indicates how the complete distortion of the cross section may be determined by a mesh of measuring lengths. The displacement vectors u_i and v_i of a point are referred to an arbitrarily selected kinematical system A–B. Generally, three displacement vectors should be known or fixed. It is advantageous to introduce in a mesh some control lengths 'c' as indicated in Figure 13. In such a manner the reliability of the individual readings can be checked. Using a computer program for data processing the mesh can be adjusted by the method of least squares, thus increasing overall accuracy. In many instances such simple measurements are carried out merely to ascertain whether a state of stable equilibrium has already been reached, will be reached, or instabilities are to be expected.

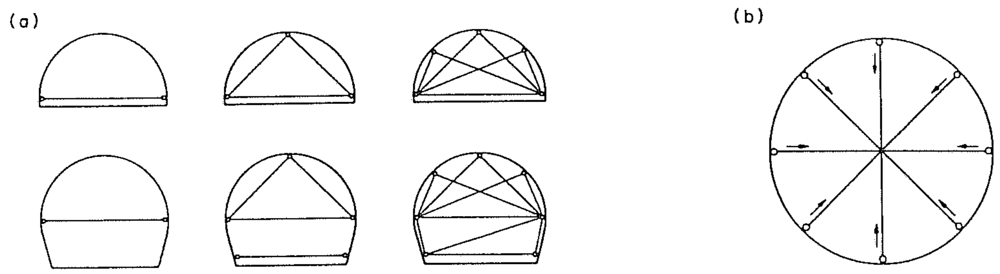


Figure 12 Convergence measurements with typical arrangements of the measuring lengths: (a) tunnel with different construction sequences and (b) control of diameter change in a gallery

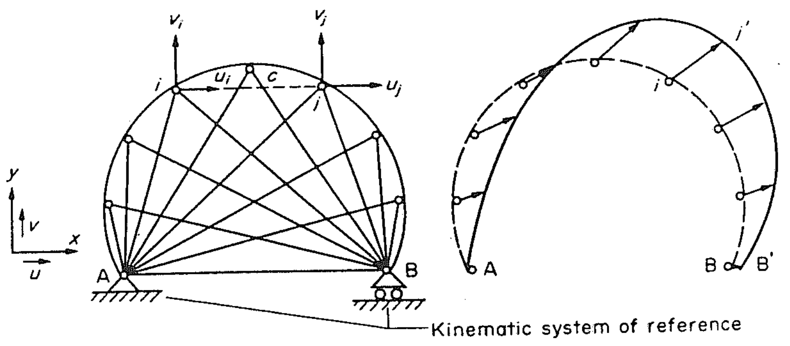


Figure 13 Determination of the complete deflection of a tunnel lining by a mesh of individual convergence measurements

20.5.1.1 Tunnels of the Imigrantes Highway

This three lane 55 km long highway connecting Sao Paulo with Santos in Brazil involved the construction of altogether 18 viaducts and 11 tunnels with the total length of 3825 m [6]. The tunnels – most of them slope tunnels – were constructed under difficult topographical and geomechanical conditions [7] and excavated simultaneously by different contractors. The tunnels present a considerable size of profile with a sectional area of 120 m^2. The general sequence of excavation is shown in Figure 14. The convergence measurements [8] with the distometer [12] were expected to give information about (i) the type of rock pressure which may develop in different sections of a particular tunnel (it was important to identify the type of possible rock pressure phenomenon); and (ii) the stability of the slopes affected by tunnel construction.

Different arrangements of the measuring lengths were used. The observations had to cover all stages of the construction, as shown in Figure 14. Special attention was paid to the behavior of the temporary lining in the calotte, which consisted of steel ribs with or without shotcrete. Figure 15 shows an example of readings in the case of a local instability, which occurred in tunnel TA-4 near to the face. By means of monitoring, this instability could be detected in its very early stage, thus permitting the installation of emergency supports formed of wooden timbers (Figure 16). The monitoring of the permanent lining in the calotte during core removal and side wall construction (Figure 17) was of great interest, too. In some cases, the effect of these constructional measures on the readings could be clearly observed (Figure 18).

When interpreting the readings of all 21 monitored measuring sections in seven tunnels the following points were kept in mind.

(i) The permanent lining forms together with the surrounding rock essentially a three-dimensional structure. This statement holds especially true when considering an asymmetrical excavation process and a step by step side wall construction. Simplification to a two-dimensional case is allowed whenever uniform conditions prevail in the vicinity of a measuring section.

(ii) The type of rock pressure phenomenon and the supporting effect of the lining may be estimated from the order of magnitude and the rate of the deformations.

(iii) Due to time limitations a useful back analysis could not be carried out.

Systematic field measurements in the Imigrantes Tunnels have shown quantitatively that no exceptional rock pressure phenomenon occurred in the tunnels. The rock was self-supporting as it had been supposed at the time of the first site inspection. Only loosening pressure has occurred. It was clear that the permanent lining of the tunnel was considerably overdesigned. Although the

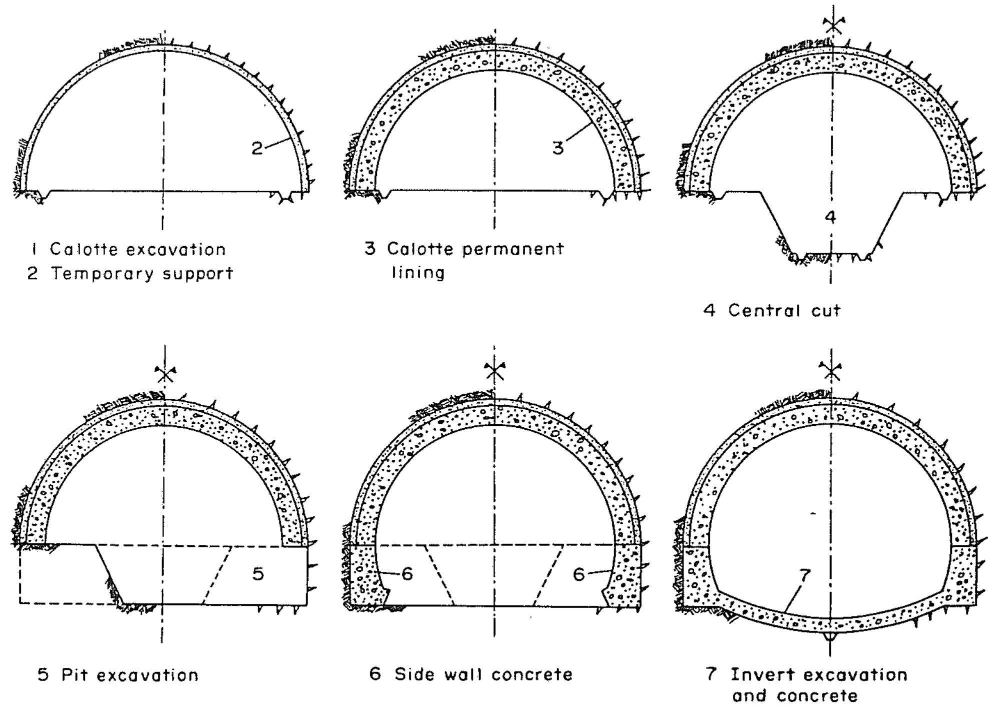


Figure 14 Imigrantes Highway – construction sequences of the tunnels [7]

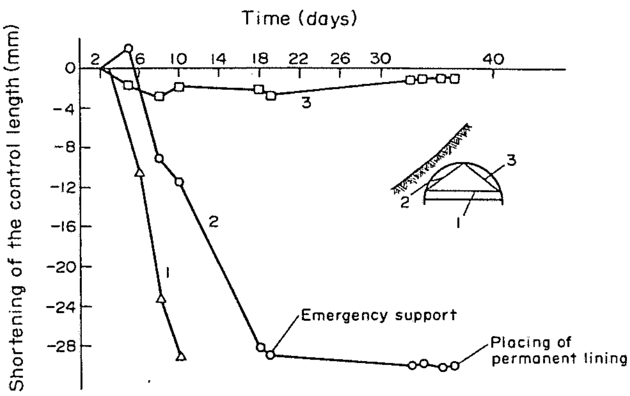


Figure 15 Indication of failure in the tunnel TA-4 by the distometer [12] reading, Imigrantes Highway [7]

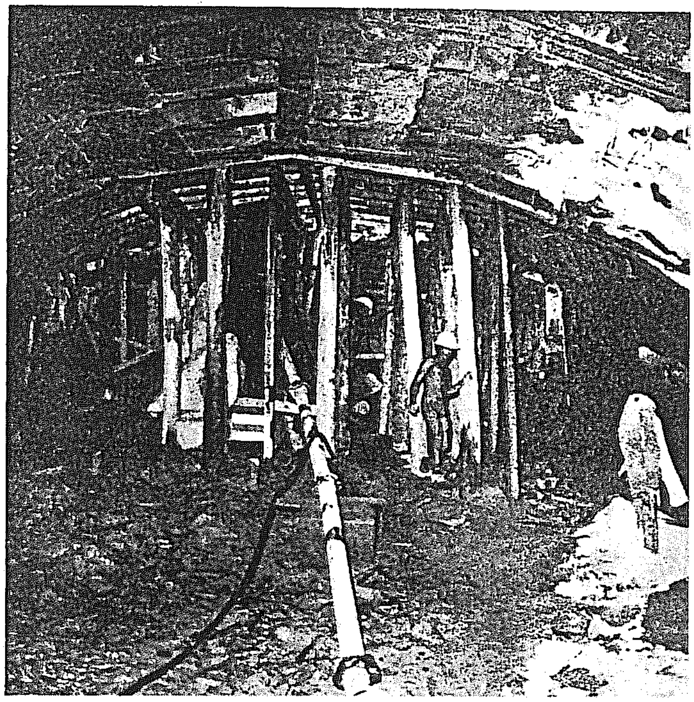


Figure 16 Emergency support in tunnel TA-4

measurements were initiated at a very advanced constructional stage, important savings in concrete and steel could still be achieved. The readings further demonstrated that the core removal could take place in one working unit and not in short trenches as it was originally assumed, thus speeding up the excavation and making it less expensive.

On the other hand, the problems related to slope stabilities in the portal zones deserved some attention. The large dimensions of the tunnel sections, the unfavorable alignment of the tunnels in steep slopes with slight overburden in the portal zones and some critical cuts for the 'service road' have raised the problem of slope stability. During the first site inspection it could easily be recognized that the problem of slope stability in the portal zones had in some cases been overestimated but in other cases almost completely ignored. By means of an adequate measuring program considerable savings in time, effort and money could have been achieved for this type of problem, too.

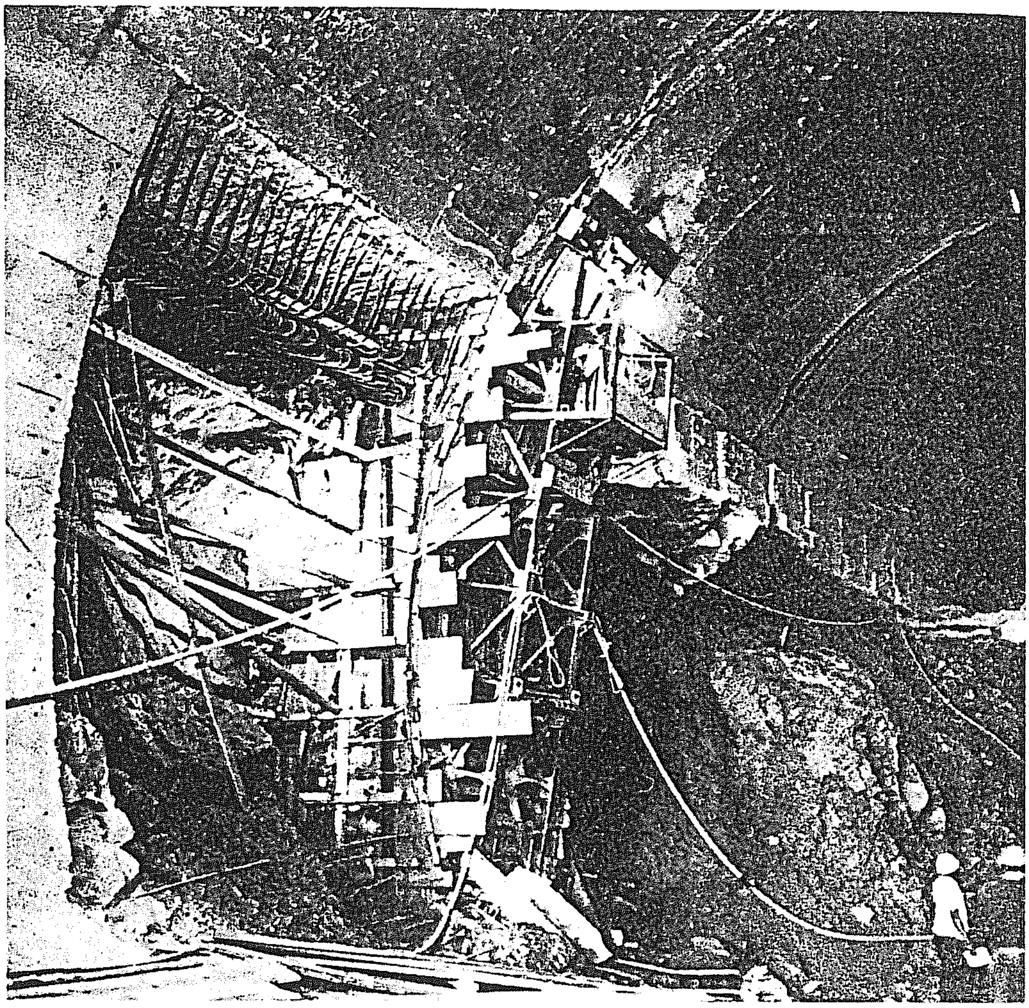


Figure 17 Pit excavation for wall construction, Imigrantes Highway [7]

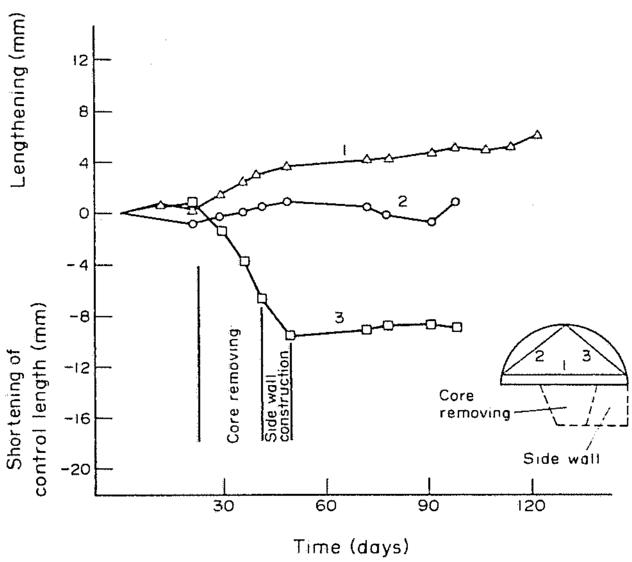


Figure 18 Influence of the core removal and the side wall construction on the control lengths. Tunnel TA-9, Imigrantes Highway [7]

20.5.1.2 Pressure tunnel with prestressed concrete lining

For the construction of the hydroelectric power scheme Grimsel–Oberaar in the Swiss Alps, the pressure tunnel in igneous rock was lined with prestressed concrete. This method of construction is especially suitable if the tunnel is subjected to a relatively low internal pressure, *i.e.* a value not much in excess of 1.5 MN m^{-2} [10]. The internal diameter of the tunnel is 6.8 m, the depth of the concrete lining 0.4 m and the internal pressure 0.75 MN m^{-2}. The cross section of the tunnel with the cable guides at position 1 of the stressing location is shown in Figure 19(a). The cables (system VSL) were laid alternatingly at intervals of 20 cm in the positions 1 to 4 (Figure 19b). The breaking capacity for each cable was 1547 kN. The function of the prestressing was to prevent cracking in the concrete due to internal pressure, in order to ensure that the pressure tunnel remains leakproof.

The interaction of the concrete with the rock was thoroughly investigated at the planning stage. Parametric studies with the finite element program RHEO-STAUB [11] were carried out in cooperation with the design engineers and the contractors. The aim of these computations was to throw some light on the question of whether the interaction of the concrete lining with the rock could hinder the desired build-up of compressive stresses in the concrete. The interaction was simulated as the embedment of an elastic ring in an elastic medium. The Young's modulus of the medium corresponded to the deformation modulus of the rock. The loading was given by the forces due to the prestressing of a single cable. In order to eliminate tensile stresses between the ring and the elastic medium the computations were carried out iteratively. As a result of these parametric studies using a simplified computation model, any apprehensions about the possible transmission of the prestress effect from the concrete to the rock could be dispelled. It was decided to check this result by *in situ* measurements as well and to investigate further the behavior of the prestressed ring for the alternating cable positions. In particular the deformation of the concrete ring and the separation of the ring from the surrounding rock had to be evaluated. By adjusting the computational model better to the actual conditions *in situ* it should also be possible to test the theoretical predictions against the real behavior of the structure.

Two measuring sections 40 m apart were fitted out with eight distometer measuring lengths (Figure 20). Based on preliminary computations the anchor position of the extensometer in a depth of 2.2 m could be regarded as a fixed point. The measuring lengths between the diametrically opposed measuring heads of the extensometer have solely a control function. Readings were taken before applying prestress and in steps of 25%, 50% and 100% of the maximum stress. The changes in diameter for the loading case of 100% prestress force are given in Table 1.

A comparison of the results of the convergence measurements using the distometer [12] with those of the single point extensometer shows that the anchor position of the extensometer is in fact a fixed point. The radial deformations of the concrete lining in two measuring sections for the last stage of loading (100% prestress) are shown in Figure 21. The variation of the deformations between the individual measuring points is unknown. The curves in this figure are based on arbitrary estimates and serve simply the purpose of giving a visual representation of a possible ring deformation. In reality, the concrete lining is a long cylindrical shell, which rests in places on the rock, and it is quite possible that in certain sections the lining is not in contact with the rock over its whole circumference. This could be the case in measuring section 2. The formation of a gap between

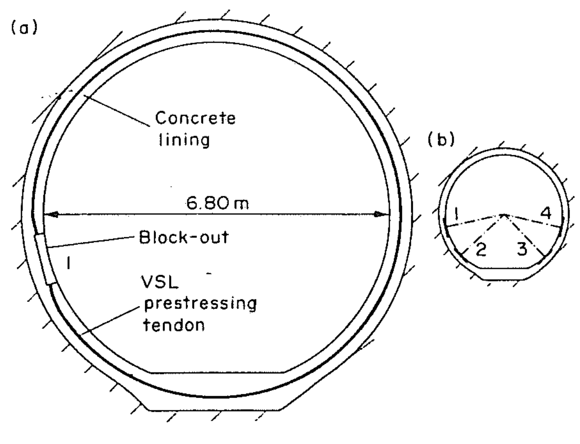


Figure 19 Prestressed concrete lining for a pressure tunnel: (a) layout of the tendon in position 1 and (b) positions of the tendon in subsequent sections

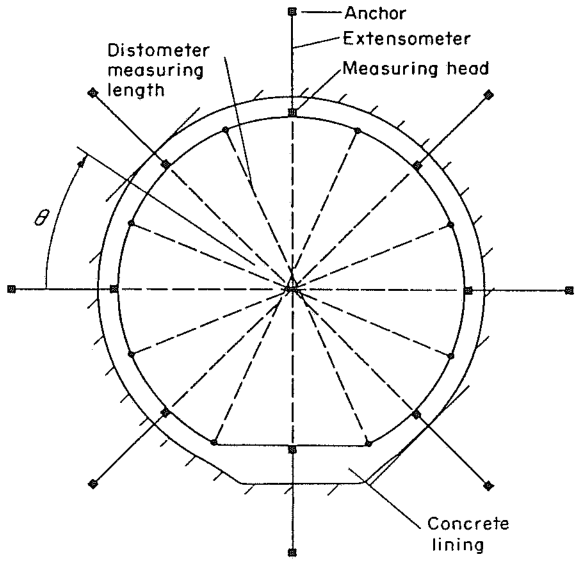


Figure 20 Layout of the extensometers and the convergence measuring lengths

Table 1 Change in Diameter, δ

	θ	δ_E (mm)	δ_D (mm)	$\left\lvert\frac{\delta_E - \delta_D}{\delta_D}\right\rvert$ (%)
Measuring section 1	0	5.85	5.26	11
	$\pi/4$	3.97	3.93	1
	$\pi/2$	9.95	9.83	1
	$3\pi/4$	4.04	4.57	12
Measuring section 2	0	3.65	3.50	4
	$\pi/4$	1.44	1.70	15
	$\pi/2$	12.90	12.74	1
	$3\pi/4$	3.41	3.50	3

δ_E, extensometer measurements: δ_D, distometer measurements.

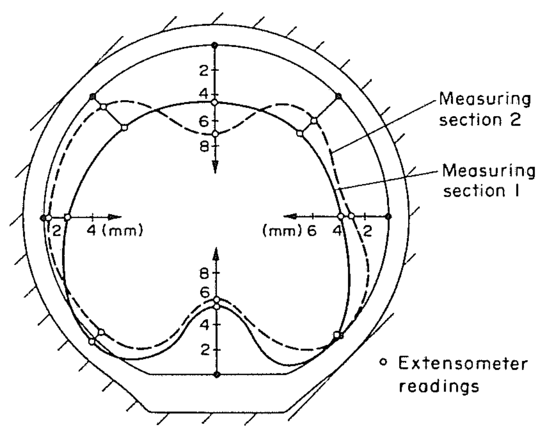


Figure 21 Results of the extensometer measurements for the loading case 100% prestress force

the lining and the rock is of practical significance in preventing a transfer of the prestress force to the rock. Having carried out the prestressing along the whole length of the tunnel and after the completion of the measurements this gap was filled by grouting. If one plots the observed changes in diameter as a function of the angle θ (Figure 22), reliable deformation curves can be obtained due to

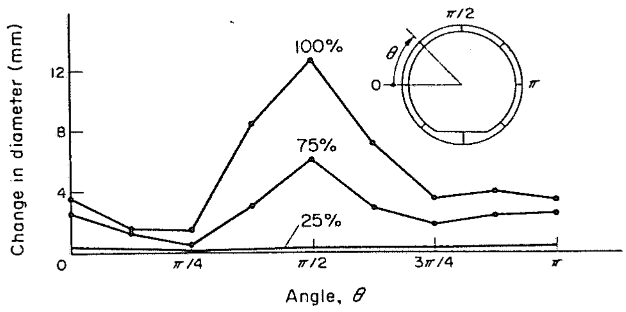


Figure 22 Distometer measurements at three different levels of prestress force in measuring section 2

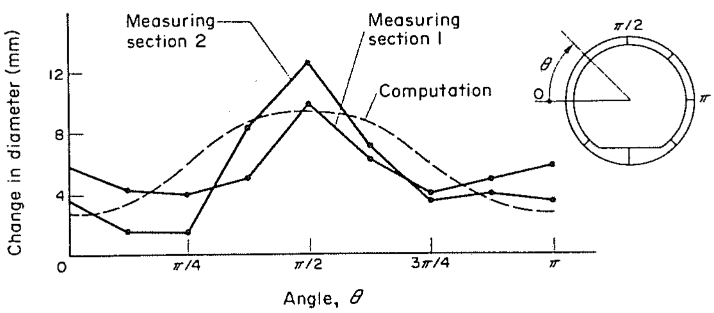


Figure 23 Results of computations and observations for the loading case of 100% prestress force

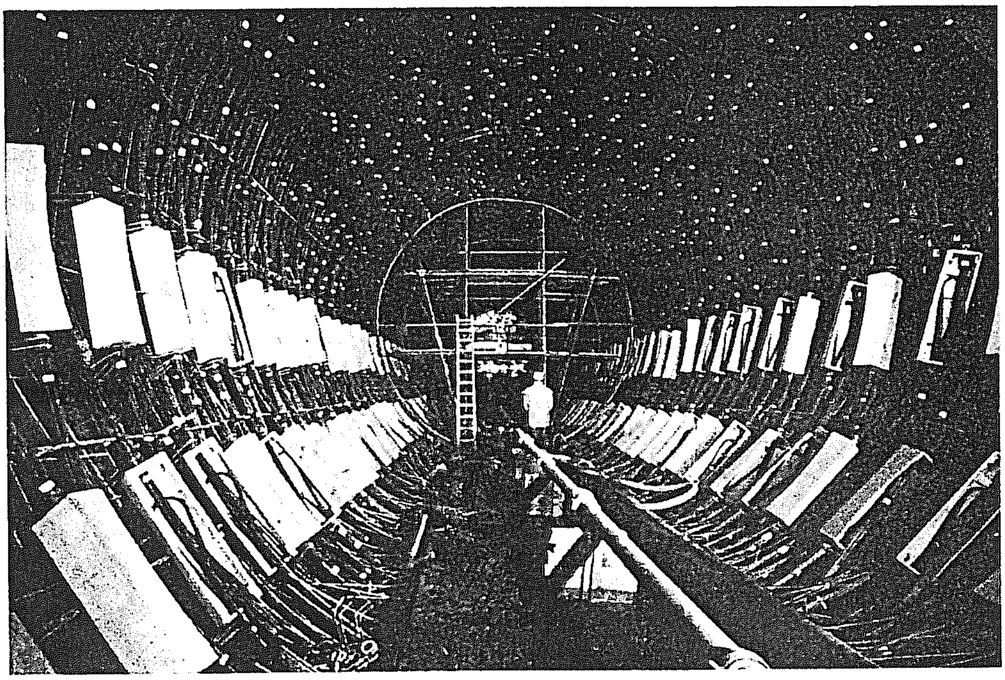


Figure 24 The prestressed concrete lining under construction

the large number of measurements taken. It was attempted, therefore, to select the assumptions for a computer model, such that the theoretical and experimental curves come together as close as possible. The distribution of the computed values shown in Figure 23 is based on the following assumptions.

(i) The support positions of the elastic ring are at $\theta_1 = -30°$ and $\theta_2 = 210°$.

(ii) The Young's moduli for concrete and rock are $E_C = 15\,000$ MN m^{-2} and $E_R = 2000$ MN m^{-2} respectively.

(iii) The influence of different cable positions is obtained by superimposing the effects of the individual cables.

The agreement achieved in this example between computation and observation is considered to be reasonable. One has to realize that the three-dimensional structure has been simplified to two dimensions for the sake of the computation. Also, the concrete behavior was assumed to be linear elastic, although (as evident from Figure 22) its behavior is in reality distinctly nonlinear, *e.g.* the deformations at 100% prestress are more than double those at 75% prestress. Figure 24 shows the prestressed concrete lining under construction.

20.5.2 Rock Pressure Determination by Measuring the Changes in Curvature and the Strain Along the Tunnel Lining

Rock pressure causes the lining of a tunnel to deform. An alternative method to determine the deflection of the lining is given by systematic measurements of both change in curvature and strains along the intrados of the tunnel lining in consecutive points. As an advantage, the internal forces (bending moment and normal force) and the rock pressure also can be calculated [9], provided that the material properties of the lining material are properly defined. A more detailed description and practical applications of this method can be found in Chapter 24 of this volume.

20.5.3 Decision Making for Tunneling in Swelling Rock Based on the Monitoring of Ground Displacements

Rocks containing clay minerals and anhydrite increase in volume when they come into contact with water; this phenomenon is referred to as the swelling of these rocks. In tunnel construction, swelling of rocks manifests itself as a heave of the tunnel floor or as pressure on the invert arch (Figure 25). When the lining remains intact, a heave of the entire opening can occur, where the crown as well as the floor experience an upward displacement. In some cases, the pressure resulting from the swelling of the surrounding rock leads to a failure of the invert arch.

In view of the good number of underground construction projects that are planned to be carried out in rock formations with a swelling potential, it is of significant economic necessity to develop viable methods of countering swelling pressures and swelling deformations. For a successful design of a tunnel in swelling rock, field measurements are an extremely important source of information. That is the reason to present the different elements and steps for the design of tunnels situated in heavy swelling rocks in more detail. For this problem, the systematic use of all sources of information such as field measurements, laboratory investigations, statical computations and last but not least the engineer's own experience is demanded.

20.5.3.1 The swelling process in the vicinity of a tunnel

Our presentation of the swelling mechanism is based on the observations of Terzaghi [21], which state that the swelling of the rock is brought about by changes in the state of stress resulting from the excavation. A relief of the stress component normal to the surface of the excavation takes place in the vicinity of the opening; at the surface, this component reduces to zero. Other factors must be

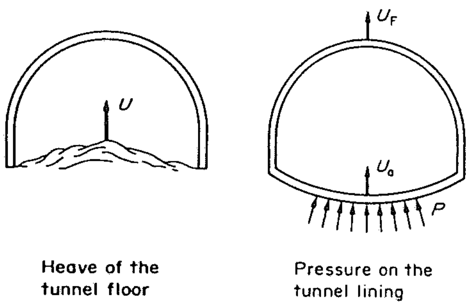


Figure 25 Effects of swelling in tunneling

relevant, however, since we observe the swelling only in the area of the tunnel floor. The principal aspects to consider have to do with the water conductivity of the rock, *i.e.* those factors which have an influence on the water seepage. Computational methods which take only the stress relief but not the movement of water into consideration would predict swelling not only in the floor but also in the walls and in the roof of the tunnel; this is clearly in conflict with observations made in the field. The water that flows towards the excavated area tends to move towards the floor of the opening (effect of gravity), leaving the upper rock zones untouched. Thus, the swelling which would be expected in these zones as a result of the stress relief does not take place. Other authors [27, 28] have proposed formulations that take into account the local three-dimensional state of stress. The results of such computations also depend highly on the initial state of stress and in particular on the relation between the horizontal and vertical normal stress components. Neither the initial stresses nor the stress redistribution resulting from the excavation, however, can be measured in swelling rock formations at the present state of measuring technology. In a recent research work, Anagnostou [38] proposed a new numerical model that takes account of the movement of water inside the rock mass and the interaction of the pore water with the rock matrix. It allows the simulation of time-dependent effects. The swelling rock is modeled by elasto-plastic constitutive equations.

The effects of the swelling of the surrounding rock on the structure are in most cases monitored in the field by means of leveling and convergence measurements. Borehole extensometers are also often called for to obtain a picture of the deformations in the rock surrounding the tunnel.

It has been possible for some time now to follow the *in situ* swelling of rock with great accuracy and detail by using a sliding micrometer (Figure 26). This instrument allows us to measure the strain profile along the entire length of boreholes. Along a vertical borehole, ring-shaped measuring marks connected by hard PVC casings are fixed at intervals of 1 m by means of grout (Figure 26). To take a reading beginning at the mouth of the borehole, the probe is lowered in sliding position (Figure 27a) and fixed in the two measuring marks (Figure 27b). There, a reading of the axial LDT gauge is taken and sent to a hand-held computer. The probe is then traversed in a stepwise manner down along the borehole taking readings at every step in a similar manner. After reaching the bottom of the measuring line, a repetition of all readings is made on the return way of the probe. This allows an immediate check of accuracy by the computer in the field. Spurious readings can be detected while still at the site and the readings can be repeated if necessary. The field accuracy of this portable type of probe is better than $\pm 5 \times 10^{-6}$. Since the standard length over which each strain measurement is made is 1.0 m, strain is usually expressed in mm m^{-1}. The change in relative distance between two consecutive measuring points can thus be determined with an accuracy of ± 5 µm. A detailed description of this instrument is given elsewhere [14, 15]. Several examples of strain measurements in road tunnels through formations of swelling rock are presented in the following.

The Pfaender Tunnel near Bregenz in Austria [18] traverses interbedded layers of marl and sandstone over a distance of about 4.4 km. The tunnel floor, which was supported only by a cover of

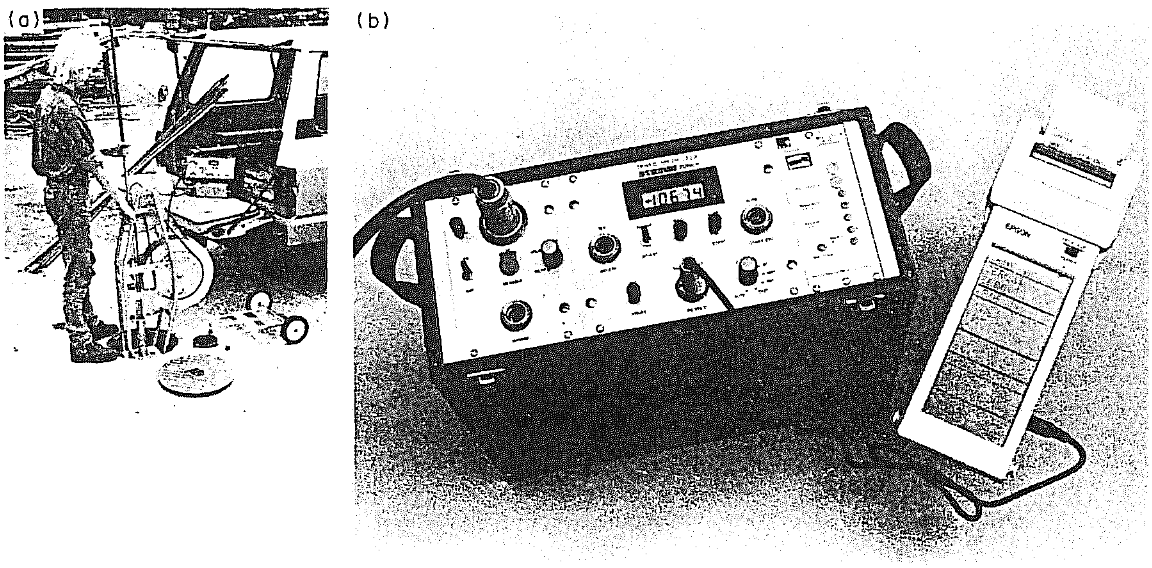


Figure 26 (a) Operation of the sliding micrometer probe in the borehole using a cable winch. (b) Readout unit with data terminal

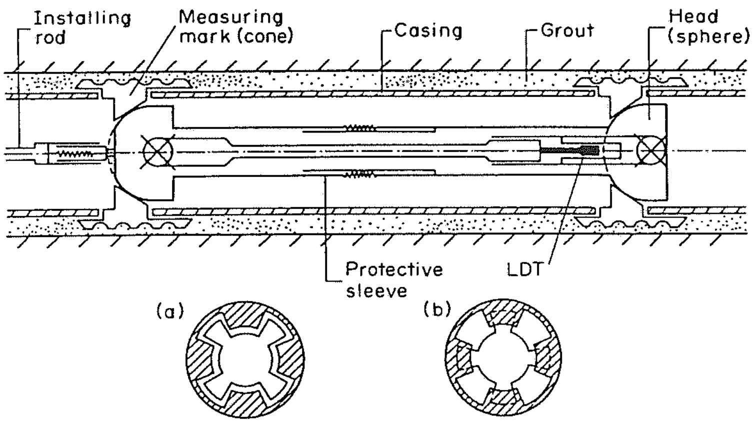


Figure 27 The sliding micrometer borehole probe. Schematic view of the instrument in (a) sliding position and (b) measuring position in the borehole

shotcrete, started heaving during construction. An anchoring system, consisting of corrosion resistant, pretensioned anchors with a design strength of 1000 kN, was called for. The length of these anchors is 10 m with a fixed anchor length of 5 m. Along with the anchoring work, two boreholes, each 18 m in length, were drilled at two locations in the tunnel, and equipped with tubing for strain measurements using a sliding micrometer. Readings have been taken regularly since 1980 and are presented in Figure 28. A layer about 2 m thick, which is situated directly under the tunnel floor, shows compressive strains. This layer is located between the heads of the pretensioned anchors and a larger, underlying mass of swelling material. Strains resulting from swelling of the rock decrease with depth, and reach zero about 12 m under the tunnel floor. The deformations, which are recorded as a function of time, are tending to level off. A total heave of only 1 mm was observed over the nine year measuring period, which demonstrates the effectiveness of the anchoring. Figure 29 shows the results for the second borehole located about 1.9 km from the first. In this case, a 2 m thick swelling layer located directly under the tunnel floor was identified. This layer caused a heave of the floor of 5 mm.

We now turn to the observations made in the T8 (Sonceboz–Biel) tunnel in Switzerland (Figure 30), which traverses layers of marl of high strength. The strain measurements in the borehole could be made only up to a level of 1.5 m under the floor. Displacements here are also remarkably small, although noticeable heaving was recorded right after excavation. A decrease of strain with increasing distance from the excavation is also noted here.

A further example of an observed swelling process in the vicinity of an excavation is the Belchen Tunnel of the N2 highway (Switzerland). Long stretches of the tunnel run though formations of

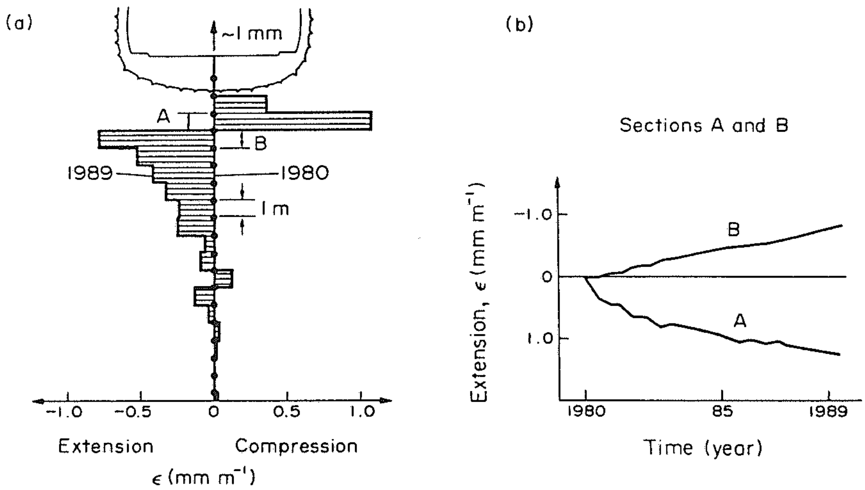


Figure 28 Pfaender Tunnel: (a) strain profile ε in the rock and (b) time dependence of strains in layers A and B in measuring section 1

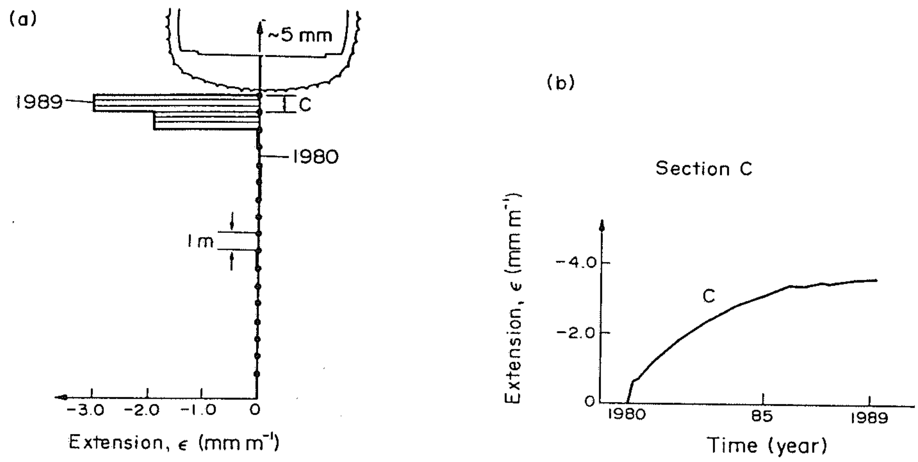


Figure 29 Pfaender Tunnel: (a) strain profile ε in the rock and (b) time dependence of strains in layer C in measuring section 2

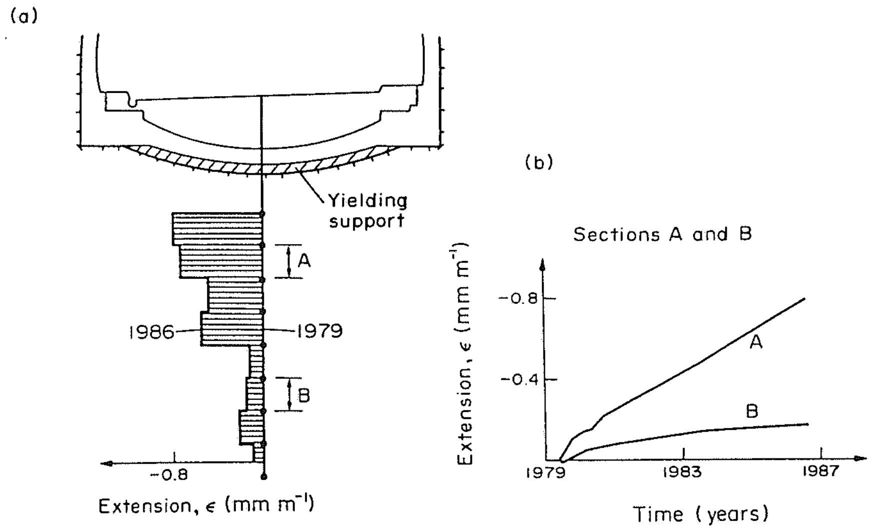


Figure 30 Tunnel T8: (a) strain profile ε in the rock and (b) time dependence of strains in layers A and B

Opalinus clay and rock containing anhydrite (Jurassic formations, ref. 'Gipskeuper'), both of which are well known for their particularly pronounced swelling characteristics. Detailed reports have been made of the damages which occurred during construction [20]. An extensive study including sliding micrometer measurements was ordered in 1986 to check the condition of the tunnel 18 years after its completion. Ongoing swelling was still noted in four measuring sections, an example of which is given in Figure 31. The strains due to swelling over a four year period can be seen in this figure to reach values of approximately 3.5 mm m^{-1}; this corresponds to an extension of about 1% per year. The increase in swelling pressure on the invert arch that goes along with these increments in strain and the cause of the gaps in the strain profile are not known. For the latter, it is assumed that the material in those zones did not swell, or that local swelling has reached a maximum value under the present stress state. It is also conceivable that groundwater had not yet found access to this area.

Summing up our present insights on the effects of swelling in tunnel construction gives the following.

(i) Swelling processes take place – to an extent which is relevant to practice – only in the floor area of excavations.

(ii) The swelling strain decreases with increasing distance from the excavation, reaching zero at a distance of about one diameter from the bottom of the opening.

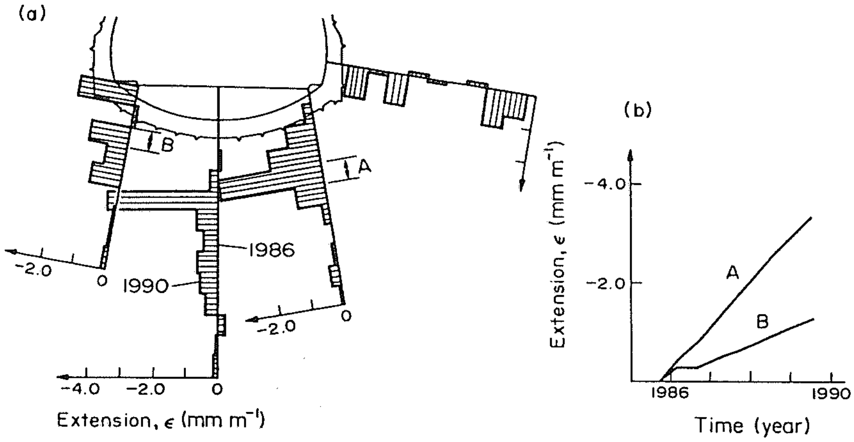


Figure 31 Belchen Tunnel: strain profiles in the Gipskeuper for two subsequent measuring periods of six months each (18 years after the completion of the tunnel)

(iii) Swelling processes can take place more or less rapidly, depending on the facility of access the water has to the floor area and on the water conductivity of the rock. In some cases, swelling processes continue over several decades after the completion of an underground opening.

20.5.3.2 Characteristic line for swelling rock

The characteristic line procedure was proposed by Lombardi for the quantitative description of the interaction between the invert arch and swelling rocks [25, 26]. 'Characteristic line' here refers to the relation between the heave of the tunnel floor U_a and the lining resistance P_a (Figure 37). The lining resistance is the pressure the lining exerts on the surrounding rock. The reaction to it is the swelling pressure, *i.e.* the pressure exerted by the rock on the lining, *e.g.* on the invert arch.

We have proposed a simple procedure for the determination of the characteristic line that does not require consideration of initial stresses [32]. Initial stresses as well as stresses resulting from the excavation can be neglected by presupposing the knowledge of the strain profile $\varepsilon(z)$ of the swelling rock beneath the invert arch (Figure 32). In homogeneous rock, the maximal swelling strain value ε_a is shown at the excavation surface with the strain decreasing to zero at about one tunnel diameter D from the opening. Along with this fundamental assumption on the strain $\varepsilon(z)$, we use a second assumption, as other authors do [20, 27, 28], namely the 'swelling rule' for the rock [32]. Not only individual laboratory tests but the entire stratigraphy below the tunnel floor must be considered in the determination of the parameters a and b (Figure 32). If only few oedometer test results on samples of small size are available, the sandwich structure of the rock must be taken into account for the determination of the parameters a and b. Representative swelling parameters a and b can be estimated on the scale of the rock formation, if the stiffness and swelling behavior of the individual layers are known.

Given the assumptions in Figure 32, the floor heave U_a for rock that is homogeneous with respect to swelling can be obtained from the integration of $\varepsilon(z)$. One thus obtains the general form

$$U_a = kD\varepsilon_a$$

where k is a shape factor for the $\varepsilon(z)$ strain curve. Three examples of possible strain distributions are represented in Figure 33. Using the so-called log–normal 'swelling rule' (Figure 32), already mentioned by Terzaghi [37], we find the characteristic line of a swelling rock formation in the general form

$$U_a = A - B\log P_a$$

The characteristic line thus derived exhibits basically the same behavior as the 'swelling rule' curve for the oedometer tests. The unrestrained floor heave U_{a0} corresponds to the unconstrained strain due to swelling. The maximal value of the lining resistance corresponds to the swelling pressure. Theoretically, the characteristic line represents equilibrium conditions if no more measurable displacements can be observed. Two interesting, practical conclusions can be drawn from the

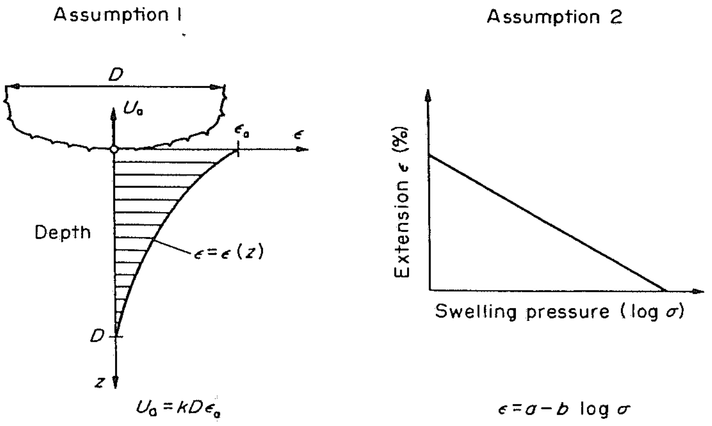


Figure 32 The two fundamental assumptions for the determination of the characteristic line of the rock: the strain profile in the tunnel floor and the swelling rule for the rock

logarithmic relation shown in Figure 34. On the one hand, a small allowed floor heave ($+\Delta U_a$) results in a relatively large decrease in swelling pressure ($-\Delta P_a$); also, conversely, a relatively small lining resistance ($+\Delta P_a$) causes a significant decrease of the floor heave ($-\Delta U_a$). Tests and measurements in the field must be made to confirm whether rock with a swelling potential in the floor area of an excavation behaves in accordance with the characteristic line in Figure 34. Individual observations which, according to this figure, confirm the effect of a relatively small lining resistance on the heave due to swelling are already available [32].

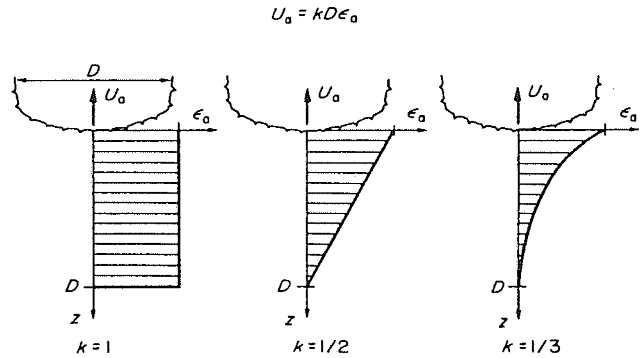


Figure 33 The shape factor k and the floor heave U_a for different strain profiles

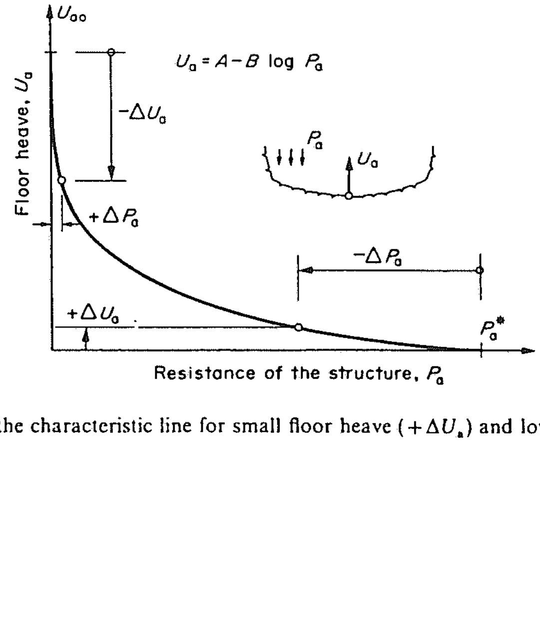


Figure 34 Attributes of the characteristic line for small floor heave ($+\Delta U_a$) and low lining resistance ($+\Delta P_a$)

20.5.3.3 Constructive countermeasures

The design solutions shown in Figure 35 can be used to counter the undesired effects of the swelling rock [22]. Using a resistant invert arch or an anchoring system aims at restricting heaving of the tunnel floor. In this case, significant swelling pressures can result. We know of only one example of a large scale anchoring system, namely the Pfaender Tunnel near Bregenz, which was mentioned earlier. The use of an open space under the roadway plate, on the other hand, is aimed at avoiding the build-up of swelling pressures [26]. The solution using an invert arch on yielding supports is a compromise between both extremes. Here, both swelling and the development of some swelling pressure are allowed in a controlled way.

The constructive design, *i.e.* the dimensioning of the individual structural support elements, is made based on two criteria [30]. On the one hand, enough bearing capacity must be available to prevent failure under swelling pressures. On the other hand, deformations (especially heaving of the floor) must be kept within acceptable limits (Figure 36). In some cases, like in tunnels for high speed trains, where strong limits are imposed on warping and alignment of the rails, the second criterion may be decisive. In this case, the heaving by sections of the entire tunnel lining or its twisting can lead to more serious problems than higher swelling pressures.

The particularities of the various design measures and the interaction of the lining with the surrounding rock can be illustrated with the characteristic line. Figure 37 shows the log–normal characteristic line derived above. Let us start by examining the case of an invert arch. The heave of the tunnel floor is strongly constrained by the great stiffness of the arch. As a result, a swelling pressure is imposed on the arch; this pressure may approach the extreme value P_a^* (Figure 37). This is particularly the case if the invert arch is placed under dry conditions before any swelling takes place. The other extreme is the open tunnel floor. The implementation of a yielding support system between the invert arch and the rock results in values of $\bar{P}_a$ and $\bar{U}_a$ which depend on the deformation properties of the construction elements (Figure 37).

The solution to be implemented for a given project should be in accordance with the swelling parameters of the rock and the design criteria (Figure 36). An invert arch seems to be the simplest and most economical solution for smaller extreme values of P_a^*, which can be taken up by an arch with a thickness of 0.4–0.5 m and a moderate reinforcement of the concrete. The open tunnel floor should be considered only where the maximal heave of the floor remains small. In this case, the lining lacks the strength of the statically advantageous closed ring form. There is also a risk of the side walls caving in if the heave of the floor is too large. The construction method using an invert

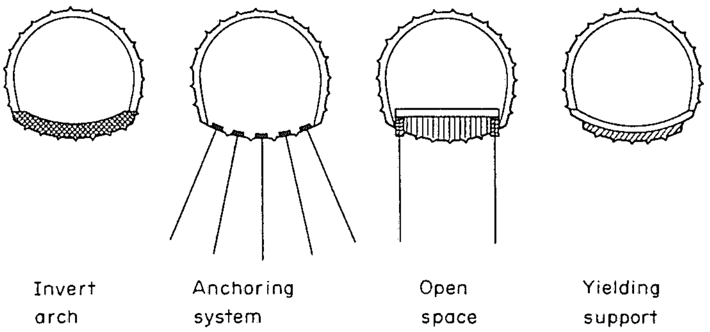


Figure 35 Design measures used in swelling rocks

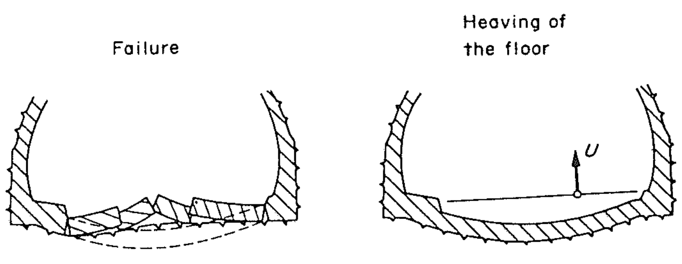


Figure 36 Design criteria in swelling rocks

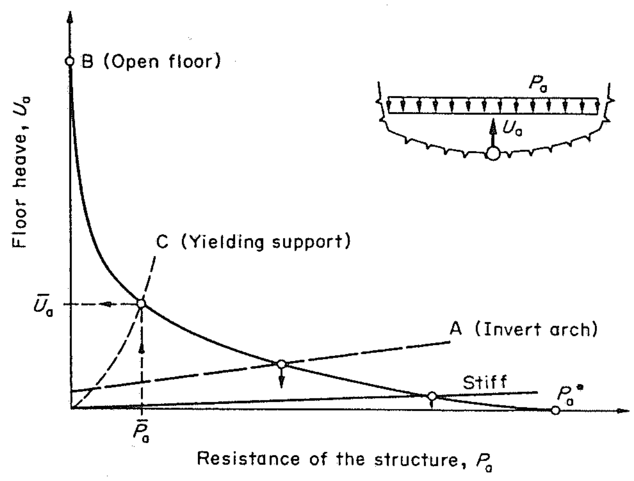


Figure 37 Log–normal characteristic lines for different design measures

arch on a yielding support should in general be the optimal solution for higher values of P_a^*. We therefore examine this construction method in greater depth here.

20.5.3.4 Tunnel design with yielding support

The fact that a reduction of swelling pressure on the invert arch results in increasing deformations of the rock is already obvious from Terzaghi's observations [21]. The concept of a yielding support using for example 'a compressible layer between the lining and the rock' was proposed in 1972 [20]; the first constructive implementation of the concept, however, was made by Lombardi [25].

In 1978, Kovari and Amstad realized a yielding support for the T8 tunnel (Switzerland), using low strength support pads of lightweight concrete having heights up to 300 mm [19]. These pads with square cross sections were installed at intervals of 2.0 m (Figure 38), and the foam plates between

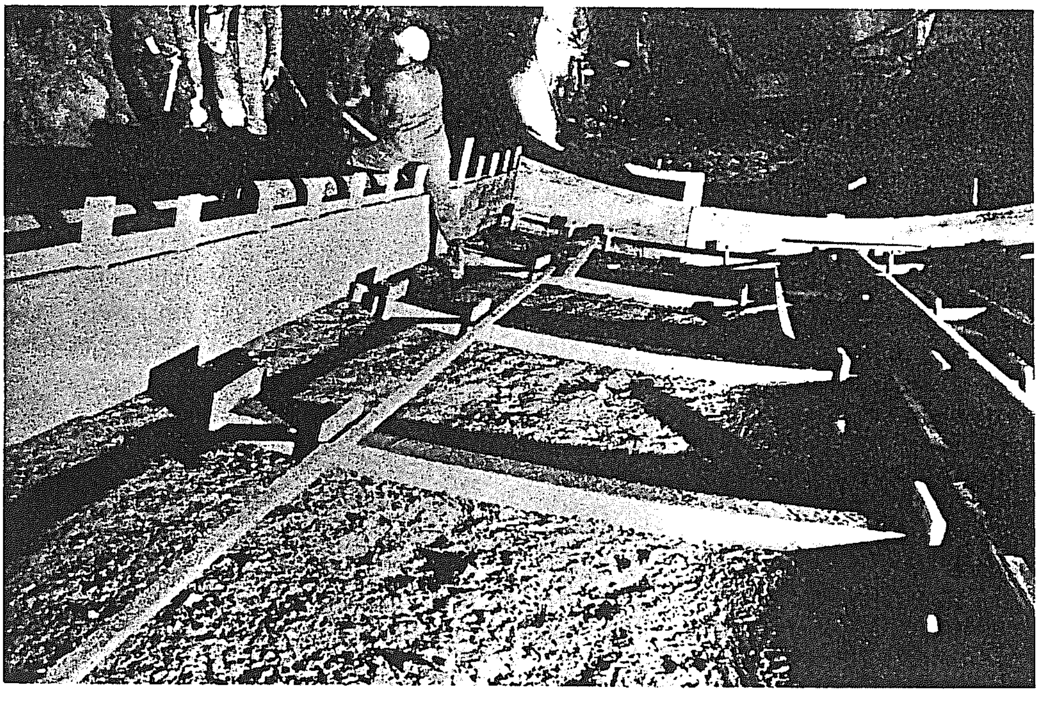


Figure 38 Construction of low strength support pads in tunnel T8

them served mainly as rock-side formwork for pouring of the concrete. The pads have two principal functions. Firstly, they serve as foundation for the invert arch, even if there is no swelling of the underlying rock. The additive (Leca) used in the lightweight concrete is resistant against chemical decay, so that rotting of the pads does not need to be considered. The second role of the pads is to allow for heaving of the rock under the tunnel floor to a certain, predetermined extent, while keeping the swelling load on the invert arch below a given design value. The design of the pads was based on an estimate of the floor heave that would take place without any constraint. In Figure 39 a comparison of the measured swelling heaves in the various measuring sections is shown. In this semilogarithmic representation the time variation of the movements during the period of observation can be approximated very well by straight lines. The extrapolation, therefore, to a further 100 years seems to be reasonable, provided that the conditions of the construction remain the same. The authors are aware of the great uncertainties involved in such an extrapolation. However, it is felt that the displacements obtained in this way are on the safe side. From these field measurements and considerations it is concluded that the heave of the flat bottom of the tunnel without the restraint of the invert arch could reach up to 20 cm depending on the height of overburden and mineralogical composition.

A second example of the implementation of a yielding support between the invert arch and the underlying rock is the Freudenstein Tunnel of the German Federal Railways. This 6.8 km long railway tunnel is part of the new high speed section between Mannheim and Stuttgart [30, 31]. The tunnel traverses the strongly leached Gipskeuper formation in the eastern section on a length of 2.3 km. This formation consists of alternating layers of water-bearing and variably weakened rock. The following section passes through unfractured, unleached strata containing layers of anhydrite and clay with high swelling potentials. The leached and the unleached strata are separated by the so-called 'gypsum level'. Leaching takes place in this interface at a relatively slow rate. The tunnel intersects the anhydrite and gypsum level over fairly large stretches. The angle between the tunnel axis and the gypsum level is relatively small, and, as a result, the tunnel floor is often situated in zones of rock with a high swelling potential, while the crown is located in leached, almost loose rock. The phreatic surface in the formation is about 60 m over the tunnel. In the following discussion, only the 4.1 km long, western section of the tunnel in the unleached Gipskeuper formation will be considered. One particularity of this tunnel is evident from the description given above: the swelling potential of the surrounding rock would present no problem if water movement from the leached to the unleached areas around the tunnel could be safely prevented over the planned, 100 year lifetime of the project. Even with sophisticated sealing measures, however, the water cannot be fully prevented from seeping along the tunnel, from the wet to the dry areas. The designer decided to call for the statically advantageous circular cross section for those parts of the tunnel lying in swelling rock. The laboratory tests conducted during the design phase of the project showed that the first estimates of the maximal swelling pressure had been too optimistic. As a result, the thickness of the lining and the amount of reinforcement had to be continuously readjusted to the increasing values of

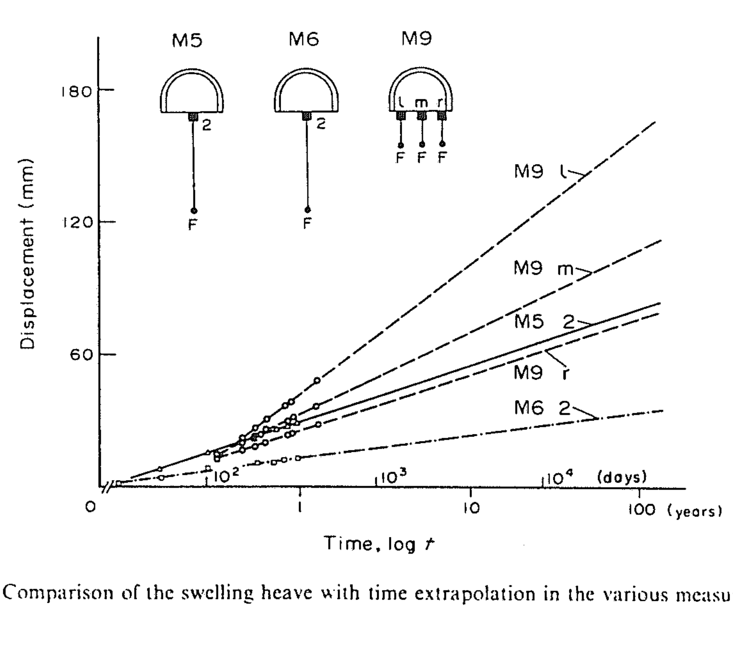


Figure 39 Comparison of the swelling heave with time extrapolation in the various measuring sections

swelling pressure. The result was a design with a thickness of the lining of 1.70 m in the floor and 1.00 m in the crown (Figure 40). In view of laboratory tests giving values of 30% for the maximal swelling strain, and over 6 N mm^{-2} for the swelling pressures, even the strongest circular section could not be accepted as being sufficiently resistant. A greater worry yet was the expected heave of the entire tunnel lining, which would make the usefulness of the structure itself questionable, even if the lining could resist the swelling pressure. The proposal worked out [32] with a yielding zone between the tunnel floor and the underlying rock presented a viable alternative. Not only could the maximal expected swelling pressure (and consequently the values of uplift) in the tunnel be significantly reduced, but a flattened tunnel floor design could also be used instead of the circular cross section (the former presenting great technical advantages in execution over the latter). The new project was then fully worked out and opened to call for tenders. The profile and the dimensions of the structure are based on the following assumptions.

(i) The rock is homogeneous in the floor area and behaves according to the 'swelling rule' in Figure 32. Its swelling parameters are: $\varepsilon = 20\%$ (swelling strain) and $\sigma^* = 6$ N mm^{-2} (swelling pressure).

(ii) The shape factor k, based on the measured strain distribution in the rock (Figure 33), is taken to be 1/3 (parabolic distribution).

(iii) The yielding support zone should on the one hand provide a sufficient capacity for the pouring of the concrete for the invert arch and on the other hand have a compressibility of up to 30% under a swelling pressure of 0.5 N mm^{-2}.

(iv) B 35 concrete and BSt 500 reinforcement were used for the tunnel lining.

The calculations done according to these assumptions lead to the profile in Figure 40. Notable differences in the dimensions (*e.g.* curvature and thickness of the concrete lining) can be seen upon comparison with the original circular profile (without yielding supports). It was intended to install the support pads (Figure 44) of lightweight concrete to provide an extra support for the lining that would compress as well in cases of greater swelling and in order to limit the forces transferred to the vault. Finally, the support pads had to be made of B 25 concrete because of the doubts concerning the longtime stability of the pads surrounded by the extreme aggressivity of the groundwater.

Upon request of the owner, a test gallery was set up near the west entrance of the tunnel for further *in situ* tests; these tests are expected to answer some of the questions still remaining on the swelling of sulfatic rocks. In four sectors of the test gallery, each floor plate (Figure 41) was pretensioned with a constant load P of 0.1, 0.25, 0.5 and 0.75 MN m^{-2}. The floor heave U_a, three years after watering the floor area of the gallery, is presented in Figure 42. These values are not the equilibrium results, because the swelling process still continues nearly linear with time. The strain distribution of the swelling rock under the pretensioned plate ($P = 0.1$ MN m^{-2}) is shown in Figure 43. Three years after watering the floor area, the swelling process concentrates in the rock layer near the floor plate. These first impressions of the *in situ* behavior of the swelling rock confirm the laboratory results. They show the extreme swelling capacity of the Gipskeuper rock formation in the Freudenstein Tunnel.

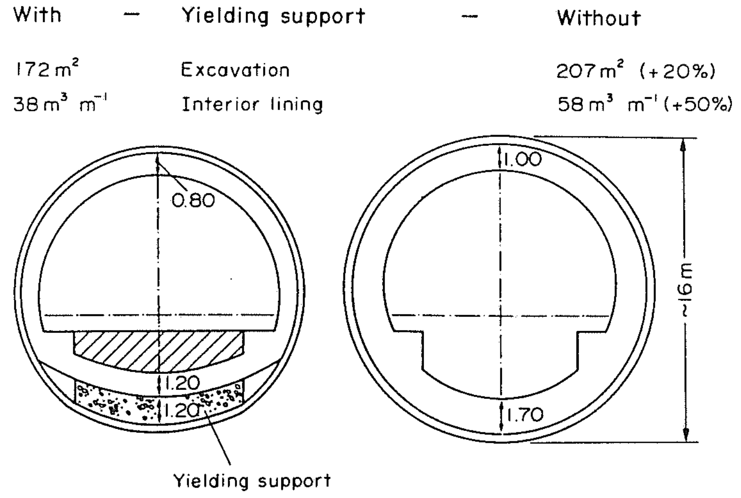


Figure 40 Freudenstein Tunnel: tunnel sections with and without yielding support in the Gipskeuper rock formation with high swelling potential (swelling strain > 20%, swelling pressure > 6 N mm^{-2})

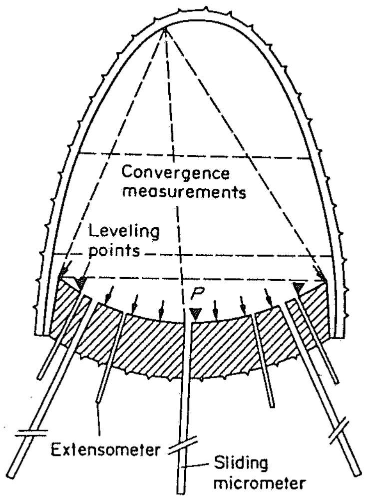


Figure 41 Example of a cross section in the test gallery with an anchored floor plate and the layout for field observations (Freudenstein Tunnel)

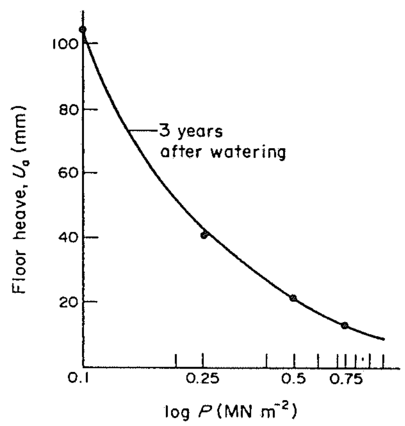


Figure 42 Measured log-normal characteristic line for the Gipskeuper rock formation, three years after watering (results from the test gallery in the Freudenstein Tunnel)

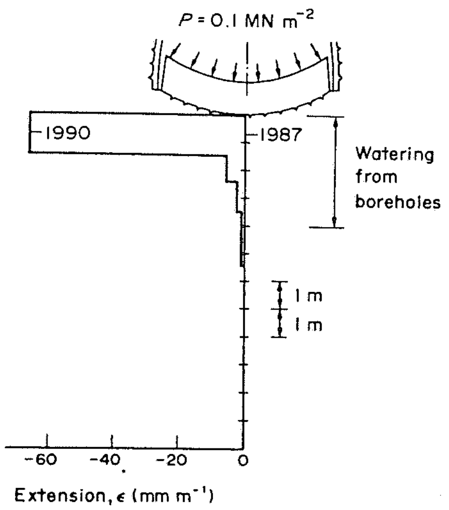


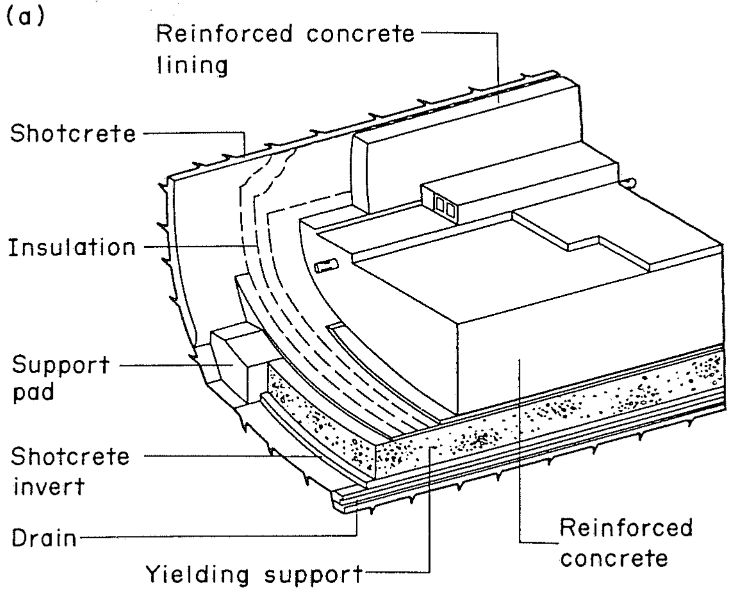
Figure 43 Distribution of the swelling strain in the Gipskeuper rock formation below the floor plate. Pretension of the invert plate with a constant load of 0.1 MN m^{-2}, three years after watering (Freudenstein Tunnel)

The following conclusions can be drawn concerning the design of tunnels in swelling rock, based on field measurements.

(i) The swelling process in rock formations can be monitored exactly by means of continuous strain measurements along selected boreholes using the sliding micrometer. These measurements give *in situ* information on the swelling potential of individual layers, on the decrease of the swelling strain with increasing distance from the opening and on the time-dependent development of the swelling.

(ii) A characteristic line for the 'homogeneous' swelling rock can be determined, based on two simple and verifiable assumptions. Firstly, the calculation is based on a 'swelling rule' that is represented for rocks containing anhydrite as well as clay rocks by a straight line in a log–normal graph. The second assumption concerns the distribution of swelling strains in the floor area of a cavity. The validity of the simplifications in the determination of the characteristic lines can be checked through direct field observation. The initial state of stress in the surrounding rock and the

(a)



(b)

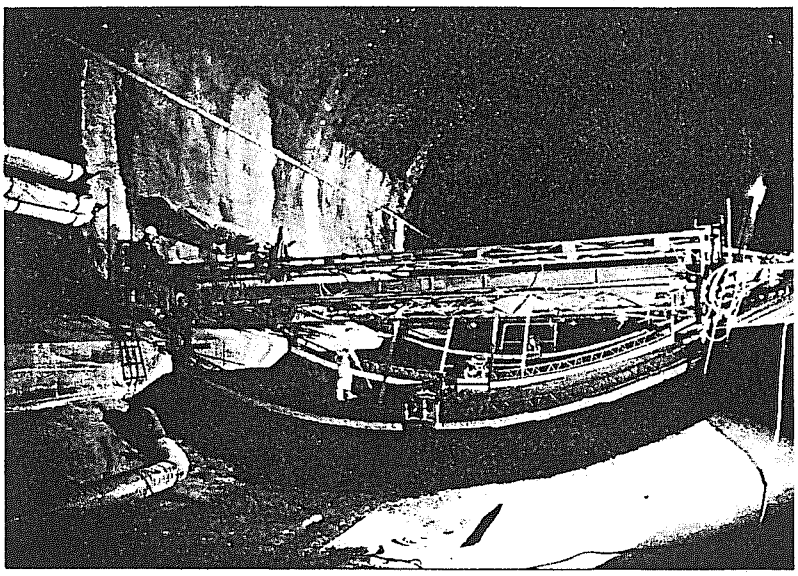


Figure 44 Construction of the yielding support in the Freudenstein Maintunnel

one resulting from excavation thus do not need to be considered, because their effects are included in the assumed strain distribution.

(iii) The implementation of yielding supports seems to be the safest and the most economical solution when the swelling potential of the rock through which the tunnel is built exceeds a certain limiting value. For projects in which the extent of floor heave is limited by the use of the opening (*e.g.* in the construction of tunnels for traffic) this yielding support zone is placed between the invert arch and the underlying rock surface. In rock formations with high swelling potentials (*e.g.* Gipskeuper) the yielding support system (Figure 44) represents a constructive solution that ensures a long-term use of the facility, in that it allows a certain amount of controlled heave while providing structural resistance.

20.5.4 Decision Making in Subway Tunneling

Two examples from the subway in Munich are presented here to show the importance and usefulness of field measurements for decision making in subway tunneling. The planned network for the Munich subway system has a total length of about 100 km with 106 stations. Since 1974 the tunneling with shotcrete has been of growing importance, resulting in a stretch of 21 km using this method [33]. Economy and safety are being given great attention and therefore field measurements always accompany the construction procedure. Figure 45 represents a typical geological section showing the two major formations, *i.e.* the quaternary deposit consisting of gravel, sand and the tertiary marl, frequently referred to as 'Flinzmergel', below it.

The latter has a varying appearance consisting of stiff or even hard clays, clayey silts, marl, marlstones and fine to medium grained sand. The groundwater in the quaternary formation is as a rule not connected with the water in the tertiary ones. There the pore water pressure can also be very different in adjacent sand lenses sometimes showing an artesian character. The clays and marls are nearly impermeable, offering a reliable protection against the water in the quaternary formation providing the thickness of the marl layer above the tunnel roof is not less than 2 to 3 m. In the cases discussed below this condition was always fulfilled.

The method of excavation for a single track tunnel is the head and bench method (Figure 46a). Emphasis is placed on shotcreting the invert very close to the head (2 to 4 m) and in a short time span of 1 to 2 days only. In this way a statically favorable action against ground deformations and surface settlement is produced immediately. The same principle is applied to the double track cross section (Figures 46b and 46c). Here, the first half of the tunnel is excavated and supported as a single track tunnel. The enlargement to the full cross section follows in a distance of approximately 15 m and again in head and bench operation. If water-bearing sand layers are encountered special measures must be taken. They may involve decreasing of the piezometric head by drainage wells and also application of compressed air as an additional measure. The tertiary sands are generally rather compact so that they are stable at the face provided that no excessive water pressure prevails.

If compressed air is applied, the whole section is constructed using shotcrete as temporary support. After the completion of the section, atmospheric conditions are restored. The shotcrete lining resists the outside water pressure until the final reinforced concrete lining is constructed. This

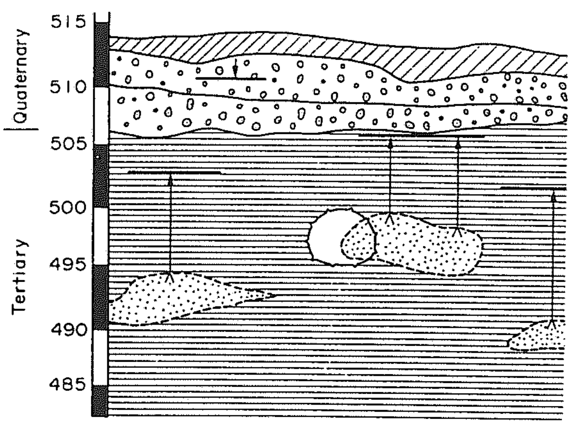


Figure 45 Stratification of subsoil indicating hydrological conditions

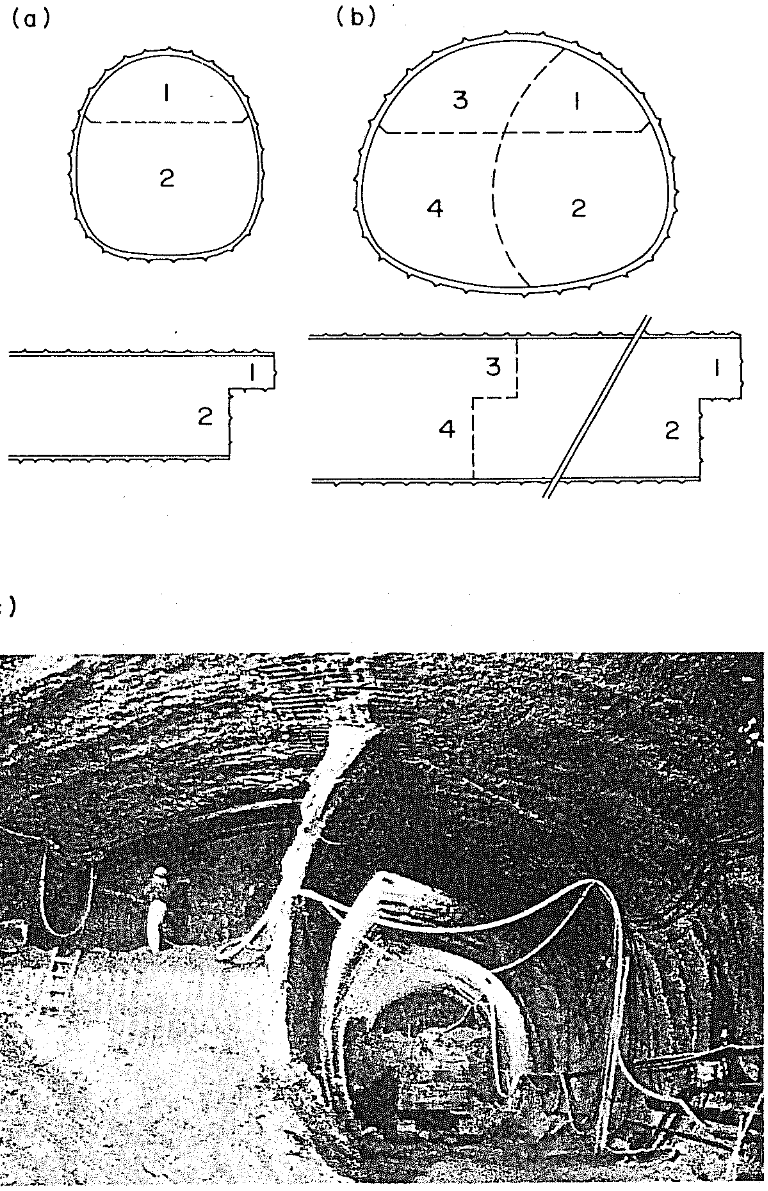


Figure 46 Method of excavation with shotcrete support: (a) single track tunnel, sectional area 38 m^2, (b) twin track tunnel, sectional area 80 m^2, and (c) photograph of twin track tunnel

procedure has proved to be very successful, being safe and having a reducing influence upon ground deformations.

20.5.4.1 Strain profiles in the subsoil due to changes in pore water pressure

In the case being discussed here, the groundwater in the tertiary sand formation was dewatered by conventional wells while the groundwater in the overlying quaternary gravel was maintained at its initial level. Decrease of pore water pressure in soils increases the effective normal stress [21] which in turn leads to compression of the material. To optimize the dewatering measures and to control the differential settlements in the different layers of the ground, strain profiles where measured with the sliding micrometer in different sections of the subway line 5/9 [34]. In Figure 47 the measured compression strains along two boreholes having a depth of 38 m are shown. The corresponding borehole logs show the start of the tertiary formation approximately at 8 m depth in both cases, whereas the stratification is different.

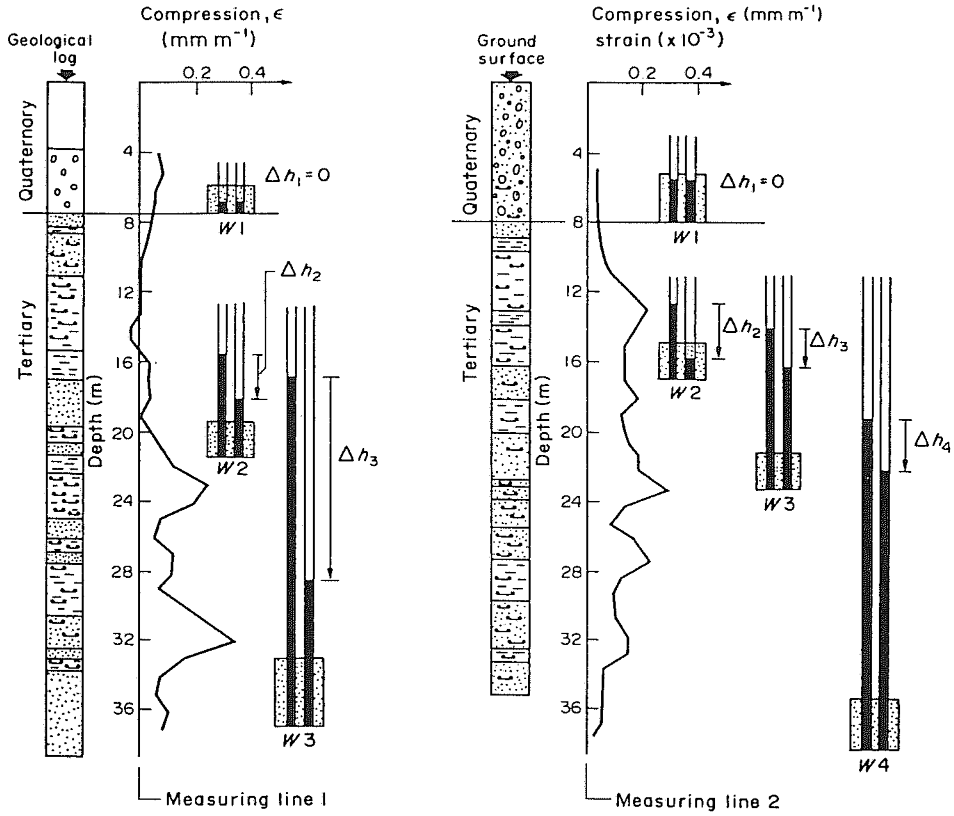


Figure 47 Strain distribution ε along two vertical measuring lines caused by partial dewatering of tertiary formation (W = observation well)

In the tertiary formation, conventional and vacuum wells were operated and their effects observed by open standpipe water level recorders. In Figure 47 the changes in piezometric heads are indicated by Δh_i for the different observation wells designated by W_i. When interpreting the measured strain distribution one has to bear in mind that apart from the details of the geology of that area also the efficiency of the pumping operation is decisive. The accumulated strains along measuring line 1 resulted in a surface settlement of about 3 mm and those of the measuring line 2 yielded 4 mm. Such surface settlements occur before the tunnel construction. Therefore, one has to instrument the boreholes to take readings well in advance.

20.5.4.2 Interaction between adjacent tunnels and the effect of compressed air

The interaction of adjacent tunnels and its effect on settlement is influenced by various factors such as the shape, span and depth of the tunnels and also by the distance between them, the method of excavation, the rate of advance, the characteristics of the subsoil and finally the groundwater conditions. Obviously the prediction of ground settlements by computational methods has major shortcomings in such complex situations. If only a limited stretch of a subway line is subjected to severe restrictions on permissible settlements, different constructional measures can be tested before the critical area is reached by the tunnels.

This was the case in Munich when undertunneling old houses with low overburden near to 'Odeonsplatz'. The tunnel section between the starting shaft and the critical area was approximately 350 m, offering a unique possibility for trial sections and an accompanying monitoring program. Along the trial stretch there were no buildings, services or major roads and therefore no severe limitations on permitted settlements. Two basically different constructional measures were tested with respect to their capability to reduce deformation. The first measure consisted of applying compressed air to control pore water pressure in the ground. The second proposal involved the excavation of the two track tunnel in five different stages (Figure 48c) instead of the commonly applied four stages (Figure 48b).

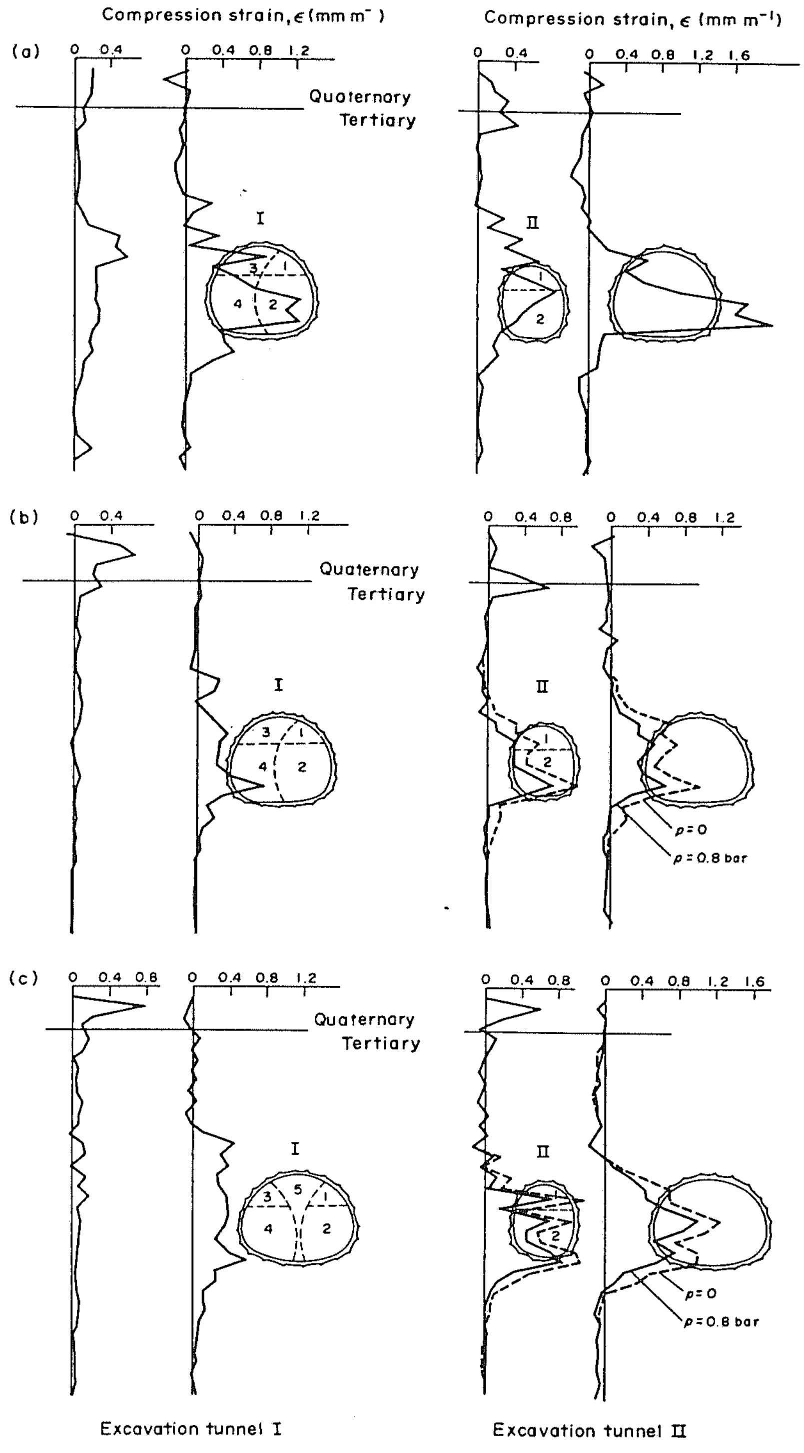


Figure 48 Strain distribution ε caused by the excavation of tunnels I and II: (a) measuring section MQ 21, atmospheric conditions, (b) measuring section MQ 20, compressed air, and (c) measuring section MQ 19, compressed air

In order to assess the most effective method of construction and to establish the distribution and intensity of the ground settlements prior to the arrival of the tunnels at the critical area, three measuring sections were installed. From the results of the comprehensive measuring program only the strain profiles measured with the sliding micrometer will be discussed here. The diagrams shown in Figure 48 reveal interesting details of the ground deformations caused by the construction of two parallel tunnels (I and II) using different excavation procedures under atmospheric and compressed air conditions. The distance between the three measuring sections was great enough to exclude interference but also small enough for the assumption of uniform ground conditions. The first section of the tunnel starting from the shaft was excavated in four steps under atmospheric conditions (Figure 48a). Next, provisions were made for compressed air application using the same four stage excavation procedure (Figure 48b). In the following section compressed air application was maintained but the method of excavation was made in five stages in the cross section (Figure 48c). In this way the benefits resulting from a more sophisticated method of excavation and from compressed air application could clearly be assessed. From Figure 48 it can be concluded that using compressed air results in markedly smaller ground deformations when compared with atmospheric conditions. On the other hand, no reduction in ground deformations can be observed due to the more sophisticated excavation method shown in Figure 48(c). Based on the unambiguous results from the trial construction sections a sound decision could be made regarding the method of construction to be applied when undertunneling the critical city area. In fact, the compressed air application (Figure 48b) was most successful throughout the whole construction section.

Two additional phenomena observed during the measuring campaign deserve to be mentioned. These are the 'pillar effect', *i.e.* the compression of the ground between the two tunnels, and the change of the pore water conditions due to the drop of compressed air pressure to atmospheric air pressure during 24 hours. The 'pillar effect' can be seen clearly from all three cases (Figures 48a, b and c), whereas the effect of a drop of air overpressure from $p = 0.8$ bar to $p = 0$ bar can be seen in Figures 48(b) and 48(c).

20.6 SUMMARY AND CONCLUSIONS

The successful design of 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 discussion. This is the main reason for the lack of standard design procedures in tunneling. The inherent weak elements in purely theoretical considerations can, however, be compensated for by direct field observations and the sound engineering experience of the designer.

Depending on the design problem, it may be necessary to make decisions well before the start of the construction. In this case, the observation of the actual deformations of the tunnel profile, the movements of the surrounding ground or the settlements at the ground surface during the excavation of the tunnel mainly have the function of checking the structural behavior with regard to satisfactory design and proper execution of the works. In contrast, using the shotcreting method with anchors or steel grid support, which may in many cases also be applied in subway construction, continuous measurements inside the tunnel and in the subsoil can serve as feedback 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 behavior of the structure, the most important corrective measures in the construction can still be applied.

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, however, exclude the conscious decision to accept a calculated risk.

Since the problem of optimization is very varied, the immediate objectives of the individual measurements may be concerned with quite different aspects, the most important of which are as follows.

(i) The investigation of the global material properties of the rock.

(ii) The determination of the type and quantity of rock pressure (loosening pressure, genuine rock pressure and swelling pressure).

(iii) The safety control of the structure.

(iv) The verification of structural response to a specific method of construction

(v) The control of the effectiveness of particular support measures.

(vi) The comparison of theoretical predictions with the actual structural behavior.

As a general rule the above classification of the objectives of measurement is not rigid. It is intended to indicate the main emphases. It should be noted that usually the same program of measurements has several aims. 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.

Field measurements are now recognized worldwide as an indispensable aid for correct decision making in tunneling. They often form the link between theory and the engineering practice. Successful measurements require both a thorough understanding of the specific problems arising in tunneling and a close familiarity with instrument techniques. In this chapter an attempt was made to show, on the one hand, the significance of monitoring by the discussion of some case histories and, on the other hand, to give information on new developments in measuring techniques.

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