The New Austrian Tunnelling Method

After describing the influence of rock-pressure effects on tunnel linings, the author underlines the inadequacy of conventional tunnel driving and lining methods in poor ground and explains the effectiveness and reliability of a new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring—called an "auxiliary arch"—the deformation of which is measured as a function of time until equilibrium is obtained. Ways are shown to determine the magnitude of active forces, which leads to dimensioning of linings on an empirical basis*. Further articles describe successful applications of the method

By Prof. Dr.techn. L. v. RABCEWICZ

PART ONE

IN the conventional tunnelling practice of the past, masonry in dressed stone or brick was regarded as the most suitable lining material in unstable rock. Concrete was rejected because possible deformation during the settling and hardening process was supposed to cause irreparable damage. The space between masonry lining and rock face was dry packed. Timber lagging, which was subject to decay when left in place, generally could not be removed, particularly from the roof, because of the danger of loosening and rockfalls.

The situation was further aggravated by a very unfavourable time factor. Merely to bring to full section a 9m-long section of a double-track railway tunnel by the old Austrian tunnelling method, after the bottom and top headings had been driven, took about four weeks, and another month was needed to complete the masonry of the section. The amount of timber used in more difficult cases was so enormous that one third and sometimes even more of the excavated space was filled by solid timber.

All these circumstances, together with the tendency of the temporary timber framework to yield, necessarily produced violent loosening pressures, which frequently caused roof settlement up to 40cm and more before the masonry could be closed. Years after construction had been finished a slow decrease in volume of the compressible and sometimes badly executed dry packing often deformed the lining asymmetrically, causing damage and costly repairs. Damage to the surrounding rock as well as to the lining itself was further increased locally by the mechanical and chemical effects of water.

It is evident that in this period of rather inadequate methods and materials for temporary and permanent supports, loosening pressures were a source of the greatest concern to tunnel engineers. All attempts to design a lining during this period were consequently carried out with sole regard to loosening pressures. Occasional subsequent deformations of linings forcibly led to the erroneous conclusion that the linings designed in this way still lacked the necessary margin of safety, whereas the failures almost without exception were due to incorrect treatment of the surrounding rock and to fundamental shortcomings of the methods.

A typical practical example of the imperfections mentioned[1]† is a double-track railway tunnel in Czechoslovakia, which was driven almost a century ago through a ridge of soft, horizontally stratified sandstone. Although the rock was fairly stable, stratification and jointing caused the corners in the roof on both sides to fall out, leaving a more or less rectangular cavity instead of an arch. The tunnel was supported by an excellent dressed-stone lining, 45cm thick, but it was not backfilled. During the following decades the unsupported layers of sandstone subsided and settled on top of the arch, causing the roof of the lining to bulge downwards (Fig. 1). Had the

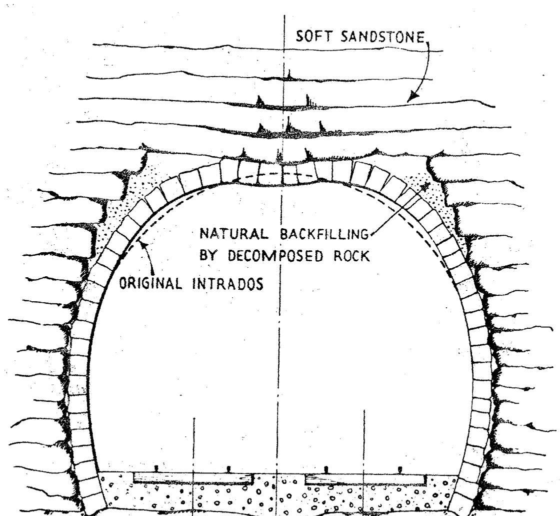

Fig. 1. Deformation of a malconstructed tunnel arch by loosening pressure

* The substance of this series was originally presented to the XIII Colloquium of the International Society of Rock Mechanics in Salzburg, October 1962, and this English version, which contains additional material, is published by arrangement with Springer-Verlag, Vienna, the publishers of *Felsmechanik und Ingenieurgeologie.*

† References will be collected at the end of the third and concluding article.

cavities ... her side behind the lining not been simultaneously filled to a certain degree by pieces of rock falling out of the weathering corners the arch would certainly have failed.

Though methods and means of temporary and permanent support have improved fundamentally since the earlier period, linings are still made as thick as they were about half a century ago. Loosening pressure is still considered by many to be the main active force to be reckoned with in tunnel design, although modern tunnelling methods actually make it possible to avoid loosening almost entirely.

Development of Construction and Lining Methods

Shortly after the turn of the century grouting was introduced as an effective means of consolidating the rock surrounding a tunnel. By filling the voids, unsymmetrical local loads on the lining are avoided, and portions of loose or soft rock are strengthened by cementation.

The next stage was the introduction of steel for supports and which, compared with timber, constituted a remarkable improvement as a temporary lining material because of its better physical properties, its higher resistance to weathering, and its reduced tendency to yield. Decreased deformability of the temporary support made it possible to replace masonry as a lining material by concrete. Dry packing then became obsolete, since the concrete filled the spaces outside the theoretical extrados.

One of the most important advantages of steel supports is that they allow tunnels to be driven full face to very large cross sections. The resulting unrestricted working area enables powerful drilling and mucking equipment to be used, increasing the rate of advance and reducing costs. Nowadays, dividing the face into headings which are subsequently widened is used only under very unfavourable geological conditions.

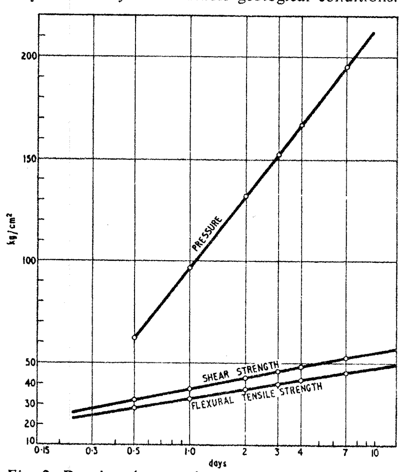

Fig. 2. Results of strength tests on shotcrete carried out in the testing laboratory of the Technische Hochschule, Graz

Remarkable progress in drilling and rock blasting, especially in Sweden, has also helped to reduce damage to the surrounding rock.

Modern Tunnelling Methods

Finally, during the last few decades, rockbolting and shotcrete* were introduced in tunnelling practice. To judge from the results obtained up to now the introduction of these methods of support and surface protection can be considered as a most important event, especially in the field of soft-rock and earth tunnelling†.

The advantages of these methods can best be shown by comparing the rock mechanics of tunnels lined by the new and by older methods. Whereas all the older methods of temporary support without exception are bound to cause loosening and voids by yielding of the different parts of the supporting structure, a thin layer of shotcrete together with a suitable system of rockbolting applied to the rock face immediately after blasting entirely prevents loosening and reduces decompression to a certain degree, transforming the surrounding rock into a self-supporting arch.

A layer of shotcrete with a thickness of only 15cm applied to a tunnel of 10m diameter can safely carry a load of 45 tons/m² corresponding to a burden of 23m of rock, which is more than has ever been observed with roof falls. If a steel support structure incorporating No. 20-type wide-flanged arches at 1m centres were used under these conditions, it would fail with 65% of the load carried by the shotcrete lining, and a timber support of the conventional Austrian type would be able to carry only a very small proportion of the same load. If the temporary support deforms or fails the erroneous conclusion is usually drawn that the proposed permanent linings are not strong enough. In this way permanent linings that are already overdesigned become still heavier.

Shotcrete as Temporary Support

A temporary support designed to prevent loosening must attain a high carrying capacity as quickly as possible, and it must be rigid and unyielding so that it seals off the surface closely and almost hermetically. The carrying capacity of a temporary support is determined by the material as well as by its structural design. Timber, especially when humid, is by far the worst; it combines low physical properties with a great tendency for the structure to yield. Although steel has much better physical properties the efficiency of steel arching depends mainly on the quality of packing between the arches and the rock face, which is always an unsatisfactory problem. On the contrary, concrete, particularly shotcrete, meets all the requirements for an ideal temporary support.

Shotcrete's high early strength is of the greatest importance in attaining a high bearing capacity rapidly, and this is particularly true of its early flexural-tensile strength, which amounts to 50 and 30% of the compressive strength after one-half and two days (see Fig. 2). A recently introduced hardening-accelerating admixture based on silification gives

* Pneumatically applied mortar, originally known as "torcrete" or "gunite" became very much improved shortly after World War II by a new type of machine designed to spray a mixture containing aggregates up to 25mm in size. For this new product the term "shotcrete" has become customary.

† The first successful application of surface stabilisation by shotcrete for tunnels in unstable ground as an integral part of the driving process instead of using timber or steel as temporary support was carried out at the Lodano-Mosagno tunnel of the Maggia hydroelectric scheme, Switzerland, 1951-1955[6, 21]. A patent was granted for this method in Austria in 1956[12].

still better results. Whereas only a few years ago, even if the water inflow was limited to dripping, efficient drainage had to be achieved before shotcrete could be applied, the new accelerator makes it possible to shotcrete a very wet surface even when dripping heavily. For instance, in one of the tunnels of TIWAG'S Kaunertal hydroelectric scheme, a ¾in jet of water was plugged off with shotcrete alone without the need to install a relief pipe.

The most conspicuous feature of shotcrete as a support against loosening and stress-rearrangement pressures lies in its interaction with the neighbouring rock. A shotcrete layer applied immediately after opening up a new rock face acts as a tough surface by which a rock of minor strength is transformed into a stable one. The shotcrete absorbs the tangential stresses which build up to a peak close to the surface

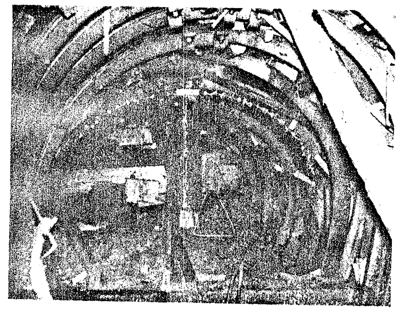

Fig. 3. Steel-supported tunnel which failed when reaching a zone of kaolinised gneiss under an overburden of 250m; water inflow 35lit/s

of a cavity after it is opened up. As a result of the close interaction between shotcrete and rock the neighbouring portions of rock remain almost in their original undisturbed state and are thus enabled to participate effectively in the arch action. The statically effective thickness of the zone of arch action is in this way increased to a multiple of that of the shotcrete. In this way, tensile stresses due to bending are diminished and compressive stresses are easily absorbed by the surrounding rock. The zone of arch action can be increased at will by rockbolting.

Disintegration always starts by the opening of a minute surface fissure; if this movement is prevented at the outset by applying a shotcrete layer the rock behind the shotcrete remains stable. This explains why cavities in bad rock lined with a skin of only a few centimetres of shotcrete remain in perfect equilibrium. Shallow tunnels in rock of medium quality, when built by customary methods, need a fairly strong temporary support and concrete lining. When the new method of surface stabilisation is adopted, only a thin layer of shotcrete, possibly locally strengthened by rockbolts, will provide both temporary support and a satisfactory permanent lining.

Experience so far has shown that shotcrete, especially when combined with rockbolting, has proved unexcelled as a temporary support for all qualities of rock with standing times down to less than one hour and even for ground which normally ... ld only be mastered by careful forepoling. Exceptionally, even almost cohesionless and plastic ground has been

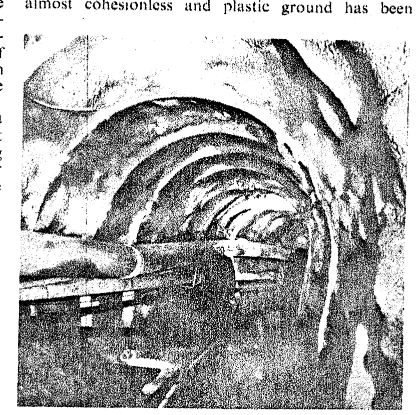

Fig. 4. The tunnel shown in Fig. 3 successfully reconstructed by redriving the deformed portion in steel and strengthening it by shotcrete

successfully handled. In very bad cases of plastic waterbearing ground where steel forepoling failed, shotcrete has been successfully employed as a stabilising reinforcement for steel support, and an example is given in Figs. 3 and 4. For reasons which we shall not discuss here, a tunnel of 8m² section for a hydroelectric scheme in the Austrian Alps had originally been driven without shotcrete, using only steel arches and steel lagging. When the tunnel, the overburden of which was 250m, reached a tectonically disturbed zone in a completely crushed kaolinised gneiss with heavy water inflow, the pressure became so heavy that the arches were deformed and their footings forced into the ground. The heavy water inflow could only be relieved slightly as the water discharge pipes became clogged shortly after placing. With the situation as shown in Fig. 3 excavation had to be stopped.

To reconstruct the deformed tunnel the contractor returned to the still undeformed portion and embedded the steel arches in a 30cm lining of shotcrete. After redriving the roof in the deformed portion new steel arches had to be placed at 60cm centres on heavy wooden sills and another arch interposed between each set. As soon as a set was placed the surface was immediately shotcreted to a complete ring (see Fig. 4). This difficult situation, which had been greatly aggravated by unsuccessful attempts at driving, was thus mastered without any further setbacks.

Effects of Stress-Rearrangement Pressures

When a cavity is made in undisturbed rock the original stress pattern is disturbed. In the course of time, the duration of which depends on the properties of rock, a new stress situation appears in the neighbourhood of the cavity, and equilibrium is attained either with or without the assistance of a lining

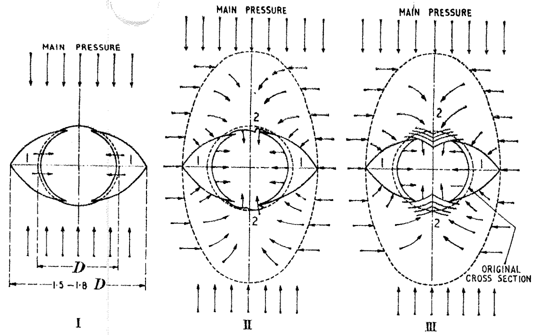

Fig. 5. Sketch of mechanical process and sequence of failure around a cavity by stress rearrangement pressure

according as to whether the shear strength of the rock is or is not exceeded. This stress rearrangement is mechanical and progressive, and generally occurs in three stages (see Fig. 5) provided the rock in the neighbourhood of the cavity has not been disturbed by earlier tunnelling. At first, wedge-shaped bodies on either side are sheared off along the Mohr surfaces and move towards the cavity, the direction of movement being vertical to the main pressure direction (*I*). The increased span thus produced causes the roof and floor to start converging (*II*). In the next stage this movement is increased; the rock buckles under continuous lateral pressure and may protrude into the cavity (*III*).

Pressures arising from this action are correctly termed "squeezing pressures." Stage *III*, though frequent in mining, is seldom encountered in civil engineering.

During the days of conventional tunnelling practice the effects of stress-rearrangement pressures were not sufficiently well known. Moreover there was no means of clearly recognising the progressive occurrence of pressure phenomena as described above, because, with the obsolete methods then used, the sections were usually not driven full face but divided into headings which were subsequently opened out. Measurement of deformations was most unusual.

Behaviour of Linings Subjected to Rearrangement Pressures

Conventional multiple-hinged arches of stone masonry withstood rearrangement pressures in different ways. Frequently the timber lining deformed during construction to such a degree as to allow the appropriate Trompeter zone to be formed, so that permanent equilibrium was attained without any or only significant lining damage such as crushing of mortar in joints or spalling of the edges of the masonry. However, in many cases linings failed by shearing off after a pattern typical of this kind of strain and had to be rebuilt.

One of the most frequent reasons for these failures—besides many other shortcomings of the customary methods—was the lack of an invert. Not only were inverts provided much too seldom but they were usually placed much too late—generally not until the roof and sidewalls of the entire tunnel had been finished. Given a rock of insufficient strength even linings of mammoth dimensions were bound to deform heavily in the absence of an invert, because they form a shell without any bottom bracing.

With the forming of the Trompeter zone, which we shall call the "protective zone," surface stresses decrease markedly while the surface deforms. The radial stress which must be counteracted by the necessary bearing capacity at the periphery of the cavity, which we shall call "skin resistance," becomes smaller as the peak of tangential stresses moves away from the cavity, the radius of which simultaneously decreases.

These relations are mathematically described by the equations of Fenner-Talobre[?] and Kastner[20].

$$p_i = -c\cot\phi + p_o\,[c\cot\phi + (1-\sin\phi)]\left(\frac{r}{R}\right)^{\frac{2\sin\phi}{1-\sin\phi}}$$

and shown schematically in Fig. 6. p_i = skin resistance, c = cohesion, ϕ angle of internal friction, R = radius of the protective zone, r = radius of cavity, $p_o = \gamma H$. H = overburden. By omitting the cohesion the

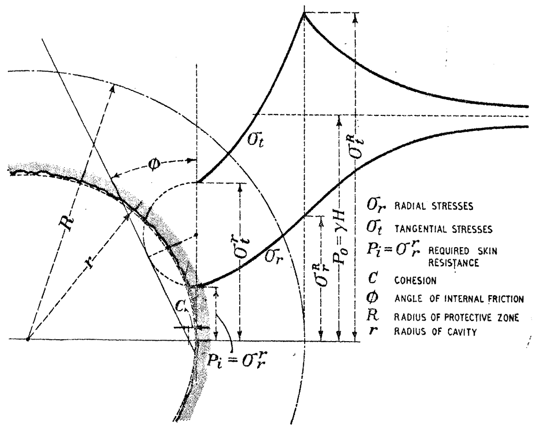

Fig. 6. Schematic representation of stresses around a circular cavity with hydrostatic pressure (after Kastner[20])

equation is simplified to $p_i = p_o\,(1-\sin\phi)\left(\frac{r}{R}\right)^{\frac{2\sin\phi}{1-\sin\phi}}$ $= np_o$. In Fig. 7 the values of n are given as a function of p_o and ϕ. With $R = r$, i.e., without formation of a protective zone, $p_i = p_o\,(1-\sin\phi)$ and the cavity attains equilibrium without any deformation.

The practical interpretation of these theoretical findings is that with a very yielding support structure, having small skin resistance, the protective zone increases but simultaneously the skin zones loosen up, and ϕ decreases. Should loosening eventually become so great that open cracks and seams are formed, the skin zone loses its bearing capacity almost entirely, which has practically the effect of a latent increase of span.

Nevertheless, these theoretical considerations do not altogether explain satisfactorily the extremely high skin resistances actually required in plastic ground when applying obsolete methods of temporary support. The reason must probably be sought in the time element. The formation of the protective zone does not arise simultaneously with the decrease of ϕ, for whereas the latter follows the excavation almost immediately, decrease of stresses due to stress rearrangement (protective zone) needs more time.

A temporary means of support to meet these complicated conditions to the best advantage must first seal the newly exposed rock face as quickly as possible; secondly, it must have sufficient skin resistance to prevent serious loosening; and thirdly it must still be sufficiently yielding to allow a protective zone to be formed.

To comply with these requirements the author tried out during the war a new method called the "auxiliary arch" which consisted of applying a relatively thin concrete lining to the rock face as soon as possible, closed by an invert and intended to yield to the action of the protective zone. Deformations of the auxiliary arch were measured continuously as a function of time. As soon as the observations showed a stabilising trend of the time/deformation curve, another lining called the "inside lining" was constructed inside. The method can be considered as a real predecessor of the "New Austrian Tunnelling Method," as it comprises all its integral features with the exception of the modern means of surface stabilisation.

At that time the method had the great disadvantage that tunnels had to be driven using obsolete methods of temporary support, which necessarily caused far too much loosening before the auxiliary arch could be built. The situation has changed with the introduction of modern tunnelling methods. By applying a layer of shotcrete to the rock face immediately after driving, or if necessary even as an integral part of the driving process, with rockbolts for additional support, an auxiliary arch is formed which complies in every aspect with the requirements for a temporary lining as described above.

There are still some difficulties to be overcome in normal methods of construction, as inverts are still usually built last of all, leaving the roof and sidewalls of the lining to deform at will. In the meantime, experience has taught us that it is by far more advantageous from all points of view, and frequently even imperative, to close a lining to a complete ring at a short distance behind the face as soon as possible. To comply with this requirement, tunnels should

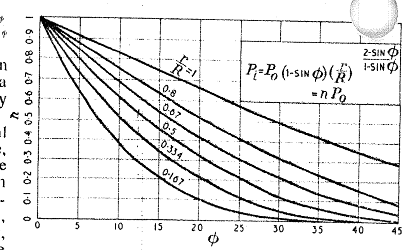

Fig. 7. Skin resistance p_i required to establish equilibrium of a cavity as a function of ϕ angle of internal friction and $p_o = \gamma H$

be driven full face whenever possible, although this cannot always be done, particularly in bad ground, where it often becomes necessary to resort to heading and benching. In the most difficult cases it may even be necessary to drive a top pilot heading before opening it out to full section.

An auxiliary arch executed in the upper heading (Belgian roof arch), though fairly effectively preventing roof loosening, represents an intermediate constructional state, which is still subject to lateral deformation. Such instability has to be removed as soon as possible by excavating the bench and closing the lining by an invert.

Design of Auxiliary Lining

When designing an auxiliary lining the magnitude of the active forces, the admissible stresses and the safety of the crew have to be considered. Accidents arising from geological causes, invariably brought about by rock falls or slides as a result of loosening or effects of water, can be avoided almost entirely by a conscientious geological survey. Nevertheless, as a general measure of safety, shotcrete linings should be reinforced by light steel sections and relief pipes installed to prevent a build-up of seepage pressure. Deformation resulting from stress rearrangement never occurs in the form of roof falls but as a slow displacement. Such deformations are in no way dangerous to the crew, particularly as the movements are normally very slow.

Under such favourable safety conditions permissible stresses may safely be taken close to the rupture limit, particularly if the auxiliary arch is intended only as temporary support. Should it adopt a permanent character a safety factor of 1·5 to 2 is sufficient.

Active forces can be roughly estimated from the formulae given above, but there seems no danger in adopting rather low values for these forces since a considerable factor of safety results from the interaction of the shotcrete with the neighbouring rock, particularly with higher values of ϕ, a static effect which is not taken into consideration by computing the thickness of the lining.

Though seemingly rather daring at first, these design rules have been tested in practice and proved correct in constructing many kilometres of tunnels, as will be described in the succeeding articles which are devoted to actual examples and to deformation measurements in a test tunnel.

(To be continued)

The New Austrian Tunnelling Method

In this second article the author describes a number of actual tunnels, in various countries, in the construction of which the new Austrian method has been applied successfully

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PART TWO

INTERESTING practical examples of stress-rearrangement effects, as well as of the soundness of the design rules for auxiliary shotcrete linings enunciated in the first article, have been encountered during the construction of numerous pressure and diversion tunnels for the Tiroler Wasserkraft A.G. (TIWAG) Prutz-Imst and Kaunertal hydroelectric schemes. The author has also had the opportunity to observe the phenomena described in a series of tunnels built abroad in accordance with the new methods. In the Kaunertal scheme alone about 70km of tunnels were built with locally rockbolted auxiliary shotcrete linings as an essential part of the driving procedure (Fig. 8). Amphibolites, schistose gneisses, eye-gneisses and mica slates of all possible qualities down to the worst have been penetrated by tunnels with cross-sectional areas ranging from 10 to 20m² and overburdens up to 1,100m.

The working sequence shown in Fig. 10 was used almost without exception. The auxiliary shotcrete lining (stage *III*), consisting only of roof and sidewalls of 5–15cm thickness, was as a rule left without an invert for a very long time. In some cases it was a year or more before the invert was placed. The side-walls were thus bound to deform under pressure to various degrees according to the quality of the rock. Particularly in the sections cutting across mylonites were local deformations up to 25cm observed, causing heavy cracking. In places the sidewalls had to be re-peatedly redressed and freshly shotcreted (see Fig. 9). Equilibrum was obtained eventually by the use of additional Perfo rockbolts. Characteristically the roofs showed no signs of pressure anywhere. Roof pressure by loosening—the prevailing cause of trouble with the older methods—did not arise at all.

In those cases of swelling pressure caused by the increase in volume of tectonically preloaded clayey or marly materials[19] due to water absorption when the pressure is released, systematically applied deep anchoring by Perfo or SN-type rockbolts grouted in place has proved so far to be the sole means of obtaining equilibrium with a minimum of deforma-

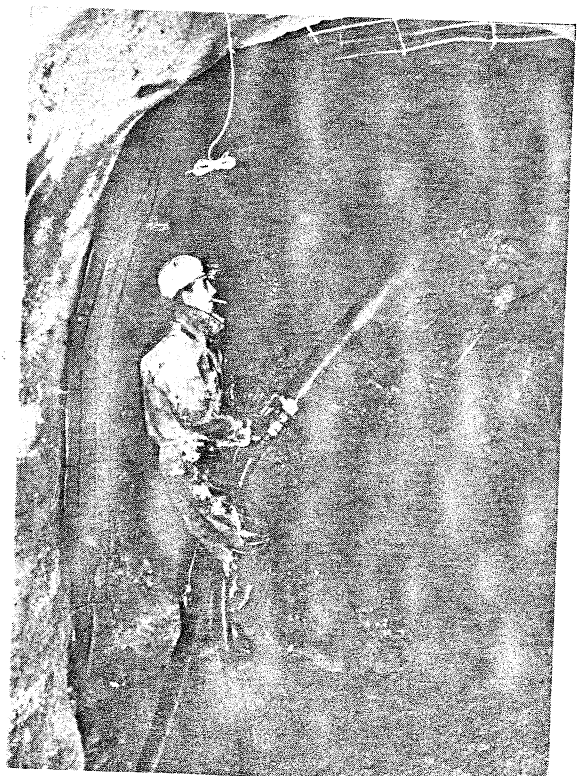
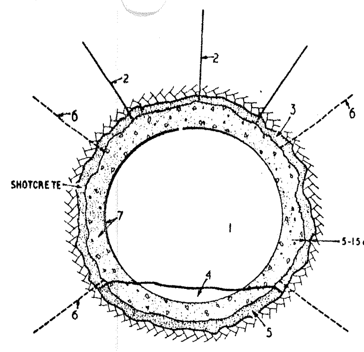


Fig. 8. Tunnel wall receiving surface protection by a shotcrete layer immediately after mucking out

Fig. 9. Shotcrete lining failure because Perfo anchors and bottom bracing were not applied in good time



1. Excavation; 2. Anchoring; 3. Shotcreting roof and walls; 4. Excavating invert; 5. Shotcreting invert; 6. Rock grouting; 7. Steel lining and concreting

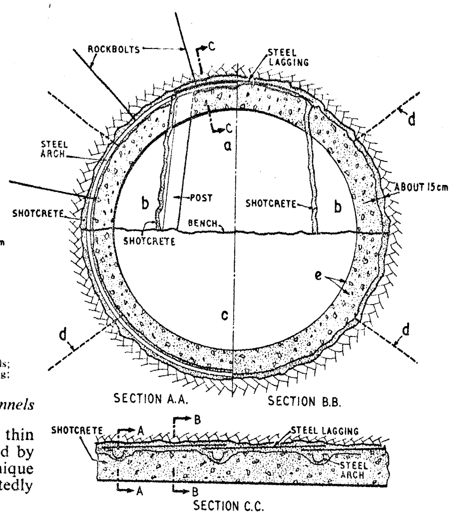
Fig. 10. Construction sequence for the Kaunertal tunnels

tion. To keep the surface from slackening a thin layer of a few centimetres of shotcrete reinforced by a net has best served the purpose. This technique developed many years ago[4] has since been repeatedly confirmed.

For the Kaunertal scheme an inclined pressure shaft of fairly exceptional dimensions was also driven, 1,650m in total length, the lower part, 650m long, inclining at 42° and the upper part at 20°. Its cross section was 16m² and the average overburden 150–200m. Geological conditions were rather unfavourable, for the shaft was driven through mica slates and sericite which was partly mylonitised and very wet.

The sequence of operations in the inclined pressure shaft, which differs from that described above for the tunnels, is shown in Fig. 11. The rock was mostly so bad that the top heading could not be driven full face and had to be started by a pilot heading. For safety reasons the roof had to be secured further by a steel-arch segment placed on timber props, together with some channel section as lagging. Immediate shotcreting reinforced by rockbolting had to be carried out not only in the roof but also as the sidewalls and breast of the heading. After widening the heading and extending the steel arches, shotcrete protection was continued up to the centreline, Fig. 12. Two to three weeks after finishing the top heading the bench was excavated in lengths of about 100m. The steel arches were closed and shotcreting was applied immediately after the excavation (Fig. 13).

Though the lining of the top heading remained unbraced for about a month, in spite of very unfavourable geological conditions no visible signs of deformation could be observed. Measurements



a. Top heading; *b.* Widening roof; *c.* Excavating bench and invert; *d.* Rock grouting; *e.* Steel lining and concreting

Fig. 11. Construction sequence for the Kaunertal pressure shaft

made after completion of the auxiliary shotcreting showed that the deformations were decreasing in a range of the order of hundredths of millimetres in several months showing that the rock was practically in equilibrium.

The following conclusions can be drawn from these experiences. When suitably sealed by shotcrete closed to a complete ring, only relatively small deformations

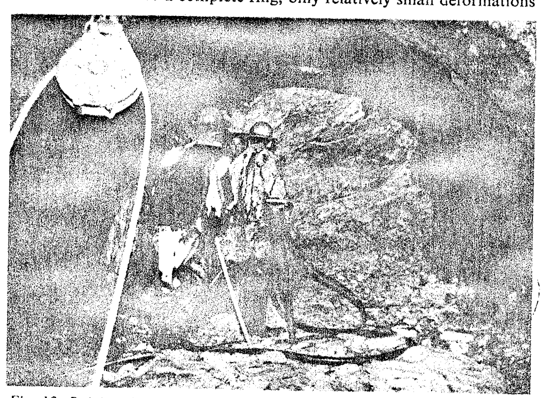


Fig. 12. Driving the top heading in the Kaunertal pressure shaft; protection by steel arches and shotcrete

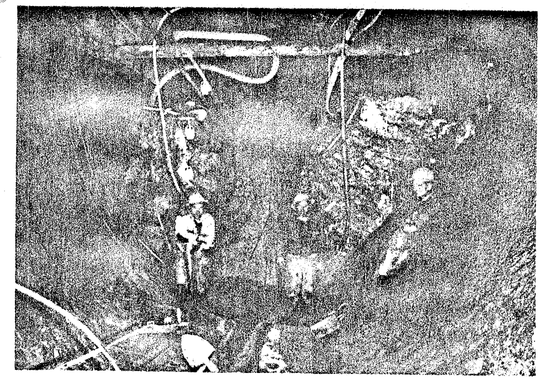


Fig. 13. Benching out the Kaunertal pressure-shaft invert; protection by steel arches and shotcrete

of the slightly decompressed surrounding zone are sufficient to attain equilibruim. By applying the new methods correctly, even rock of fairly bad quality can be handled successfully by heading and benching provided the auxiliary lining is closed by placing the invert as soon as possible, the actual time being determined by the quality of the rock. An auxiliary shotcrete lining having a thickness of only $^1/_{40}$ to $^1/_{50}$ of the diameter suitably reinforced by steel arches and rockbolts is sufficient to allow the lining to be closed without damage by deformation. It is important that the lining be made as thin as possible to allow a relatively large deformation without being damaged to any extent.

We shall now compare these favourable results obtained with the new means of stabilisation with

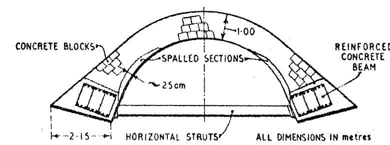


Fig. 14. Roof-arch failure in a clayey-mylonite section of the new Semmering tunnel. The masonry consists of shaped concrete bricks, and high early-strength cement was used for these and for the reinforced-concrete beams

observations made only 15 years ago during the construction of the Semmering tunnel by the Kunz method—a modification of the Belgian method. This is a typical example of the formerly common but erroneous practice of overdimensioning the permanent lining, because of violent deformations that occurred during the intermediate constructional stages as a result of static instability.

A roof arch braced by heavy timber struts in a section of clayey mylonites (Fig. 14) had been constructed in concrete and concrete bricks. To excavate the section a bottom heading was driven followed by a top heading, the latter being subsequently widened to the extent shown in the picture. By the time the masonry of the arch had been closed the roof had settled 40–70cm. Lateral deformation of the order of 2cm daily not only led to spalling (see Fig. 15) but also caused invisible damage inside the arch, probably similar to the very typical destruction shown in Fig. 16 and the 40cm-diameter timber struts at 1·50m centres were compressed and buckled. Construction was continued by excavating the remaining portion and completing the lining. Movement stopped in the course of time and equilibrium was finally attained. There can be no doubt that by far the greatest part of the distortion of the lining was due to loosening (stage *II* in Fig. 5) which consequently led to progressive softening of the rock. In view of the great damage the lining suffered by violent distortion it is obvious that a much thinner lining would have sufficed provided the distortion was kept inside the elastic range.

A similar example is that of the Serra Ripoli 422m super-highway twin tunnel on the Autostrada del Sole Bologna-Firenze in Italy, which was driven through an extensive old slide consisting of chaotic

Fig. 15. Spalling of the concrete bricks in the Semmering tunnel as indicated in Fig. 14

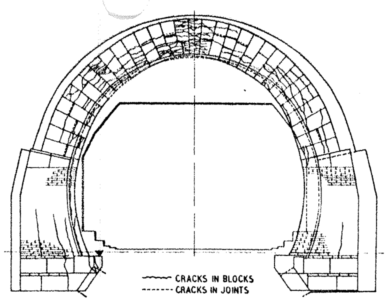


Fig. 16. Destructive deformation of a lining in Karawanken tunnel by stress-rearrangement pressures caused by the absence of an invert

masses of loam and boulders, superposed on layers of black flaky clay with interstratification of more or less compact layers of marl and sandstone. Two sections, each about 90m long, of particularly bad ground consisting of plastic clay, were encountered near the portals. The geological conditions were apparently even somewhat worse than at Semmering.

The twin tunnel was started by driving one tube with a cross section of 110m² by the German method of leaving a core in the middle against which the walls were strutted. When opening out the top heading, loosening pressure became locally so great that the timbering settled from 1m to 1·80m (Fig. 17) which necessitated extremely difficult and expensive redressing. Although the tunnel was finally completed by this method, the experience was so discouraging that the management decided to try surface stabilisation by shotcrete for the second tube. Instead of continuously struggling with masses of timber leaving no room for tunnelling equipment, it then became possible to drive the top heading full face, using customary machines for mucking and transport (Fig. 18). The average rate of advance was trebled, no settlement occurred, and an average financial saving of 20% compared with the first tunnel was achieved. It must be mentioned that by taking full advantage of the possibilities of the new method the thickness of both the auxiliary and inside linings could still have been considerably reduced. The Serra Ripoli tunnel was built over the period 1957–1960.

There is no doubt that the difficult section of the new Semmering tunnel described above, as well as any other similar case, could have been executed successfully by applying only a thin auxiliary shotcrete lining reinforced by light steel arches and rockbolts. In very bad ground, however, it would be advisable to protect the roof, walls, face, and exceptionally even the floor, by shotcrete, at intermediate constructional stages.

Another super-highway twin tunnel constructed in 1957–1958 using anchoring and shotcrete was built in Venezuela in thin laminated limestone interleaved with clayey graphitic layers, the maximum overburden being 100m. The originally proposed reinforced-concrete lining, from 60cm thick at the top to 1m at the sidewalls, was changed on the author's advice to a layer of shotcrete with an average thickness of only 20cm for both the auxiliary and permanent linings together, and reinforced by systematically applied prestressed Perfo rockbolts. The final shotcrete closely followed the irregularities of the rock (Fig. 20). Having regard to the overbreak

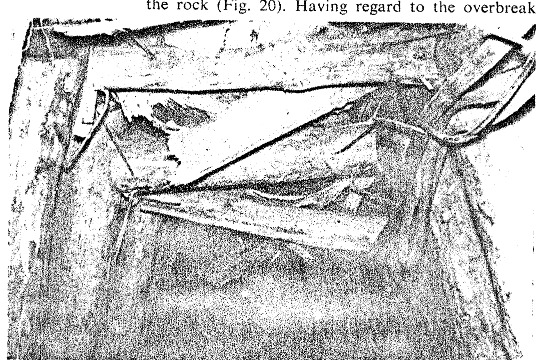


Fig. 17. Failing timber frames in the upper heading in the first tube of the Serra Ripoli super-highway twin tunnel, Italy

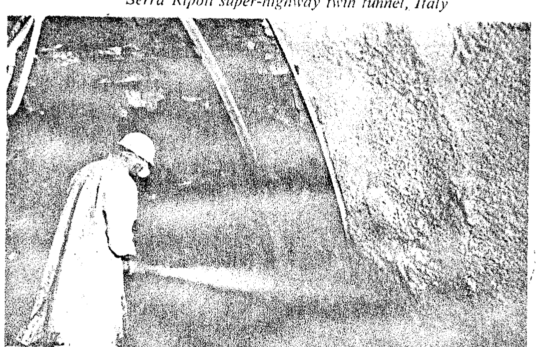


Fig. 18. Full-face driving of the roof section in the parallel tube of the Serra Ripoli tunnel, using shotcrete reinforced by sectional steel. Same geological conditions as in Fig. 17

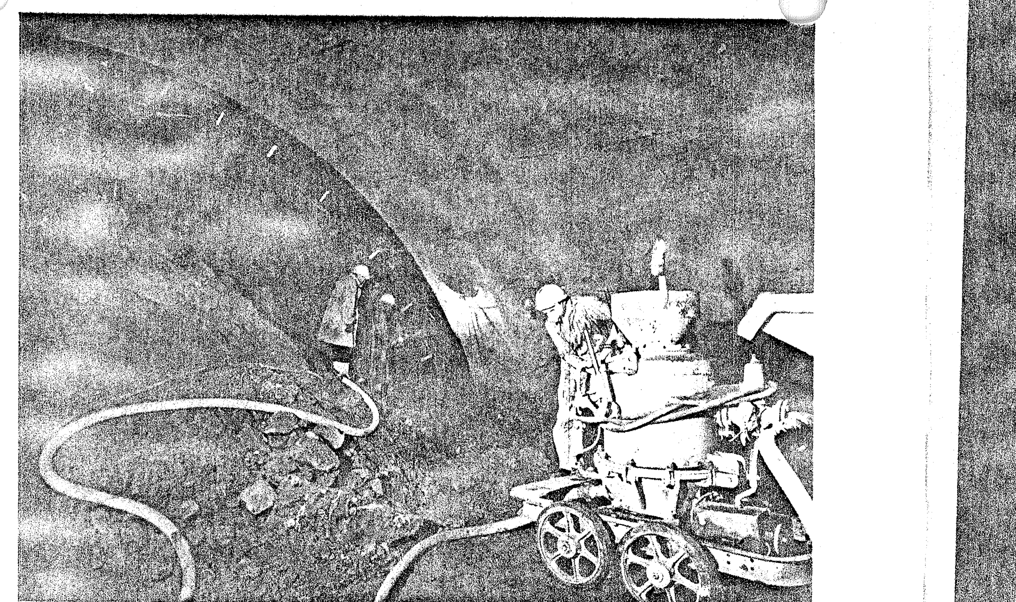


Fig. 19. Super-highway tunnel in Italy south of Firenze. Forming the auxiliary lining of the upper part of the tunnel with shotcrete reinforced by steel netting and steel arches. Contractors: Impresit G. Lodigiani, Milan

together with the low quality of rock, the two tunnels were placed much too close to each other, leaving in places only 6m of poor rock between them. In spite of this error in design which caused very unfavourable pressure conditions for the intermediate pillar the structure remained in perfect equilibrium. Changing over to modern methods resulted in an economy of about 25%.

In the case of the Venezuelan tunnel just described, Perfo rockbolts were used as the main means of stabilising the shotcrete, which was otherwise left unreinforced. A super-highway tunnel built in 1962 south of Firenze used much the same process of construction as the Serra Ripoli tunnel, and a view inside this tunnel is given in Fig. 19.

In Sweden an equipment has been constructed called "The Robot" which enables the roof of large tunnels to be sprayed with shotcrete immediately after blasting. The equipment incorporates a beam cantilevering over the muck pile, and the operator thus works safely under the portion of roof already protected by shotcrete.

(To be continued)

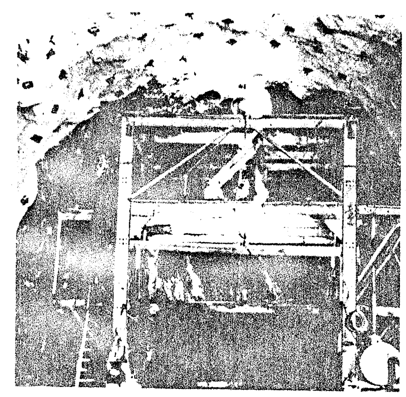


Fig. 20. Full-face driving of a super-highway twin tunnel in Venezuela using shotcrete and prestressed Perfo rockbolts

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Allis-Chalmers Engineering Review. The issue of this publication for the first quarter of 1964 has been received from Allis-Chalmers, Milwaukee, Wisconsin, and contains articles on various subjects of interest. These include the first of a series on the science and art of circuit breaking, an article on a new design of edgewound field coil, and another on a fixed-electrode water rheostat which provides a variable load.

The New Austrian Tunnelling Method

In this final article the author stresses the value of rock deformation measurements in determining the thickness of the lining. A test tunnel has been constructed in Austria to investigate this subject. A failed tunnel construction which was rescued by applying the new Austrian technique is described

By Prof. Dr. techn. L. v. RABCEWICZ

PART THREE

THE fundamental importance of deformation measurements with respect to time was emphasised by the author 20 years ago. A knowledge of the displacement of linings and of the surrounding rock as a function of time not only makes it possible to determine whether equilibrium will be reached or not, but can also be regarded as a valuable means of ascertaining the magnitude and distribution of the forces around a cavity. Further information, such as checking the formation of the protection zone by geophysical measurements as was done in the case of the Isère tunnel, is naturally extremely valuable.

Based on all this practical experience and theoretical findings a new tunnelling method—particularly adapted for unstable ground—has been developed which uses surface stabilisation by a thin auxiliary shotcrete lining, suitably reinforced by rockbolting and closed as soon as possible by an invert. For the first time in tunnelling history systematic measurement of deformations and stresses enables the required lining thickness to be evaluated and controlled scientifically. The method has been called "The New Austrian Tunnelling Method," since Austrian engineers have taken a decisive part in its development.

A conspicuous example of an almost complete application of this method is given by a 400m long highway tunnel built recently in the southern part of Austria. With the exception of a small portion in limestone the tunnel is situated in slightly waterbearing graphitic argillaceous schists, partly very soft and

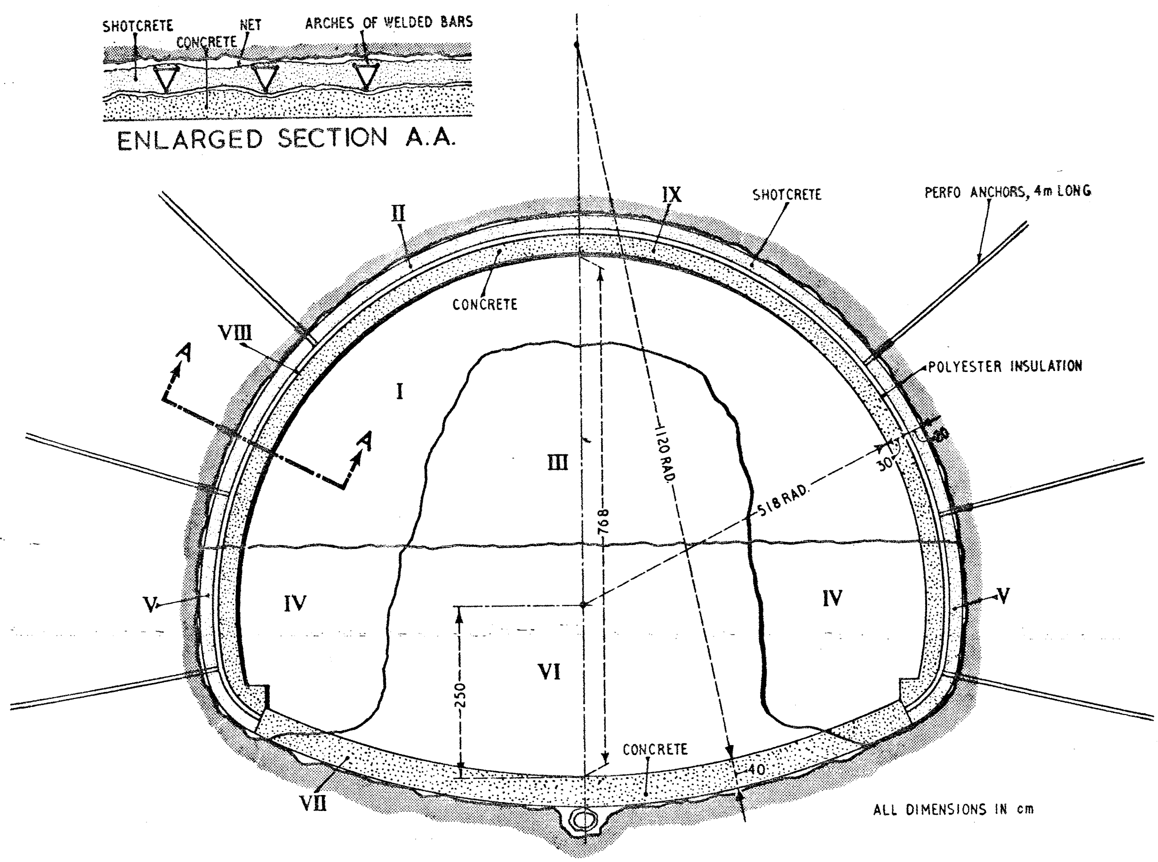

Fig. 21. Construction sequence for road tunnel in southern Austria

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in various degrees of disintegration. The maximum overburden was 60m, the strike, though very irregular, was generally about 60° to the axis, with a steep dip of about 60–70°.

The tunnel was started according to the Belgian tunnelling method in very bad ground by driving the upper half of the section at a speed of about 25m a month. As temporary support a very strong auxiliary shotcrete lining 45 to 60cm thick reinforced by arches of welded steel bars and netting had been applied. When the top heading had reached 67m the bench was started by the customary procedure of making a centre trench, but as soon as a small portion of the sidewalls was excavated and shotcrete lined, heavy settlement occurred and the first 27m of the tunnel caved in. Though the top heading had been carried out successfully by immediately lining it by shotcrete, the construction was bound to fail, as not only the method of driving but also the design—which did not even provide an invert—fundamentally violated the principles explained in these articles.

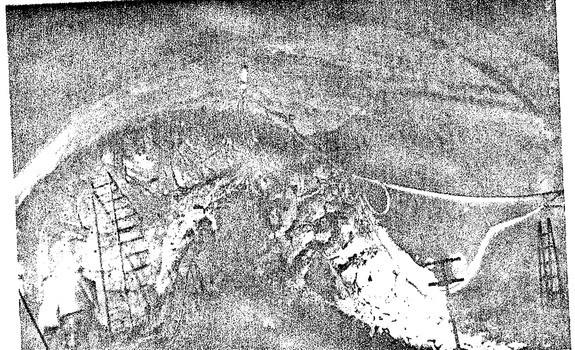


Fig. 22. Excavating top heading and forming auxiliary lining (Stages I and II)

The author, commissioned by the owner to advise on the repair of the damage and on further construction, introduced as soon as possible a system of measuring deformations. After finishing the reconstruction of the collapsed portion a method of driving was carried out as shown in Figs. 21–25. The top heading was excavated in rounds of 0·6–1·20m in the form of a half-ring, leaving the central portion to brace the walls against bulging. Immediately after finishing the excavation *I* a net was attached to the circumference, which was then protected by a 3–5cm layer of shotcrete (Fig. 22), and after placing the welded arch (Fig. 23), the auxiliary lining was increased to final thickness *II*. Then the core *III* was removed, and the walls were subsequently excavated *IV* and provided with a shotcrete lining also reinforced by a net and arches *V*. Finally the bench *VI* was excavated and the invert concreted *VII*. While stages *I* to *V* were advanced daily, *VI* and *VII* were carried out once a week. The platform of the bench was left as short as possible—just enough to give the mucking machine space to turn. In this way the concreted invert followed the face at a distance of 15–25m. The time interval between excavating the face and closing the lining by concreting the invert consequently never exceeded 15 to 25 days.

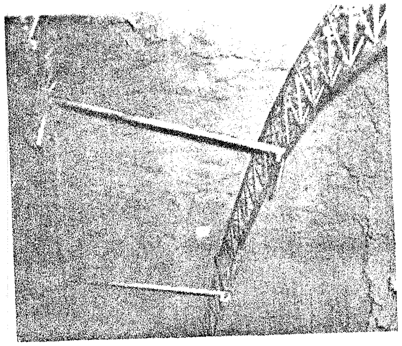


Fig. 23. Placing the welded arch

At a distance of 100–150m behind the face the lining was insulated by spraying glassfibre-reinforced polyester *VIII* (Fig. 25), followed at a short distance by concreting of the inside lining *IX*.

The time interval between operations *VII* and *VIII* of 3 to 5 months was required to observe the movements of the auxiliary arch and establish deformation/time graphs for it. Measurement was carried out weekly by checking the vertical and horizontal diameters every 15m of tunnel length. In addition, measuring anchors, 6m long, were inserted in both sides, consisting of a steel bar inside a pipe, fastened at the outer end in the rock and provided with a measuring device at the inner end to establish the extension of the rock by measuring the movement of the lining relative to the point of fastening. Further absolute movements were measured geodetically of the top of the auxiliary arch. Diameters and anchor deformations were measured with an accuracy of 0·1mm, and the geodetic measurements of the roof with an accuracy of 0·5mm.

Although the methods of measurement used were rather simple and could be improved upon in many ways they entirely served the intended purpose, which was to prove that in a rock as bad as that in question a shotcrete lining of only 20cm thickness and of a compressive strength of 250kg/cm² in 28 days, reinforced as described, sufficed to reach equilibrium in a short time after the arch had been closed by the invert. Fig. 26 gives typical examples of the numerous graphs established for each measuring section, which show throughout almost the same course. Though

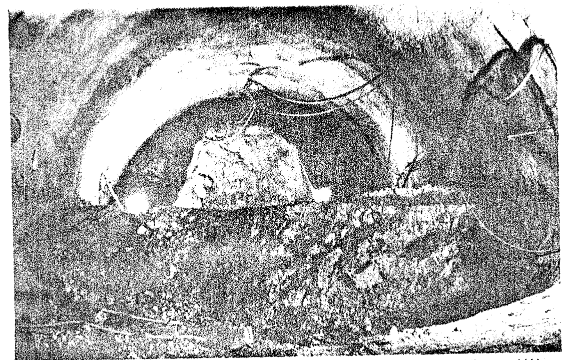


Fig. 24. Benching (Stage VI) and concreting the invert (Stage VII)

frequently the first and—as can be guessed from several observations—considerably greater part of the deformations could not be registered, since the measuring points had to be placed some distance behind the face, the deformation/time graphs clearly show without exception the rapidly tranquillising trend of movements and the obvious reason—the closing of the auxiliary lining.

It was never intended with this simple and cheap way of measurement to gather sufficient data to calculate the actual magnitudes of the active forces. Nevertheless, it has been definitely ascertained that with the present geological conditions a 20cm shotcrete lining provides a certain, though unknown, margin of safety, and that by applying an additional inside lining 30cm thick this safety factor is multiplied correspondingly. By additionally measuring the stresses by decompression slits the actual strains of the auxiliary lining could be established and its required thickness computed.

Test Tunnels

Unfortunately measurements of all kinds necessarily cause some trouble to the crews, since they reduce progress somewhat by repeated small losses of time. Experience has shown that carrying out of systematic observations is rather difficult, particularly if great accuracy is called for. Regular observations requiring high exactitude can consequently better be accomplished in proper test tunnels, the purpose of which is only scientific research.

A practical example of the foregoing is afforded by a 1km test tunnel which was driven in southern Austria to provide the basic design data for a twin road tunnel. The most favourable tunnel line was chosen so as to cross deposits of slightly to fairly well cemented sand of medium grain size for three-quarters of its length, while the rest was in fairly stable clay. The maximum overburden was 80m. The research work in the test tunnel was carried out by Dr. Ing. L. Müller and Dipl. Ing. F. Pacher, the Salzburg consulting engineers, in collaboration with Interfels, who provided the instruments.

The actual test section consisted of a circular gallery, 2·5m in diameter, driven in the sand in regular advances of 1m. Immediately after mucking, the freshly exposed rock face was covered with a 5cm layer of shotcrete; then measuring pins were placed at the circumference and the radial and polygonal distances measured by the instrument shown in Fig. 27 at regular intervals. In order to obtain absolute test results, the axis was centred optically by a precision theodolite before readings were taken, the theodolite being rigidly mounted in a chamber at the entrance to the tunnel. Readings were continued for a year (Figs. 28 and 29.)

Simultaneously the physical properties of the shotcrete with respect to time were tested by measuring the elastic moduli and the compressive strength. The properties of the ground were established by load tests. After deformations had obviously stopped, the ultimate stresses in the shotcrete were measured by three-point strain gauges, of the Electricité de France type, thus providing a check on the final readings. Ultimate stresses in the shotcrete measured in this way were as much as 26kg/cm², and the average proportion of load carried by the shotcrete lining ranged from 0·17 to 0·33γH. Load diagrams in which deformations are related to load and time for the shotcrete as well as for the rock (see Fig. 28) were prepared for each test point. The load curve of the shotcrete can be plotted by determining time element, deformation and *E* value for each point, and then computing the load. Since the process of evaluation is still in the development stage it will not be described in greater detail.

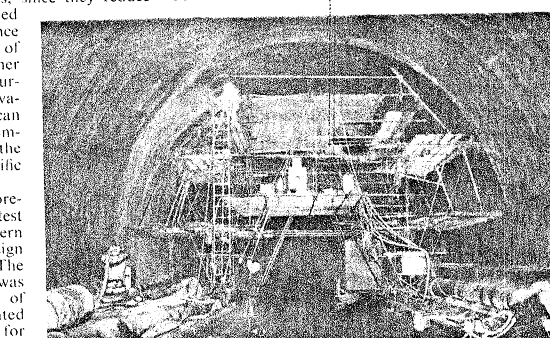
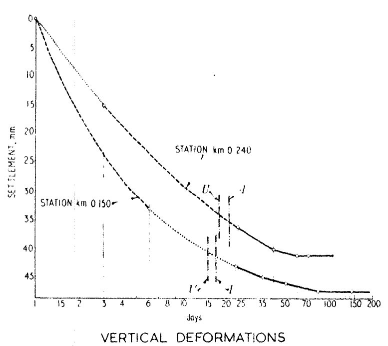
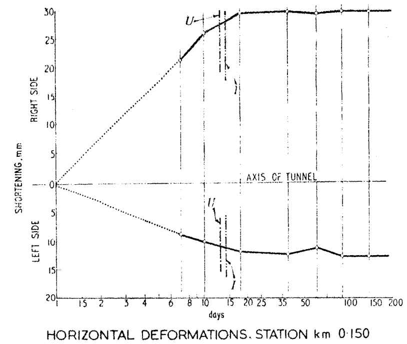
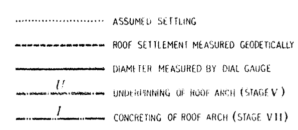


Fig. 25. Placing the polyester insulation, seen completed in the foreground







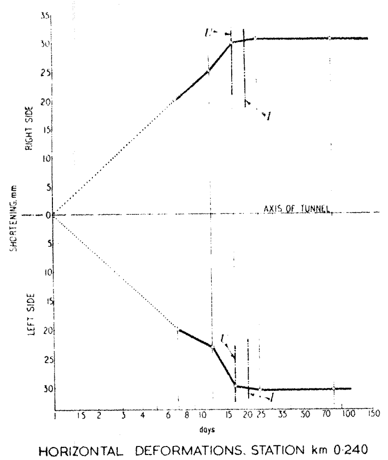


Fig. 26. Curves of horizontal and vertical deformations of the auxiliary lining in respect of time

The Fenner-Talobre formulae were used to compute the forces acting on the full-size tunnel, which was to have a cross-sectional area about 15 times greater than the test tunnel and four times as much overburden. Including a satisfactory factor of safety, the required lining thickness was found to be between one third and one quarter of that assumed for the preliminary design on the basis of experience with customary methods in earlier tunnels in the same region in similar ground. The saving thus made amounts to about 30% of the total cost of the preliminary design.

New Waterproofing Techniques

The auxiliary arch can also be used economically to carry a high-quality waterproof layer. After the rock face has been carefully drained by one of the conventional techniques, such as the Oberhasli method, the auxiliary lining can be sprayed with a film of polyester, which would then be protected by another layer of shotcrete or concrete to prevent the formation of blisters by the water pressure. The fact that insulating agents can now be applied to irregular surfaces by spraying, instead of laboriously bonding bituminous sheets to concrete surfaces which have to be perfectly smooth, is a further important advantage of the new method.

Inner Lining

A secondary lining inside the auxiliary lining may be required for structural or for waterproofing reasons. The first case arises if the auxiliary lining has either been overstressed or if the stresses in it are near the elastic limit, so that it does not possess the desired degree of safety. In the second case the inner lining protects the waterproof layer against water pressure and frost.

If the auxiliary lining has actually been overstressed, the inner lining should be designed according to the following principles. Whereas loosening pressure can be left out of consideration as the active force—provided modern methods have been used—rearrangement pressures and possible squeezing pressures have to be considered, and their magnitude and distribution judged by the results of deformation and stress measurement on the auxiliary lining or additionally by results obtained on a test tunnel. Since rearrangement pressures will mostly act laterally, a lining to suit this type of load should properly be elliptical with the longer axis horizontal, and the lining should be thinner at the sides and heavier at the roof; but if it is desired to suit all possible main pressure directions a circular section with a uniform liner thickness should be adopted.

By far the most frequent mode of failure of such a lining in close contact with the rock face and closed by an invert is shearing along planes which continue in the Mohr planes of the surrounding rock. Model tests have shown that by eliminating the friction between lining and environment the pressure line remains close to the axis everywhere and results in almost equal compressive edge stresses. This effect is largely obtained by spraying a bituminous layer on the inside of the auxiliary arch or by keeping the E value of the waterproofing medium relatively low, and the thickness of the separating layers has to be adjusted according to the irregularities of the surface, taking into account the probable amount of tangential movements. In this way the inner lining can be made considerably thinner and thus more economical; simultaneously its deformability increases. If the auxiliary lining has become stable without exceeding its limit of resistance, the inside lining only serves to increase the factor of safety.

If the auxiliary lining fails, its protective zone becomes larger and the skin resistance required to ensure stability decreases. The advantage thus gained is largely counterbalanced by uncertainty in estimating the magnitude of the external forces. It is consequently always preferable from all points of view to build the auxiliary arch so that stability is attained within the limit of resistance.

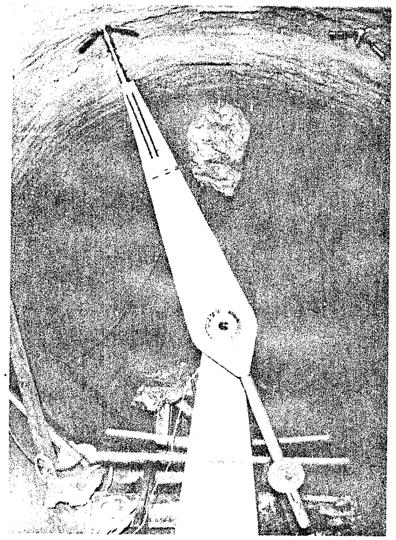


Fig. 27. Deformation measuring apparatus in the Wetzawinkel test gallery

Stability of Linings with Discontinuous Surfaces

One of the conspicuous features of the new method is that the excessive use of shotcrete in overbreak areas is avoided by the fact that the lining approximately follows the irregularities of the rock face. It is often maintained that a lining with a discontinuous surface is a structural impossibility. The error of this opinion has not only been demonstrated in these articles but has clearly been proved in practice (Fig. 30). In sound rock a cavity with discontinuous surface also remains in equilibrium without any reinforcement. Apart from the fact that protruding portions of rock are automatically of better quality, an unstable rock becomes a stable one as a result of the interaction between shotcrete and rock, which transforms the uniaxial state of stress into a triaxial one.

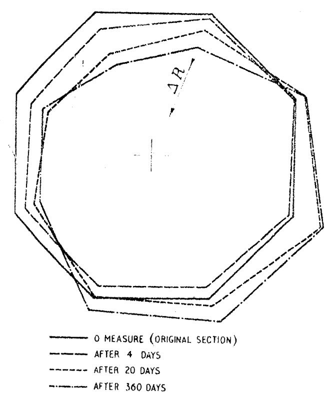


Fig. 28. Deformation of radii with time in test gallery

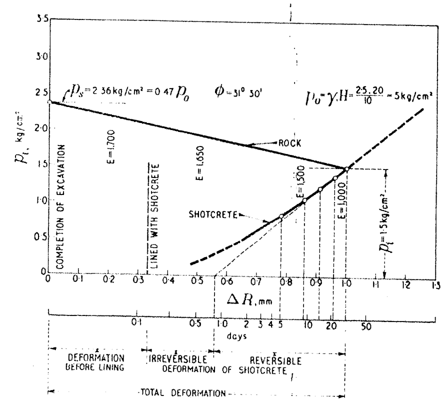


Fig. 29. Determination of skin resistance p_i, after Müller & Pacher

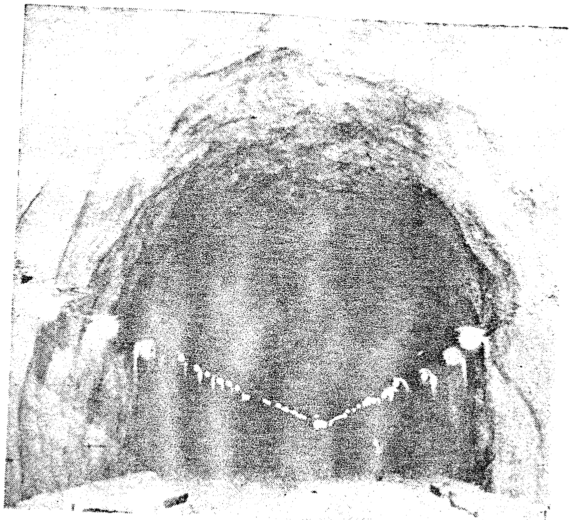


Fig. 30. Permanent shotcrete lining of a super-highway tunnel in Venezuela, following the irregularities of the rock

Final Remarks

Shotcrete is unfortunately an expensive construction material, and when used in the wrong way it may not always show a saving compared with concrete placed in shuttering, particularly when unit shuttering costs can be lowered by re-use. As has been explained, shotcrete is not intended to be used like a conventional concrete lining on a fairly good rock far behind the face, where the rock is already decompressed and sufficiently safeguarded by rockbolting. On the contrary the method aims to reduce anchoring to a minimum by preventing initial superficial loosening as a result of the more effective interaction between shotcrete and rock; the worse the rock, the greater the savings that can be made by the new method.

If shotcreting is not introduced as an integral part of the driving process its advantages will not be fully utilised. Further, the new technique is rather delicate in application, particularly in soft and possibly waterbearing ground. Applying the method correctly needs as much practical knowledge as, say, forepoling in difficult ground, and it requires in addition much closer collaboration with the engineering geologist. Evaluating forces by measurements with respect to time is the very basis of the method and the sole means of economical design in accordance with the actual properties of rock. The design and dimensioning of auxiliary and inner linings and the associated steel reinforcements should therefore be carried out exclusively by engineers who have not only practical tunnelling experience but also an extensive knowledge of rock mechanics. The handling of the purely practical part of the work can be entrusted to experienced foremen especially trained in the method. Disregard of these rules has already caused several serious accidents and failures as well as financial loss by overdimensioning. Dimensioning linings on the basis of experience with tunnels built by obsolete methods is entirely out of fashion and is thus no longer acceptable, new experience having superseded what was done in the past. It is a grave blunder to apply obsolete and unscientific methods to expensive modern shotcrete linings.

This causes irresponsible waste and is liable to discriminate against the new method.

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Fuse Catalogue. G.E.C. (Engineering) Limited, Switchgear Division, Birmingham 6, have issued a new fuse catalogue (SP 910) which supersedes their Technical Description 466. The high rupturing capacity fuse links described in this catalogue are available with current ratings from 5 to 200A at voltages ranging from 3·3kV to 66kV, the highest current ratings being associated with the lower voltages. The publication contains graphs showing pre-arcing time/current characteristics, total operating time/current characteristics and the relationship between prospective and cut-off currents for all the voltage ratings. It also contains an explanation of virtual time and includes a table giving the value of I^2t_v for all the fuses. The catalogue is available from the G.E.C. Publicity Department, Witton, Birmingham 6.