Ground supports for tunnels in weak rocks

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The various factors and how they influence the loading and deformation of supports needed in deep tunnels in weak rocks are presented. Broadly, these are the rock and water conditions and their response to excavation, the excavation and support timings and procedures, and the support details and their workmanship. Particular attention is drawn to the need to consider the truly three-dimensional influence of rock discontinuities on excavation collapse mechanisms with the aid of models and field observation as a guide to support and safety requirements; to consider during construction the positioning and timing of support placement in relation to the rate of advance of the face (another three dimensional effect); to allow a degree of rock yielding by means of yielding supports on the basis of concepts developed from simple yielding rock/support interaction theory combined with construction monitoring; and to consider quantitatively the real load–deformation properties of the structural supports and their interface with the rock. The effects of these variables are illustrated by observations from tunnel projects in hilly and high mountain areas. There is no such thing as *the* load on a support. Proper assessment and control of the many variables which combine to produce a satisfactory and efficient support system cannot be done at the design stage. It is strongly recommended that construction should be aided and guided by monitoring of tunnel performance carried out by engineers relieved of contractual responsibilities as the only positive means of taking sensible, economic and safe decisions during progress of the work. The technical superiority of the observant engineer can cause the rock to work for him.

On analyse les différents facteurs et la façon dont ils influencent le chargement et la déformation des appuis nécessaires dans les tunnels profonds en roc sans consistance. En général, ce sont; les conditions du roc et de l'eau et leurs réactions à l'excavation, les ajustements d'excavation et d'appui et des procédés et les détails d'appuis et leur exécution. Une attention particulière est portée sur la nécessité de considérer la véritable influence à trois dimensions des discontinuités du roc sur les mécanismes d'affaissement d'excavation avec l'aide de modèles et d'observations au chantier comme un guide des conditions requises d'appui et de sûreté; de considérer pendant la construction la mise en position et l'ajustement de placement d'appuis quant à la vitesse d'avancement de la façade (un autre effet à 3 dimensions); de permettre un degré d'affaissement du roc à l'aide d'appuis d'affaissement basés sur des concepts développés à partir d'une simple théorie d'interaction de roc/appui d'affaissement combinée au monitoring de construction; et de considérer quantitativement les vraies propriétés charge-déformation des appuis de structure et leur interaction avec le roc. Les effets de ces variables sont illustrés par des observations de projets de tunnels dans des étendues accidentées et de hautes montagnes. Il n'y a rien de tel que *la* charge sur un appui. Une évaluation et une surveillance exactes des nombreuses variables qui se combinent afin de produire un système d'appui satisfaisant et efficace ne pouvant pas être faites à phase de projet. Il est fortement recommandé que la construction devrait être assistée et conduite par le monitoring de performance de tunnel exécuté par des ingénieurs débarrassés de responsabilités contractuelles, cela étant le seul moyen positif de prendre des décisions raisonnables, économiques et sûres, pendant le cours du travail. La supériorité technique de l'ingénieur observateur peut faire faire travailler le roc pour lui.

INTRODUCTION

The British civil engineering industry has a long experience and a considerable reputation in soft ground tunnelling. In these conditions it is well known that it is necessary to restrict the displacement of the ground during tunnel excavation so as to reduce surface subsidence even where tunnels are deep. This is done by tunnelling fast, using shields, temporarily pressurizing the tunnel, and building permanent segmental tunnel supports close to the face. It is well established

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in saturated soft and stiff, clays and silts that, in tunnels acting as drains, supports eventually carry loads approaching the full overburden, and, where the tunnel is porous and pressurized with water, that they carry the effective overburden. The complete ground loading develops largely irrespective of the displacements that are allowed to occur during construction, because the soil has swelling characteristics, e.g. when a piece is placed in water it swells freely, absorbing water. There are rocks too which have swelling properties and they may be found among the many varieties of Carboniferous shales. These are not rocks in the context of this Paper.

The excavation of a tunnel disturbs the equilibrium of the ground and of the ground water. The stressed condition of the ground changes and so may its strength, but as long as the new stress conditions around the tunnel do not exceed the ground strength at any time, the ground moves into a new position of equilibrium and no structural supports are required to contain the ground. Weak rocks are those which, in tunnelling, require the use of structural supports, either to re-establish equilibrium or to limit rock displacements around the tunnel. The rock itself may be soft or hard.

Weak rock tunnelling requires a very different approach to soft ground tunnelling, though it has some analogies with tunnelling in sand and gravel. Time is ripe for the British tunnelling industry to pay more attention to weak rock tunnelling, because the public demand to make more use of underground space grows steadily throughout the world and industry will be faced more and more with the need to tunnel in weak rock. This Paper is an attempt to stimulate a greater interest in weak rock tunnelling.

Rock is dominated by structural weaknesses of many kinds, commonly called discontinuities. They may be so frequent, as in the wide thrust zones of the Alps and Himalayas, that the rock is a dense pulverized mass, or a highly slickensided mylonite. Such a rock behaves rather like a sand and needs to be confined to develop any strength. There are hard rocks, with much less frequent discontinuities that are open, as in a sandstone or limestone near a Pennine valley side. This rock has both high strength and no strength. Supports are needed solely to prevent naturally loose blocks from falling from the tunnel roof. In between these extreme types there are rocks like shales and siltstones which may be intrinsically weak, yet possess discontinuities with varying degrees of bonding or interlocking.

In rock, even weak rock, unless the tunnel is very close to the surface, the subsidence at the surface is negligible, because large displacements permitted around the tunnel attenuate rapidly outwards. Many tunnels in rock are deep where the existing ground stresses are high and under these conditions even hard rocks are weak.

It would be extremely expensive, possibly impracticable, to provide supports to carry loads comparable with the ground stresses which would be approached if ground displacements near the tunnel were kept small and soft ground tunnelling techniques were followed. To adopt this policy would not make full use of the intrinsic strength of the rock mass or of what is often called 'arching action'. The effect of allowing some displacement of the rock to occur, during excavation, on the loads required to contain it is quite dramatic.

Rocks are basically dense materials and have the property of dilating or bulking strongly as they yield, and this is what happens close to a tunnel in weak rock. By allowing the rock to yield and crush to some extent, its potential strength can be fully mobilized. The high ground stresses close to the tunnel dissipate and the displacements do not extend far because the rock bulks. In this way the ground is encouraged to support itself and only a light support is needed to contain the crushed rock surrounding the tunnel. It is this simple, but important philosophy that will be developed and illustrated in this Paper, from personal tunnel experiences.

Frequent reference will be made to the Kielder Experimental Tunnel in Weardale, County

Durham (Ward *et al.*, 1976). This is a purely experimental tunnel (3.3 m diameter, 100 m deep) built in association with the 32 km long tunnels of the Kielder Water Scheme. The initial stage of its construction was completed during 1974, it was extended last summer and we plan to continue working there until 1980. The experiments consist primarily of a study of a variety of satisfactory and less than satisfactory support systems built in the same rock—a typical Carboniferous shale called the Four Fathom Mudstone. Excavation has been by blasting and by machine and rates of construction have been varied. The tunnel also included an unsupported length where rock collapse was studied. The Northumbrian Water Authority, their consultants, Babbie, Shaw and Morton, the Water Research Centre and DOE Directorate of Water Engineering, as well as the Building Research Establishment, are all involved in this experiment.

COLLAPSE OF TUNNEL EXCAVATIONS IN WEAK ROCKS

The profound importance of discontinuities on ground stability can be witnessed in most rock tunnels during excavation. The best way to study the influence of discontinuities on collapse mechanisms, to understand how to excavate safely and to provide suitable supports, is to excavate a length of tunnel in which a protective shelter is provided and to record in detail the rock displacements and collapses that ensue. This was done at the end of the first phase of construction of the Kielder Experimental Tunnel (Ward *et al.*, 1976).

It has become commonplace in detailed geological surveys for tunnels to record spatial information on the discontinuities and to provide information on their character, roughness, bonding, openness and infilling. Various methods of condensing, presenting and using the spatial information about discontinuities are in use, but most of them do not make full use of the data. For example, hemispherical plots discard spacing and analytical models, which include discontinuities, are usually two-dimensional. Tunnel construction is decidedly a three-dimensional operation and discontinuities certainly need to be considered in three dimensions to understand their influence on the safety and instability of tunnel excavations, and on support needs.

There is no better method of appreciating the significance of collapse experiments or of discontinuity information than by making use of a three-dimensional model. Unfortunately it is difficult to convey such information to the reader on two-dimensional paper. One has to resort to two-dimensional photographs of the model.

It is obviously essential in any model, to have the orientations correct in relation to the direction of the tunnel, but it is perhaps not so obvious that the spacing of all sets of discontinuities should be correct in relation to the size of the tunnel. The ratio of discontinuity spacing to tunnel diameter in the model should be as in nature. Where this ratio is very small, or very large, say outside the range 0.1 to 2, in relation to any set of discontinuities its absolute value is not very critical.

The figures that follow are views of a wooden block model (Ward, 1976) showing the influence of a common pattern of discontinuities in a sedimentary rock on the instability and yielding of a circular bored tunnel. It is a simplified model, but it was built to demonstrate to others the mental picture which the Author acquired from observations made during the construction of the Kielder Experimental Tunnel.

The simplification consists merely in regularizing the spacing and orientation of the discontinuities and omitting most of the very frequent bedding surfaces, and setting the strike directions of the two sets of near-vertical joints at right angles. This was done partly to ease construction of the model, but primarily to demonstrate geometrical principles more clearly. However, the technique may be used quite generally in any discontinuous rock mass.

Most sedimentary rocks (and other types) have at least three dominant sets of discontinuities very approximately orthogonal to each other. First, a set of bedding planes, one of which is the top surface of the model in Fig. 1. This surface is taken to be horizontal as it was at Kielder. There are also two sets of near-vertical joint surfaces. Thus the model consists of a stack of inclined wooden prisms. The front surface is a vertical surface perforated orthogonally by a horizontal bored tunnel. The near-vertical joints intersect the front surface and form two sets of straight inclined traces. There is also one horizontal bedding plane intersecting the front surface just below the crown of the tunnel. The right-hand side of the model is cut away along several near-vertical joint surfaces.

Figure 2 is a view of the model from the right-hand side showing the left wall of the tunnel. The two sets of near-vertical joints intersect the cylindrical surface of the tunnel to form two sets of elliptical traces. It should be noticed that the excavation completely dissects the inclined prisms of rock descending across the central parts of the tunnel, but only cuts partly into them in varying amounts along the side wall. There are three stages of collapse.

Stage one—free collapse of shoulders

If all the discontinuities are plane, unbonded surfaces, conditions are set for the first stage of rock collapse. Unless both sets of near-vertical joints in the roof are wide open, observation shows that rock blocks do not fall out of the crown initially and that collapse is limited by a horizontal bedding plane that is just above or just below the crown as in the model. Sequences of shoulder blocks, marked with spots in Fig. 2, fall down as the tunnel excavation is advanced. Each sequence starts with a small tetrahedral-shaped shoulder block (forming the underside of one of the completely dissected prisms) which is completely free to fall down, see Fig. 3 where the first block of the second sequence has fallen. When this has fallen the adjoining block—a five-sided one—is equally free to fall and successively larger blocks become free to fall from the same row of inclined prisms until a partly dissected prism is encountered at the side wall (see Fig. 4). Figure 5 shows all the blocks that fall out from the left shoulder of the completely excavated tunnel in three similar sequences from three adjacent rows of completely dissected prisms. In reality there may be other lower bedding planes within the tunnel which will allow each shoulder block to fall out in a succession of horizontal slices until the final state in Fig. 5 is attained and the first stage of instability is complete.

Notice that the sequences of falling blocks on the left shoulder follow the direction of excavation into the model and within each sequence the blocks increase in size as excavation proceeds. Similar sequences of blocks also fall from the right shoulder of the tunnel (cut away). These rockfalls are more hazardous because they progress in increasing size *contrary* to the direction of excavation. These features should be recognized when considering safety in construction procedures. For example, in a limited excavation advance before supports are applied it is generally safer to stand in the near left corner or the far right corner of the model.

The model stripped of its free-falling blocks is the potential 'overbreak' which is predictable from the pattern of discontinuities. It is also the immediate shape of the tunnel to expect when excavation is made by blasting, largely irrespective, it seems, of the pattern of explosive charges. Notice that the roof of the tunnel follows a horizontal bedding plane which, in places, can be much wider than the tunnel.

Whether all the shoulder blocks demonstrated in the model fall concurrently with excavation, or are delayed in falling, or do not fall at all, depends in reality on the roughness and bonding at the discontinuity surfaces and on the amount of rock deformation. In any event, tenuously held blocks become loosened readily by the excavation process as well as by general rock deformation and they have little supporting value before falling. Notice also that similar

Fig. 1. Rigid-body model with horizontal bedding and two sets of near-vertical joints. A horizontal tunnel penetrates the front face which is normal to the tunnel axis

Fig. 2. The model viewed from the right, showing elliptical traces of joints and a bedding plane near the crown on the left wall of the tunnel and the three sequences of unstable shoulder blocks

Fig. 3. Initial, unrestrained, tetrahedral-shaped, shoulder block at start of second sequence of unstable shoulder blocks fall down

Fig. 4. Second sequence of shoulder blocks completely fallen. The sequence stops at the first partly dissected prism which forms a prominent shoulder buttress

Fig. 5. All three sequences of shoulder blocks fall. The potential overbreak and completion of the first stage of collapse

Fig. 6. Direct view of the intact left wall of the model tunnel

sequences of loosened blocks occur on either side of the invert and would fall out if gravity was reversed in direction. These too have little supporting value. Thus the initial shape of the excavated tunnel, so far as rock with any significant structural strength is concerned, is anything but circular. It has a horizontal invert and crown, in places is more than a diameter wide, has zig-zag walls, and is expensive to convert into a circular profile.

Experience too has shown that whereas in blasting most of the potentially free shoulder blocks drop out with the blast, they tend to be delayed in falling out when the excavation is made by road-header or full-face machines. Indeed, they are likely to fall out on the machine in awkward situations and present problems in muck handling and to the bearing of thrust pads.

Stage two—crushing collapse of shoulders

The second stage of collapse involves rock crushing and it is necessary to envisage some deformation in the rigid-body model. At regular intervals along either side of the tunnel it will be seen (Fig. 5) that there are near-vertical prisms very nearly dissected by the tunnel excavation and yet retaining a slender column element. These are the prisms immediately beyond the termination of each rock fall sequence of Stage one, where the shoulders stand out like buttresses to the roof. On account of their prominence in the enlarged roof area these prisms will carry a large share of the vertical loading. Moreover, their minimum horizontal area is the smallest of any rock prism, actually at a level which occurs in the particular model just below axis level on the left hand side and just above axis level on the right hand side. Thus the largest vertical stress concentrations arise at those particular positions and rock crushing is more likely there. Slow crushing happened in those positions at Kielder in the unsupported length and in several other lengths (Ward *et al.*, 1976) where supports did not restrain the action adequately. With continuing vertical deformation the prominent shoulder buttresses forming the upper parts of the highly dissected prisms collapse and the sequence of falls started in the first stage of collapse moves a step ahead. This important feature occurs sporadically along a tunnel whenever it runs obliquely to two sets of discontinuities. Successful support systems need to restrain these particular prisms effectively to control further collapse.

When the shoulder buttresses belonging to the weakest, most highly dissected prisms have started to yield, greater stress concentrations arise near axis level at the adjoining, less-dissected prism in the same row and the process of shoulder buttress collapse can continue like the sequence of free-falling blocks did in the first phase of the collapse.

Ultimately, where the rock is sufficiently weak, the intact prisms closest to the original side walls not cut into by the excavation will also yield, and again this will progress in ordered positions along the tunnel. Even if no rock has fallen from the periphery of the tunnel the potentially unstable blocks and highly stressed positions can be identified during safety inspection by means of some simple rules.

Collapse is initiated at 3-sided areas bounded above by a bedding plane and by ellipses, one from each set of joints, such as block 1 in the right-hand sequence of spotted blocks in Fig. 6. Subsequently unstable blocks are all four-sided areas with one pair of opposite sides comprising a bedding plane and an ellipse from one set of joints, and the other pair of opposite sides formed by consecutive ellipses from the other set of joints. These are the remaining four-sided spotted areas in Fig. 6.

The most highly stressed parts of the rock prisms occur where the cusps of ellipses from each set of joints come closest together, as at X in Fig. 6 and this is where the second stage of collapse occurs. These positions adjoin the least stressed prisms which occur where the cusps of ellipses from each set of joints intersect each other by a minimum distance, as at Y in Fig. 6. In reality, of course, the discontinuities are not regularly spaced, their dips may vary to some

Fig. 7. View of the left shoulder before the face in the Four Fathom Mudstone of the Kielder Experimental Tunnel

Fig. 8. View of the side wall in the Four Fathom Limestone of the North Wear drive of the Kielder main tunnel

extent, and they may bifurcate. Nevertheless, these simple rules still hold and the regular model provides basic guidelines for field interpretation.

Figure 7 is a view towards the left shoulder before the face in the highly-laminated Four-Fathom Mudstone of the Kielder experimental tunnel where excavation was by a Dosco road header. Joint traces, one from each set, intersect just below a horizontal bedding plane where a small tetrahedral-shaped block has fallen out during excavation. The light teeth marks of the Dosco machine roughly follow the tunnel circumference.

Figure 8 is a side view at axis level of the Four Fathom Mudstone in the North Wear drive of the Kielder main tunnel (3.5 m diameter). The bedding is essentially horizontal but somewhat wavy. The elliptical cusps of two joint traces, here bonded together with secondary calcite, come close together at axis level and show between them the presence of a highly dissected prism which will be highly stressed. It is a critical place for measuring the vertical rock strain with time and for monitoring possible yielding and safety.

Stage three—roof stoping and Kielder experimental collapse

We will now examine the displacements which took place in the rock surrounding the 8.5 m long unsupported length of the Kielder Experimental Tunnel during its first two stages of collapse before examining the third stage of roof collapse.

The unsupported length was excavated by a Dosco road header machine at the end of the initial drive in the Four Fathom Mudstone (Ward *et al.*, 1976). A protective shelter of steel arches lagged with some bank bars and steel mesh was provided and kept well clear of the rock.

The vertical displacements of the rock above and below the tunnel as the face approached and passed by in the unsupported length, as recorded by a special magnet extensometer inserted from the surface in a 100 m deep borehole, have been reported elsewhere (Ward *et al.*, 1976). In addition five extensometers, EX 1 to 5, were placed at almost the same cross section just after the face had passed by (see lower centre Fig. 9). Each extensometer recorded the rock displacement at 0.3, 1, 2 and 3 m from the original tunnel periphery relative to datums fixed 6 m away.

Some first stage shoulder blocks fell out on the machine during excavation, and the rest came out in the next 2 weeks. The second stage of collapse was completed in about 100 days. Some of the debris was too large to be removed by hand. The roof, which originally followed a bedding plane along the central part of the tunnel, had now considerably widened on either side.

The displacements at each extensometer position are plotted with time for some 750 days in Fig. 9. The displacements, which occurred during the first two stages of collapse, are marked SC (shoulder collapse) on the EX 2 plot and the fallen rock areas are marked SC in the tunnel section. At EX 1 the 0.3 m anchor fell out with the rock after 16 days when the displacement, roughly horizontal, was about 18 mm and there was negligible movement at 1 m and beyond. It was possible, nevertheless, to continue observations at the latter positions for about 300 days when the whole of the remaining shoulder rock debris was removed (marked SR) and seriously bent the whole extensometer, although the anchors at 1 m and beyond remained in position. At 300 days the rock displacements were less than 2 mm at 1 m and 0.1 mm or less at 2 m and 3 m. On the opposite side of the tunnel the 0.3 m anchor of EX 5 fell out with the shoulder block after 100 days when the displacement was about 17 mm, and again the whole outer end of the extensometer was bent when removing the massive debris at 300 days. At this time the lateral displacements at 1 m and beyond were not more than 0.1 mm.

After 100 days, when all the shoulders had fallen, not a single flake of rock had fallen from the original roof which followed a bedding plane. However, it will be seen from Fig. 9 that the roof had sagged and dilated significantly at all levels and certainly more in the centre (17.5

Fig. 11. A view of the cavity after collapse RC1

mm at 0.3 m and 3 mm at 3 m) at EX 3 than on either side at EX 2 and 4. Both sets of near-vertical joints in the central part of the roof had opened noticeably. The first small flake of rock peeled off on a bedding plane adjacent to near-vertical joints in the centre of the roof at 150 days (marked RF on EX 2 plot).

Although some of the shoulder rock debris which had come to rest on the protective steel shelter of the tunnel had been removed soon after it fell, it was not until 300 days (marked SR on EX 2 plot) that the larger masses were completely removed. On account of their bulking the rock debris had clearly been restraining the roof rock to some extent because at all levels the roof displacements suddenly accelerated (see Fig. 9). At 400 days a large flake fell from the centre of the roof (marked RF on EX 2 plot) and at 450 days a major collapse involving the whole roof occurred (marked RC 1 in EX 2 plot). The extent of this third stage collapse is marked in the section as RC 1.

The roof debris from RC 1 collapse was removed at 525 days (marked RR in EX 2 plot) to reveal the extent of the cavity and to allow further collapsing to be followed.

A contoured plan of the cavity at this stage of collapse is given in Fig. 10(a). The elevations (in m above the crown of the shelter) of the several horizontal bedding surfaces that form the roof of the cavity are given, together with the joints that form the various ledges in the sides of the roof and those that define the outer boundaries of the cavity. The outer boundaries of the cavity form a parallelogram. It is defined by the near-vertical joints to which the shoulder blocks fell out in stages 1 and 2 of the collapse and it was this parallelogram which defined the boundaries of the enlarged flat roof before it started to collapse. Each cavity is bounded almost entirely by bedding planes and two sets of joints. The bedding planes that define the several roof levels in the cavities are shown as contoured areas. Each area is marked with its elevation in metres above the original tunnel roof and by a dotted tone which increases in density with elevation. The joints which define the maximum lateral extent of the cavity are shown as solid lines hachured, and those which define ledges within the cavity as solid lines.

Figure 11 is a view of the cavity after collapse RC1 from the lower edge of the plan in Fig.

Fig. 12. A sketch of the cavity compiled from six panoramic photographs after RC1 collapse from the 'viewpoint' of Fig. 10(a)

10(a) showing one of the larger roof blocks about 1 m high resting on the shelter and still to be cleared out. Notice the steel mesh covering the shelter below, the two long ledges above the left wall with EX 2 emerging from the higher ledge and a large block about 1 m high resting on the shelter. The magnet extensometer and its grout pipe descend in front of bent EX 3 which is suspending a block of rock. EX 3 is on the right and is bent.

Figure 12 is a sketch of the cavity compiled from six panoramic photographs taken from the 'viewpoint' marked in Fig. 10(a) after all rock debris was removed. The wall AB bounding the cavity to the right of the viewpoint (see Fig. 10) is outlined by dashed lines. It shows the parallelogrammic shape of the cavity with its axis rotated in plan relative to the axis of the shelter. The extensometers EX 2, 3 and 4 have been reconditioned, and the magnet one omitted. EX 1 and 5 on either side and below the crown of the shelter are bent.

Surprisingly enough the extensometers in the roof survived this major rockfall with little damage, though continuity of observation was lost in many cases. The 1 m anchor on EX 2 was re-anchored a little higher into a side ledge in the new roof and observed by levelling, but continuity was maintained on the 2 and 3 m anchors. Continuity was lost everywhere on EX 3, but it was renovated and observations continued with levelling as checks at all levels. The system at EX 4 could not be renovated and a new anchor at 2 m just in the new roof was followed by levelling.

Further roof collapses, each of quite substantial size and constituting the second major collapse RC 2, took place at intervals between about 760 and 800 days at which time further collapse was prevented by timbering above the shelter to provide a safe condition for the tunnel to be extended.

The second roof collapse and its extent in section is marked RC 2 in Fig. 9. Its contours and extent in plan in relation to the protective shelter are shown in Fig. 10(b). The plan extent of RC 2 follows the same near-vertical joints as RC 1 and notice again that RC 2 produced side ledges on all four walls. Almost certainly these ledges collapse before the high part of the roof collapses. The highest roof in the RC 2 cavity at 3.2 m has reached a sandstone stratum (called the Quarry Hazle) overlying the Four Fathom Mudstone.

Fig. 13. Diagrams of sagging and collapsing slabs of roof rock articulated at joints illustrating some modes of stoping seen in the Kielder Experimental Tunnel

The mechanism of the third stage of collapse, which appears to be synonymous with 'stoping', is not as clear as the earlier stages of collapse because its development was much more difficult and hazardous to inspect. It was evident from visual observation of the opening of near-vertical joints in the central parts of the flat roof that horizontal tensions exist at roof level and for some distance above. It is also apparent from the extensometer observations that vertical extension occurred particularly near the centre of the roof and progressed upwards. On the tunnel section in Fig. 9 the distributions of the vertical extension just before RC 1 collapse of up to 15 mm at EX 3 between 0.3 and 1 m above the roof and of up to 10 mm at EX 4 between 1 m and 2 m above the roof are indicated as thin shaded areas.

Near the centre of a flat roof horizontal and vertical tensions are to be expected from elastic theory (Terzaghi and Richart, 1952), especially when horizontal ground stresses are low compared with the vertical stresses and this is believed to be the case at Kielder. It is a near-valleyside situation and lateral displacements along the whole mudstone tunnel have been shown to be extremely small, even long-term, beyond about 1 m either side of the tunnel.

It appears that the initial sagging of the flat roof arises from the field stresses as excavation takes place and with time is augmented by self weight-stresses. The frequently-bedded mudstone segregates at weak horizons into hinged sagging beams such as ABCDEF in Fig. 13(a), spanning between the limiting near-vertical joint walls AD and CF, bounding the parallelogrammic roof, and with one or more near-vertical joints such as BE dissecting the beam. Thus in Fig. 13(a) the equilibrium of the heavy beam hinging at D, B and F is a simplified model of the real situation which is in fact the equilibrium of a heavy plate articulated in two directions when the unsupported tunnel excavation is of limited length.

When the sag is small, the vertical joint contact areas at D, B and F are small, the horizontal

Fig. 14. View of the unstable face from the front of the model in Fig. 1 with two blocks fallen out below a bedding plane

reactions at those points are finite and the normal contact stresses high. This leads, in a weak rock, to local crushing and splitting along bedding surfaces as indicated by dotted lines about corners D, B and F. As the hinge B descends to a position level with hinges D and F the horizontal reaction increases rapidly to infinity, crushing must continue, and instability becomes inevitable. In reality collapse is likely well before such a large sag develops, indeed RC 1 collapse occurred when the roof sag was not more than 35 mm in a roof span of 4.7 m.

In Fig. 13(b) the joints are inclined at 75° which is typical of many joints in the Kielder rock. The beam depth is the same as in Fig. 13(a), the diagonal distance BF in one half beam is shorter than the other DB, and it is then evident, assuming symmetrical deformation, that collapse will occur at a much smaller sag than in Fig. 13(a).

There are also two other modes of collapse which have been seen at Kielder during excavation and these are indicated in Fig. 13(c) and (d) with thinner slabs, and with joint dips of 45° which sometimes occurred locally. In Fig. 13(c), if the joint at B happens to be slightly less steep than at A or C, the right-hand slab separates first at the centre and cantilevers locally at C with the left-hand slab perched on it for a short time until it breaks away, sometimes leaving a locked-in corner piece at C. Relics of this nature were often seen. On the other hand, especially if the near-vertical joint at A is somewhat less steep than at B or C, as in Fig. 13(d), the whole slab cantilevers temporarily at end C and breaks away leaving a corner fragment.

There is a whole area of more detailed examination and theoretical analysis of instability actions to pursue here which is of considerable interest. Provided the bulked rock is cleared away as it falls and is not allowed to pile up on light supports, the stoping action appears to propagate continuously upwards so long as the rock conditions remain the same. Stopping is after all a well-known mining technique of cheap excavation.

Face instability

The stability of the tunnel face in discontinuous rock is just as important as the stability of

Fig. 15. Distributions of radial displacements ($F = u_t/p_0 r_t/2G_r$) of the circular boundary of a tunnel of unit radius (r_t) in the vicinity of its face normalized in terms of the plane strain displacement ($p_0 r_t/2G_r$) in an isotropic stress field p_0 , for $K_0 = 1$ and 0

the walls and roof in relation to support design as will become evident later. It is all part of the same three-dimensional stability problem and needs to be examined in the rock model. If the face is being cut, say vertical, and any of the discontinuity surfaces have a component of dip towards the face surface, then the face may be unstable. In the model mentioned earlier the face driven in one direction is inherently stable, whereas it is unstable in the other direction. This is illustrated in Fig. 14 which is a portion sliced from the front end of the model in Fig. 1. In the tunnel only the darker circular area of rock can be seen and two unstable wedges of rock have fallen out on joint planes emerging below a bedding plane. The action starts at the left shoulder with the first tetrahedron falling and progresses right across the face. The third wedge is just held in the face, but will be highly stressed in the right bottom corner where crushing could start and cause a further collapse.

Summary

The collapsing modes of an unsupported tunnel excavation in a specific, but common, set of discontinuous rock conditions have been examined, and it suggests immediately ways in which collapses may be prevented and safety maintained. The rock conditions in the mudstone of the Kielder Experimental Tunnel are only moderately weak. Its collapses were distinctly three-dimensional and arose from the rock yielding locally in specific small regions determined entirely by the locations and orientations of the discontinuities in relation to the excavation geometry. Firstly, certain blocks fell from shoulders in a pattern almost without restraint, then in certain ordered places along the sides. Rock crushing ensued and finally rock crushing occurred in small regions in the roof to produce stoping. There is no symmetry of the rock yielding about the tunnel excavation. The shape of the ultimate collapse cavity is dominated completely by the discontinuity pattern.

No numerical modelling of tunnel deformation behaviour has yet been made which assimilates all the complex features of the various stages of collapse seen in the experimental tunnel.

THE ADVANCING FACE EFFECT

We have been considering so far the three-dimensional effects that arise in a fixed tunnel excavation because discontinuities intersect the tunnel axis diagonally. There is yet another three-dimensional effect which arises in the vicinity of the advancing face where the all important processes of excavation and supporting take place.

Elastic theory

We shall neglect the presence of discontinuities, at least for the moment, and illustrate the face effect in a simple way by examining the deformations that occur near the flat end of a horizontal circular tunnel in an infinite, homogeneous, elastic solid. This represents the conditions one would encounter in a massive unjointed rock which does not strictly require support, but, as will be seen, if supports are applied close to the face they become loaded.

Several experimental and numerical solutions to this problem exist (for example Daemen and Fairhurst, 1972; Descoeudres, 1974; Panet, 1976; Hocking, 1976), some are approximate and not in good agreement with each other. The most reliable seems to be the boundary integral equation solution (Hocking, 1976)¹ where the hole was subjected to field stresses $p_z = 1$ and $p_x = p_y = K_0 p_z$ applied at infinity parallel to the coordinate axis with a Poisson's ratio (ν_r) of 0.25 and with values of K_0 of 0, 0.33, 1 and 2, the axis of the hole being the x-axis, and r_t the hole radius.

For the simplest case with $K_0 = 1$, i.e. an isotropic stress field (p_0), the radial displacements (u_r) of the circular boundary are axisymmetric and their distribution relative to the tunnel face (d/r_t) is given by the uppermost curve in Fig. 15.

In the tunnel well back from the face the displacement for $K = 1$ is asymptotic to the plane strain solution of Lamé:

$$u_t = \frac{p_0 r_t}{2G_r}$$

where G_r is the shear modulus of the rock. G_r is equal to $E_r/2(1 + \nu_r)$, where E_r and ν_r are the Young's modulus and Poisson's ratio of the rock respectively. Thus in Fig. 14 the displacements have been normalized in terms of the plane strain value and expressed as a ratio:

$$F = u_t / \frac{p_0 r_t}{2G_r}$$

Thus, in effect, there is in the vicinity of the face for $K_0 = 1$ a fictitious radial support pressure of $(1 - F)p_0$, which is potentially available for release on to any support as the face is advanced.

Figure 15 also gives the displacements of the boundary of the tunnel for $K_0 = 0$. In this case the displacements are symmetrical about the vertical plane through the tunnel axis, being inwards in the crown and invert, and outwards at the spring line. The values converge well back from the face to the plane strain values of $F = 1.5$ in the crown and invert, and $F = -0.5$ at the spring line.

In Fig. 15 it will be noted that the zone of influence of the face extends into the rock beyond the face amounting to a 5% effect 2.25 radii in front of the face. Similarly at about 2.25 radii before the face into the tunnel the displacement converges to within 5% of the plane strain value. Figure 16 demonstrates how a support placed a short distance back from the face

¹ The Author is indebted to Dr E. T. Brown of Imperial College for running Hocking's solution again for the purpose of this Paper.

At any point back from the face the unsupported rock displaces:

$$u_t = (1-F) \frac{p_0 r_t}{2G_r} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

when the face is advanced sufficiently.

If a support is placed at this same point a radial pressure p_s builds up between the rock and the support as the face advances. Under this pressure an annular support, e.g. a concrete or steel ring of thickness t , will experience an inward radial displacement:

$$u_s = \frac{p_s r_t}{K_s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where K_s is a measure of the stiffness of the support ring. Thick ring theory applied to the support gives:

$$K_s = \frac{E_s}{1+\nu_s} \frac{[r_t^2 - (r_t - t)^2]}{[r_t^2(1-2\nu_s) + (r_t - t)^2]} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

where E_s and ν_s are the Young's modulus and Poisson's ratio of the support material, and when t is small compared with r_t this simplifies to:

$$K_s \approx \frac{t E_s}{r_t}$$

But there is interaction between the loaded support and the rock. Under the contact pressure p_s the rock is prevented from displacing by an amount:

$$u_r = \frac{p_s r_t}{2G_r} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Now the total displacement (u_t) which would have occurred if no support had been placed is equal to the sum of the displacement which occurs with the support in place and the displacement which it prevents, thus:

$$u_t = u_s + u_r \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Eliminating the displacements between equations (1), (2), (3) and (4) leads to:

$$\frac{p_s}{p_0} = \frac{K_s(1-F)}{K_s + 2G_r} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

This expression may be used to demonstrate the effects of

- (a) the position of the support placement relative to the face,
- (b) the relative stiffness of rock and support, and
- (c) the thickness of the support relative to the tunnel radius on the support effectiveness and the load it carries.

Certain conclusions of a general qualitative value can be drawn, but considering the highly idealized nature of the assumptions involved they should not be considered of value in design, except in very special circumstances.

- (a) The closer to the face the support is installed and the stiffer it is relative to the rock stiffness, the greater the load it carries.
- (b) When the support is very stiff relative to the rock it will carry a maximum pressure $(1-F)p_0$. Hence the maximum value of p_s/p_0 is about 0.72 for the conditions considered (see Fig. 14).
- (c) The same loading can arise with supports of different stiffnesses (e.g. different thickness) when placed at different distances from the face.

Fig. 17. (a) displacements (u_t) recorded 0.3 m above the crown of the Kielder Experimental Tunnel during instantaneous advances of the face and while the face was stationary: (b) displacement-time plots during the stationary face periods

- (d) If the face is advanced intermittently in short lengths followed by placement of the same supports over each length, which is a common practice, then the loading on the support will vary from higher to lower values along each length in rhythm with the construction procedure.

Results of a similar nature can be calculated for other values of K_0 . In particular for all values of K_0 except $K_0 = 1$, there will be circumferential moments and shear forces in an annular support as well as a thrust. In the case when $K_0 = 0$ the average radial convergence around the tunnel periphery due to face advance is one half the value for $K_0 = 1$. Hence the thrust in a ring not bonded to the rock would be half the $K_0 = 1$ value.

In a weak rock the convergence of the rock as the face is advanced depends not only on face position, but also on time. Thus rock displacement and the loading on a support near the face increases whether the face advances or remains stationary.

The stability of the rock forming the face is also very important, and the directions of discontinuities in relation to the face can be critical in this respect. If the face disintegrates, yielding develops ahead of the face, and a large proportion of the face advance displacement occurs ahead of the face. Support loading is then less sensitive to its position behind the face.

Weak rock displacements and support pressures near Kielder faces

In the Kielder Experimental Tunnel many sets of measurements have been made of rock displacements in the vicinity of faces advancing at different rates with different excavation methods and in somewhat varying discontinuity situations. Figure 17 shows one set of many observations of the downwards displacement of the rock in the crown centre, 0.3 m above the roof, in the Four Fathom Mudstone.

In the lower part of the diagram the vertical displacement (mm) is plotted against the nor-

Fig. 18. Development with time of the mean ring thrust in a steel lining in the Kielder Experimental tunnel

malized distance from the face (d/r_t). It is imagined that the face remains fixed at the left hand edge of the diagram and the rock (or the observation point) moves to the right away from the face. The face advances (about 2–3 m) instantaneously by blasting and then remains stationary for several days. The displacement that occurs while the face is stationary shows as a vertical line in the lower diagram and is plotted with time in each of the small upper diagrams. Thus the effect of face advance can be separated from the displacements that arise from time. It can be seen that most of the face advance effect has disappeared after the first two advances. Of the total measured displacement of about 12 mm only about 4.6 mm is due to face advance and the rest (7.5 mm) is due to rock yielding with time. This is an under-estimate of the absolute total rock displacement because the measurement was started a little way back from the face and after it had been in that position for 20 days. During that time the rock will have yielded significantly and become less stiff, as observations elsewhere in the tunnel have shown. When the face has been stationary only for a short time before another advance is made, the instantaneous displacement close to the face when it is suddenly advanced is much smaller because the rock has yielded less and is stiffer. Observations demonstrate these profound effects of excavation advance rates on rock deformation and how delays reduce the loading on supports dramatically. If a calculation is made on the basis of elastic theory and face position, the dotted curve of roof displacement shown in the lower part of the diagram is obtained. The total displacement is only about 0.6 mm. This was calculated assuming that $K_0 = 0$, $\nu = 0.25$ and $E = 4.8 \text{ GN/m}^2$. The latter value was the lowest of several measurements of the secant modulus at 50% failure stress during *increasing* deviator stress in uniaxial compression taken on intact vertical cores 75 mm diameter during site investigation. We are, of course, concerned with unloading conditions as well as a rock mass that is certainly not intact near the excavation. It is therefore not surprising that the calculated value is an order of magnitude smaller. In fact, if the rock retains its original stiffness and the field stress conditions are known, measurements of rock displacement at one point on the tunnel boundary as the face is advanced is sufficient to determine G_s as Hocking (1976) suggested.

The effect of inserting supports at different positions in relation to the face on support loading was well demonstrated in the Kielder Experimental Tunnel in the length where a circular steel liner (Ward *et al.*, 1976) was used. The steel liner is 12.7 mm thick and was erected in rings, 0.7 m long, each ring comprising four equal segments. The segments and rings were butt-welded together. After the face had been advanced 2.1 m by a Dosco road header

machine, three consecutive rings (1, 2 and 3) were erected with the leading edge of ring 3 spaced 0.3 m back from the face (see Fig. 18). The space (with a mean thickness of 10.8 cm) between the lining and the rock was filled with a special weak grout four days after the above face advance. The circumferential thrusts and moments were measured at eight equally spaced positions around the centre of each ring, i.e. at distances of 0.65, 1.35 and 2.05 m back from the temporary face. During the 3 days after grouting and before the face was again advanced a small thrust developed in each of the three rings, mainly in ring 3 close to the face. When the face was advanced 2.7 m very substantial thrusts developed in each ring, the load in ring 3 being more than twice that in ring 1, with ring 2 carrying an intermediate load. A further advance 7 days later caused only a small further increase in load, but in every ring the load gradually increased with time and this has continued to date.

Figure 18 shows a long-term plot of the mean ring thrust/metre of lining in each of the three rings. It will be noted that the original loads developed in the three rings largely during the first advance of the face have persisted, but slowly increased at practically the same rate in each ring. The mean thrust in ring 3 is equivalent to 30% of the total overburden of 100 m of rock. There is no water pressure on the lining and only minor perched water tables in the beds above. The slightly greater increase in the rate of loading at about 600 days appears to be associated with the driving of the main Kielder Water Tunnel some 17 m below and to one side of the experimental tunnel. The lack of convergence of the long-term thrusts in each ring is important, since it indicates that the rock does not exhibit swelling properties.

Summary

The first advance of a face loads stiff supports markedly even in a strong rock, the magnitude of the load depends on the positioning of the supports relative to the face and the stiffness of the supports in relation to the rock. The loading on supports placed behind a face that is advanced intermittently will be quite variable and in rhythm with the separate advances. An unstable or yielding face will subdue the face advance effect. In a weak rock deformation continues with time, it increases with face standing times. Delays in placing supports after advance reduce support loading, but the loads in placed supports increase with time.

THE DEVELOPMENT OF YIELDING ROCK THEORY

The mining industry, and especially the coal mining industry, has been practising tunnelling in weak yielding rock for a very long time. In early times they proceeded by trial and error, i.e. full-scale modelling, and with sufficient experience in one set of ground conditions they developed efficient support systems for their purposes. They were concerned with producing a maximum amount of mineral at minimum cost, safety was often neglected and little attention was given to long-term stability. Little was quantified, no measurements were made and with such an approach costly errors arose when conditions changed, or a new mine was opened.

In roadways, needed for long periods, maintenance and repair of supports was a regular practice and collapses often occurred. Thus early attempts to quantify support needs considered that it was necessary only to hold up the rock that was seen to fall out when timbers rotted, or were not placed. It was probably not appreciated that where supports needed no maintenance the rock might be continuing to yield slowly and shedding its crushed debris unnoticed through gaps in the lagging. The miners did indeed use yielding supports and allowed lots of rock yielding to occur.

Design methods of the roof fall type were developed further by consideration of possible failure planes, beam action in bedded strata and arching action. Later on attention started to

be given to the application of elastic and then plastic analysis to the deformation of the rock surrounding the tunnel. The complexity of the problem began to be appreciated.

Two approaches to support design are currently popular. One is based on case records of existing practice and attempts to give a simple answer by correlating information about the rock conditions with the supports that were used. Thus with an adequate range of availability of rock conditions and support practice one expects to be able to limit the support requirements for a particular set of conditions. This has led to several rock classification systems designed solely to elucidate support requirements (Bieniawski, 1973; Barton, Lien and Lunde, 1974).

This approach unfortunately perpetuates existing practice, and does not necessarily distinguish conservative work, or even perhaps unsatisfactory practice. It takes no account of the complex rock-support interaction mechanics or the fact that the same supports can be both satisfactory and unsatisfactory in the same rock conditions depending on construction procedures. Additional correlation results provide no further insight into this important problem.

The second approach tries to obtain a more complete knowledge of tunnel support mechanics by means of better continuum and discontinuum analysis of rock behaviour, studies of arching and instability along discontinuities, more comprehensive structural studies of support systems and detailed consideration of the rock support interaction, including the effects of progressive construction. Moreover it believes in continuously gaining knowledge by checking its construction procedures by effective monitoring of rock and support, displacements and strains.

As more and more relevant information on the properties of the rock and the supports, and the construction sequence parameters, are taken into account, closed-form solutions become exceedingly difficult to handle. Nevertheless, within their limitations they provide guidelines and have allowed some general concepts to be developed. Complete detailed solutions of support loading are, in principle, possible with appropriate computerized numerical analyses. But these require a very comprehensive amount of information which still lacks complete identification. The most difficult problems will be to obtain this very large amount of relevant information, either before construction or during construction, and to be able to control the dominating influence of construction procedures and timings in practice.

A simple understanding of the rock-support interaction behaviour in weak rock can be demonstrated by the 'characteristic line' or 'ground reaction curve', which is the pressure-convergence curve for a tunnel in a given rock mass and stress field. Such a curve, when used in association with the pressure-deformation characteristics of tunnel supports, enables, in principle, the load on the supports and their deformation to be determined.

Elasto-plastic tunnel support theory

The 'characteristic line' concept has been developed from considerations of two-dimensional elasto-plastic thick-ring theory applied to a cylindrical tunnel. Terzaghi (1925) suggested that the theory might lead to an evaluation of the plastic load-bearing zone around a tunnel, but did not apply it to the design of tunnel supports. Fenner (1938) did, but got into difficulties. Since then methods have been steadily modified and improved over the years by many authors, e.g. Goguel (1947), Labasse (1949), Kastner (1962 and earlier), Morrison and Coates (1955), Rabcewicz (1963), Pacher (1964), Hobbs (1966), Lecian (1968), Peck (1969), Deere *et al.* (1969), Lombardi (1970), Daemen and Fairhurst (1972), Dessenne and Duffaut (1970), Wagner (1970), Egger (1973), Ladanyi (1974), Daemen (1975), Panet (1976), Atkinson and Potts (1977).

The treatment of the theory by Ladanyi (1974) is chosen mainly for ease of illustration. Prior to tunnelling the rock is assumed to be weightless, homogeneous, isotropic and elastic,

Fig. 19. Plane-strain elasto-plastic rock theory applied to a circular tunnel: (a) notation and distribution of radial (σ_r) and tangential (σ_θ) stresses: (b) displacement notation

Fig. 20. Assumed rock behaviour, after Ladanyi (1974): (a) strength of original rock at failure and of broken rock after failure: (b) stress-strain curve: (c) volume strain

and the rock stress is isotropic with $p_x = p_y = p_z = p_0$. Referring to Fig. 19 we now imagine that a circular tunnel of radius r_{t0} is excavated so that a uniform radial pressure p_t applied at the rock boundary is reduced gradually causing a radial convergence of u_t and reducing the tunnel radius from r_{t0} to r_t . As p_t is reduced the radial stress in the rock σ_r reduces from p_0 and the tangential stress σ_θ increases, until at the tunnel boundary the deviator stress $\sigma_\theta - \sigma_r$ exceeds the strength of the rock. A homogeneous, isotropic, annular zone of broken rock of radius r_b then develops and widens as p_t is reduced further. A displacement of u_b at the outer boundary of the broken zone is associated with a displacement of u_t at the tunnel boundary.

It is assumed that both the short term and long term properties of the rock are known. The former are used to find the instantaneous convergence of the tunnel before the supports are

applied, and the latter for determining the long-term value of the rock pressure and taking into account the deformation of the support.

The assumptions about rock behaviour under stress are indicated in Fig. 20. The rock is assumed to fail according to the Mohr criterion and Ladanyi considered both the Coulomb failure envelope and the Fairhurst (1964) parabolic failure envelope (parameters C_0 and the uniaxial tensile strength T_0). The latter is shown in Fig. 20. In the broken zone or post failure region the rock becomes perfectly plastic following a Mohr–Coulomb failure theory with reduced shearing resistance parameters, cohesion c and angle of shearing resistance ϕ .

Volume strain in the elastic region is governed by Poisson's ratio. While at failure and to some strain beyond failure it is determined by the associated flow rule of plasticity theory. All the rock parameters are assumed to be time dependent, but only their limiting short and long-term values are used in the analysis.

Ladanyi illustrated his analysis by means of a specific tunnel example, the results of which are given in Fig. 21. This is a normalized plotting of p_t/p_0 against tunnel convergence expressed as a percentage of the original radius u_t/r_{t0} and shown by the solid curves of the long and short term characteristic lines. The dashed curves give the corresponding thicknesses of the broken zone expressed as $(r_{b0} - r_{t0})/2 r_{t0}$. In the short term before the support is inserted, the characteristic line is linear elastic down to a critical value of p_t , marked p_{cs} in Fig. 21, when the broken zone starts to appear. The support pressure–convergence lines for a concrete lining of constant modulus and for one that yields at a certain pressure are shown by arrows.

The tunnel convergence amounts to about 1% when p_t/p_0 drops to zero and equilibrium is established before the support is inserted. Equilibrium is always established according to the theory provided the broken rock has some cohesion, which is the case here.

The long-term characteristic line is associated with larger convergence because the long-term parameters are inferior to the short-term ones. The long-term broken zone starts to appear at p_{c1} which is much larger than the short term value p_{cs} . In between the limiting short-term and long-term characteristic lines there is an infinity of similar curves, each corresponding to a given time interval after tunnel excavation.

A tunnel support is placed when p_t/p_0 is zero on the short-term characteristic, the radial pressure it develops after a long interval of time is represented by a point on the long-term characteristic at which the radial displacement of the rock and the support system are equal.

Now the radial displacement of the support system under uniform radial pressure depends on the load–deformation properties of the particular system. These have been discussed by several authors, e.g. Lombardi (1970, 1973) and Daemen (1975). It will be appreciated that any straight line in the plot of Fig. 20 is a linear relation between pressure and displacement, hence a concrete lining which does not creep and has a constant modulus would have a support characteristic which rises steeply from the short-term to the long-term characteristic rock line and mobilizes in this example about 0.09 of the original ground stress as shown in Fig. 21. On the other hand another lining which was initially as stiff as the concrete, but yielded plastically at a certain small pressure would only mobilize that rock pressure. The rock convergence would be larger, but the motion would terminate on the long-term characteristic as shown for the yielding lining in Fig. 21.

Notice how small the support pressures are relative to the original ground pressure and again how much smaller the support pressure is when a yielding or compressible support is used as opposed to a stiff one.

Account is taken of the dilation of the broken zone arising from rupturing of the rock and opening of discontinuities by means of an average plastic dilation making use of the associated flow rule of the theory of plasticity applied only to a limited portion of the post-failure strains.

Fig. 21. Presentation of the characteristic line concept for yielding rock/support pressure and convergence in a particular case, after Ladanyi

Fig. 22. Convergence-time curve for the partially concreted part of the Giri tunnel shown in section in Fig. 23. In other parts of the tunnel similar curves, without the sudden increase at about 220 days, were obtained

The problem here is that in the broken zone the dilation varies from place to place depending on the shear strain and the mean normal stress, and, of course, in reality on the dispositions and orientations of existing discontinuities.

Ladanyi's closed-form solution did not account for gravity forces on the rock mass, though he introduced them as an additional term. Daemen (1975) was able to include gravity forces in the broken zone. Gravity becomes important where the tunnel is shallow, its diameter large, when the long-term rock or large strain properties of the rock become poor and the broken zone becomes large. It leads to an increase in the 'weightless' roof support pressure and an equal and opposite decrease in the invert support pressure.

Daemen (1975) explores in considerable detail the influence of different dilation and strain-dependent post-peak strength decrease characteristics of the rock on the characteristic line. When the large strain residual properties become marked in the broken zone the characteristic line after falling starts to rise and requires larger support pressures, similar to the form of

characteristic line postulated by Pacher (1964) when he described the rising pressure part of the characteristic curve as 'loosening pressure'. The field evidence for such behaviour has not been found by the Author either in the literature or in construction projects in deep tunnels in quite a variety of weak rocks.

It seems that in the zone of dilating rock around a tunnel the idealizations used in the modelling of the broken zone can be unrealistic, that abrupt changes in the rock behaviour at the elastic-broken zone boundary do not occur. Far more needs to be known about the physical behaviour of discontinuous rock masses in sustained unloading conditions near yield.

Yielding in the Giri tunnel

The conditions of a uniform broken zone developing around a tunnel in weak rock are most nearly approached when tunnelling through wide thrust zones, where the rock is already highly discontinuous and sheared in some direction. For example, the recent road tunnels through the Alps, like the Tauern, Arlberg and Gotthard tunnels, and the many Indian hydro-electric projects in the Himalayas.

An extreme example of allowing large and continuous yielding of tunnel supports to take place with consequent continuous reduction in loading comes from one of the latter projects. The headrace tunnel of the Giri hydro-electric project in Himachal Pradesh passes through one of the major thrust zones at a depth of 200–300 m. It was excavated oversize in high slickensided and fragmented phyllites to a diameter of about 5.5 m and lined simultaneously with circular and strutted horseshoe steel ribs of 150 × 150 mm section at 0.5 m centres with pre-cast concrete lagging. A diametrical convergence of some 0.8 to 1.0 m developed in 80 to 120 days before reasonable stability was established, see the first 200 days of the convergence curve in Fig. 22. The steel ribs developed multiple sideways buckles in sympathy with each other along the circular periphery of the tunnel. Bolted joints and welds in the ribs frequently tore apart, as can be seen in Fig. 23. Clearly the rock loading had decreased at least from the critical buckling load of the original ribs to that of the highly buckled and disjointed ribs.

Subsequently, the ribs that were bent to a radius of 600 cm or less were replaced and the disjointed ribs were welded together.

The tunnel was then lined with 30 cm of unreinforced concrete to an internal diameter of 3.66 m, and finally back-grouted. Substantial lengths had been completed and had remained satisfactory for some months when seen in 1977. This is a remarkable example of the use of a yielding lining providing a substantial reduction in support pressure. It also points to the need to develop a support system which can yield in a controlled way up to 20% of its diameter for use in such poor rock conditions.

Such large convergences give rise to a large zone of highly dilated rock around the tunnel, which if not grouted could become unstable when disturbed by shocks or by ground water when the tunnel carries water. An incident in the Giri tunnel illustrates the point. The incident occurred when another part of the tunnel was being driven from two faces which were approaching each other. One face had been stopped, and, after the convergence had substantially ceased with the buckled ribs in place, the unreinforced concrete lining was cast to an internal diameter of 3.66 m but with the invert section omitted and no back grouting. Observations of the invert gap over several weeks showed no closure. But when the approaching face came within about 100 m of the concreted length of tunnel the concrete lining suddenly fractured as shown at about 220 days in Fig. 22 and closure continued as the other tunnel came to within about 50 m and then ceased. Figure 24 shows a cross-section of the tunnel construction before and after the incident and Fig. 25 is a view of the right side at one end after failure. Notice how the rock has folded and flowed like the buckle in the adjoining steel rib.

Fig. 23. View of buckled steelwork (150×150 mm section at 0.5 m centres) in Giri tunnel

This sudden resurgence of tunnel closure in the partly concreted tunnel cannot be associated directly with permanent rock deformations arising at the approaching face since it is much too far away. It is almost certainly associated with the seismic effects of blasting on the highly loosened phyllites surrounding the particularly rigid, but incomplete and weak, concrete ring where the rock had not been grouted at the time.

The experience raises the question as to how much yielding and dilation of the rock should be allowed to take place around a tunnel in order to minimize support loading. Fresh disturbances such as vibrations and changing ground water conditions may cause additional loading unless particular attention is paid to grouting the dilated rock zone and that costs money. It may be more economic in some circumstances to restrict the ground deformation at the expense of higher support loading and less rock grouting.

Summary

The mining industry learned by experience the value of allowing the rock to yield and the use of yielding supports, but their purpose was to win minerals rather than build permanent tunnels. The development of yielding rock theory, which neglects discontinuity and face advance effects, and makes considerable simplifications about rock properties and ground stress conditions, leads to 'characteristic lines' or potential rock pressure-convergence curves. These may be used in conjunction with pressure-deformation characteristics of support systems to estimate support loads and total convergence.

Fig. 24. Cross-section of Giri tunnel supports and partially completed concrete ring showing form of displacement as other face approached 100 to 50 m away

Fig. 25. View of right side end of Giri tunnel supports with invert gap in concrete after failure

Conditions approaching these assumptions in highly discontinuous rock at the Giri tunnel gave stabilization at very low support loads at considerable convergence and drastic support yielding. The example pointed to the need for supports which can yield in a controlled way without wasteful buckling and the hazards that this generates. The example also drew attention to the potential instability of highly dilated rock around sensitive incomplete supports.

Further, it should be appreciated that the short-term characteristic lines become modified on account of face advance effects, and that in reality in discontinuous rock every location around and along the excavation boundary will have a different set of characteristic lines which will create moments and shears in the supports as well as direct loading.

LOAD-DEFORMATION PROPERTIES OF SUPPORT SYSTEMS

The load-deformation properties of support systems are as important as the rock characteristics and the construction procedures in determining support loading. Their properties are more complex than they appear at first sight. There is a great variety of systems in use, most use combinations of materials and components with differing characteristics. None of them is strictly elastic as a whole and details of workmanship can vary the properties. There is a scarcity of quantitative information.

In large tunnels where construction is carried out progressively in benches there is the added complication that the supports have differing properties at each stage of construction. Leaving aside this problem, the main features of some of the common systems can be described qualitatively.

The interface

A distinction can be made between the main structural support and its interface with the rock. The interface involves both the radial fit between the structure and the rock, as well as the degree of bonding or shearing resistance that can be developed in the tangential plane between the support and the rock.

The systems in use seem to fall broadly into two interface groups. In the first, all the intermittent, timber set, steel arch and rib support systems using preformed lagging and timber or rock packing adopted from the mining industry have weak interfaces, highly compressible and non-linear radially, with low shearing resistance and much out of plane loading on the main structural supports. Daemen (1975) has discussed modelling such systems in some detail. As was found in the circular steel rib length of the Kielder Experimental Tunnel (Ward *et al.*, 1976) considerable yielding of the rock can take place before load of any significance develops in the ribs, and, if the lagging is not continuous, loading is also relieved by the discharge of crushed rock debris through the lagging. In the experimental tunnel the rock continues to yield slowly with little increase in support load even after 3 years. The early displacements of the roof rock shown in Fig. 17 are, in fact, taken from this part of the tunnel where the displacements are as unrestrained as they were in the unsupported length. On the other hand, all the continuous lining systems of cast in-situ concrete and sprayed concrete, as well as the preformed liners of steel and concrete which are grouted to the rock, provide a stiff interface both radially and tangentially once any grout has set.

The main structural supports

Coming now to the main structural supports, the most compressible and weakest are the intermittent steel rib systems, when combined with preformed lagging and timber or rock packing. The ribs are particularly prone to buckle sideways, as at Giri, because the shear connections between the ribs themselves and between the ribs and the ground are poor. Sometimes the ribs are made more compressible by using clamped frictional couplings between the rib segments. Attempts to resist convergence where ribs have buckled sideways by adding more ribs or using ribs with deeper webs or thicker flanges are often futile because more rock load may be mobilized and the sideways buckling resistance hardly improved.

When such rib systems are solidly blocked with sprayed concrete between the ribs themselves and between the ribs and the ground the sideways buckling tendency completely disappears. Then, unless the rib spacing is quite small, the properties of the sprayed concrete dominate the support behaviour. Figure 26 is a view of such a support system in the ventilation tunnel of the Katschberg road tunnel in Austria.

In very weak rock conditions such a support can be much too stiff. In Alpine thrust-zone

Fig. 26. View from the portal of the ventilation tunnel to the Katschberg road tunnel on the motorway between the provinces of Salzburg and Karnten in Austria

Fig. 27. Upward view of the roof and one side of the Tauern road tunnel on the Salzburg–Klagenfurt motorway in Austria during construction

Fig. 28. Load-deformation characteristics of some support systems

rock conditions in the Tauern and Arlberg tunnels in Austria, where this system was applied progressively in 1 or 2 benches in tunnels about 10 m diameter, it was found necessary to increase the ring compressibility by introducing clamped frictional couplings between the rib segments combined with several longitudinal slots about 0.5 m wide in the sprayed concrete so as to allow its steel mesh to buckle inwards and reduce the amount of spalling concrete. Even so it was necessary to limit the diametrical convergence (to about 1 m in the Arlberg tunnel) by introducing a high density of grouted rock bolts to improve the shearing resistance of the highly broken rock zone.

Figure 27 is an upward view of the left shoulder and crown of the Tauern road tunnel showing longitudinal slots in the sprayed concrete exposing its steel mesh, running parallel to the ventilation duct and crossing the circular ribs where there are bolted frictional couplings. Up to nine such slots were placed overhead in parts of the tunnel. A further lining of cast concrete was placed later on when the rate of diametrical convergence had reduced to about 1 to 2 mm per month.

Sprayed concrete is often applied close to the face as a primary support and even as a final support. Accelerators are usually added to speed stiffening and retention on the rock, but often at the expense of lower long-term strength. The concrete fits the irregular rock surface rather well and often bonds well to it, except in the invert where debris invariably lies. The concrete is strained continuously by the rock while it is curing, particularly as the face is advanced. Large compressive strains were always measured in the young sprayed concrete length of the Kielder Experimental Tunnel (Ward and Hills, 1977) and their effect on the subsequent properties of the concrete has yet to be determined. Concrete, and particularly sprayed concrete, creeps under load; it also contracts thermally during curing by varying amounts, depending on the type of accelerator used and the rate and thickness of application. Drying shrinkage is not normally large because of the wet conditions.

This is one of the few applications of concrete where it is used structurally during curing. As yet there is insufficient knowledge about its load-deformation properties with time. If continuous sprayed concrete proves too stiff as a support in large diameter tunnels such as the Tauern and Arlberg tunnels constructed with benches, it can certainly be too stiff in small tunnels in similar rock conditions where the whole tunnel periphery is sprayed close to the face.

Continuous thin steel liners and segmented concrete liners are the stiffest support systems, though the latter creep noticeably under load (Ward and Thomas, 1965), and are highly resis-

tant to buckling provided the grouting is complete. They will invariably mobilize high loadings when placed close to the face in weak rock conditions and are not often used for this purpose. The general load–deformation characteristics of the systems described are summarized in a relative way in Fig. 28. Again it will be seen that the systems as a whole fall broadly into two groups. The sprayed concrete system with longitudinal slots will produce load–deformation curves with adjustable low load, high compressibility features similar to the steel ribs with packing, but without the sideways buckling weakness. However, the bonding of the sprayed concrete to the rock between the longitudinal slots could give rise to local compression failures in the concrete. There is clearly scope for the development of systems whose ring compressibility as a whole can be varied uniformly and readily in a controlled and stable way, which can be placed as excavation proceeds and yet function as a finished tunnel surface without the need for additional finishes to suit the particular use of the tunnel.

CONSTRUCTION MONITORING

Construction monitoring is not research, but a simple, essential and only means of knowing whether construction is proceeding satisfactorily or likely to become a hazard to health, safety and further progress. It is just as necessary for the client, engineer and contractor to make sure by constant measurement of tunnel stability that two approaching tunnels will be safe and capable of meeting and that access to their faces can be sustained as it is to ensure, by regular measurement of tunnel direction, that the two faces will coincide eventually.

All those who return consistently and safely from journeys into largely uncharted territory follow the infrangible rule to secure a means of retreat by careful observations on the outward journey. In tunnelling there is always uncharted territory and usually only one escape. Therefore routine monitoring to ensure this escape is secure will yield successful enterprises regularly. Unfortunately engineers on tunnel projects, besides ensuring tunnel direction, seem to be concerned primarily with recording and urging the speed of construction according to the politics of the contract and follow with blinkered eyes the principle that time costs money. This is not an engineering principle as they soon discover when technical problems arise which call a complete halt to progress and take weeks to overcome.

The handsome benefits that could be reaped by employing just one young observant engineer who is not involved in the politics of the contract, but engaged exclusively on real engineering observation, measurement and deduction at the sharp end of construction, have yet to be realized on many projects. Moreover it is just as important to have regular meetings between the client, consultant and contractor about the engineering of the work as it is about progress, because bad engineering leads to lack of progress.

It is evident that there are three broad factors which determine whether supports are satisfactory and economically efficient:

- (a) the rock and water conditions and their response to excavation,
- (b) the excavation, support timings and procedures, and
- (c) the support details and workmanship.

All these factors combine to give an ultimate support response.

While it is absolutely essential to measure the ultimate response, that is, to demonstrate by measurement that the convergence of the supports and of the surrounding rock is decreasing with time or has stabilized, this provides no indication of the efficiency of the construction because a satisfactory support can be achieved in many ways.

In unfamiliar weak rock, one does not learn how to build and support a tunnel efficiently at the design stage. There is a learning period at the beginning of every tunnel and again

every time the ground conditions change. These periods should be marked by intense engineering observation.

Monitoring to aid design and construction decisions

One accepts that information obtained during feasibility studies and pre-contract ground investigation provides some framework of knowledge of the rock and water conditions and their response to excavation, but there is always a lot of unknown territory. First hand and full knowledge of the rock and water conditions and their response to excavation can only be found by monitoring during construction. Construction monitoring is most important in the early stages of the work and, if convenient, should be carried out in access headings or shafts to the tunnel.

For each type of ground, discontinuity patterns should be established and the displacement and detachment of the rock followed in detail as it is excavated. Simple surveys and labelling, combined with the use of a 35 mm stereo camera provide a quick method of recording discontinuities metre by metre. A three-dimensional model should be made to study possible modes of collapse. It should be used to instruct workmen how to recognize hazards and operate safely and to show critical positions for safety monitoring and rock displacement measurement.

Where the ground and water conditions permit, an unsupported excavation, with a protective shelter, should be left for displacement observation. Otherwise in very weak rock conditions circular steel ribs with adjustable frictional couplings between segments, and timber lagging and packing with forepoling, if necessary, should be used as supports. Extensometers should be inserted, generally radially outwards in all directions, also into the face, yet in relation to the rock structure to record the rock displacement out to at least one diameter from the tunnel. They should be inserted close to the face and displacements followed in detail during advances and subsequently with time. Convergence of supporting ribs should be measured and knowledge of the yield force at the couplings should be available.

The location, extent and rates of rock movement will indicate the nature of any instability development, show the need for supports and give a first idea of the yielding to be allowed to reduce support loading to reasonable levels. It will also indicate the need for urgency or otherwise in placing supports.

Routine construction monitoring

Having established a tentative procedure for excavation, support and operation timings, a series of arrays of rock extensometers should be inserted in directions now known to be critical and at the same position in relation to the face as construction proceeds. The diametrical or circumferential strains of the supports, or of critical positions in the rock walls where supports are not used, and of bodily displacements of rock bolts should be measured. Particular attention should be given to strains in supports placed promptly close behind a face that advanced rapidly to that position and remained there for a short time.

Complete details of the timing of face advances and of the insertion of supports for about three tunnel diameters before and after each measurement position should be recorded, together with complete details of the support workmanship and its load-deformation properties, since all these features influence the support performance.

The results of the rock and support displacements and strains must be analysed in relation to the timings of advance and support insertion and the support properties. It will show whether supports are stabilizing, or moving at a slow steady rate and liable to accelerate and collapse long before the event occurs. It will also show what adjustments may be needed to

support characteristics and/or to construction timings, and hence lead to economy and safety in further construction.

As work progresses to a routine stage, a close eye should be kept for changing rock conditions and regular simple measurements of convergence maintained at intervals either on the rock where it is unsupported or on the supports. The use of load or stress measurements for monitoring purposes has not been discussed so far. On their own they are of little value and need to be combined with displacement measurement. For example, the load at the end of a rock bolt or in a support may remain constant or even decrease, yet the support could be collapsing or the rock bolt advancing into the tunnel.

If the simple precepts outlined above are adopted, monitoring will become a prime requirement of the engineering of tunnels in weak rock equally respected by the client, consultant and contractor as part of the contract. It is the only positive means of arriving at sensible, economic and safe decisions in the course of construction. The sooner it is adopted by the British tunnelling industry the sooner it will reap the benefits of efficiency and technical superiority.

CONCLUSION

This Paper does not define precisely how to design supports for tunnels in weak rocks. Its purpose is to draw attention to the many factors which influence support loading and convergence, and how they operate. In this way it is hoped that the Paper will help in anticipating the consequences of a wide range of decisions. Tunnelling in weak rock is much more of an art than soft ground tunnelling, that is why it is important to monitor the performance.

There is no such thing as *the* load on a support in weak rock. It involves not only the rock properties and their discontinuity geometry in relation to the tunnel, but also the details of the particular supports used, as well as the timings, sequences and workmanship of the excavation and construction processes, which are rarely constant in practice and difficult to specify or control.

The consequences of attempting to be conservative by adopting stronger and stiffer supports in deep tunnels, when yielding is seen to develop, can be extremely expensive and often futile. The skilled engineer who has *made* the opportunity to observe the behaviour of the ground can cause it to work for him by controlling its yield and hence its ability to support load.

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VOTE OF THANKS

In proposing a vote of thanks to Dr Ward, Mr David J. Coats of Babbie, Shaw and Morton, Consulting Engineers, said that, as a Scot, he was delighted to be associated with this lecture which commemorated the name of that illustrious Scot, William John MacQuorn Rankine.

Mr Coats commented on Dr Ward's long connexion with, and interest in tunnels of all types and acknowledged the great benefit he had received from association with Dr Ward in connexion with the Kielder Experimental Tunnel in particular. Mr Coats said that this Rankine Lecture had allowed Dr Ward to translate site observations, laboratory experiments and theoretical analyses carried out over many years into a most delightful and helpful presentation. He was sure that the lecture was not only of extreme value in itself but would also stimulate others to further research in the field of tunnelling.

The vote of thanks was accorded with acclamation.