Experiences in the Measurement of Rock Dilation with Three-Depth, Rod-Type, Borehole Extensometers

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SUMMARY. Three-depth rod-type borehole extensometers are used to measure rock dilations in the rock surround of advancing tunnels, commencing within 0.3 m of the face and, initially, giving the dilation caused by stress change and blasting and extending to time effects up to 540 days.

From results obtained and the measured primary rock stresses, a deformation modulus was derived for one project for stress-deformation studies of a large excavation. For another project an alternative approach was used. The excess dilation (dilation due to crack opening and creep) was calculated for the design of a pressure tunnel lining.

1 INTRODUCTION

Borehole extensometers of various types have been used widely in the past for the investigation of civil engineering and mining projects. The three-depth, rod-type, borehole extensometer, (B.E.), developed for the Snowy Mountains Scheme has been employed successfully for more than 10 years for the rock mechanics investigations of several hydro-electric projects in both Australia and Papua New Guinea. During this time, experience has been gained in its use and analysis and today it is one of the most useful tools for rock mechanics investigations.

The B.E. is commonly used to measure the rock dilation around a tunnel, commencing adjacent to the face, as it is advanced by blasting. However, other uses are made of the instrument such as measurements of long-term movements in machine halls and in open cuts. The B.E. is relatively simple in construction, installation and measurement, and is robust and inexpensive. It has the high sensitivity required for small tunnels at moderate depths.

The data obtained from the B.E. provide information on the immediate stress relief and blast-induced movements and later changes with time in the intact rock and on the discontinuities in the rock mass. Depending upon the purpose of the investigation and the method of design, the data are analysed in different ways. The B.E.'s are usually used in conjunction with measurements of the primary rock stress and the elastic modulus of intact rock.

In the hard rocks found in the Snowy Mountains Area and at Ramu 1 Project, Papua New Guinea, a range of values of deformation modulus around the excavations was determined by the B.E. for application to machine hall design, whereas in the Kangaroo Tunnel of the Shoalhaven Scheme, N.S.W., where the rock is a fine-grained silty sandstone with time and moisture dependent dimensional properties, the amount of dilation due to crack opening and creep (including moisture change) was estimated. This information was directly applied to the design of the pressure tunnel lining.

2 REVIEW

The borehole extensometer has developed through many stages. Tensioned and untensioned, grouted and ungrouted rock bolts have been used (Ref. 1). Also sophisticated rod and electronic types have been employed (Ref. 2). The rod-type extensometers have been found to have the best combination of simplicity and accuracy (Ref. 3).

Sellers *et al* (Ref. 3) used rod-type extensometers with anchors, at 1.5 m intervals up to 6 m depth, near an advancing tunnel, to determine the deformation modulus of the rock. Twin span rod-type extensometers with mechanical anchors at 1.5 and 4.5 m depth have been used at the face of an advancing 5 m tunnel to determine deformation modulus and to assist in the design of rock support and lining requirements, (Ref. 4). Multiple depth rod extensometers, with electrical-hydraulic anchors at intervals to 6 m from a 1.8 m × 3.6 m tunnel have been used to determine modulus from readings during tunnel advance (Ref. 5).

Part of the initial movement of the rock is not taken into account by installing the extensometers some 0.6m to 2 m from the face as is done by the above workers, and the use of two-dimensional theory to analyse the data only aggravates this shortcoming. For maximum effectiveness extensometers must possess high immunity to blast effects and fly-rock, be placed close to the face, and have sufficient accuracy to detail the movements occurring in small tunnels at shallow depths. The three-depth rod-type borehole extensometers described by Alexander (Ref. 6) are installed within 0.2 m to 0.3 m of the face and are sufficiently sensitive to detect the movement between successive blasts. When analysed with three-dimensional theory the accuracy of the readings is enhanced. Extensometers of this type are described below.

3 THE BOREHOLE EXTENSOMETER

The three-depth, rod-type, borehole extensometer consists of three unstressed rods of different lengths protected by three copper sleeves, and a three-holed headpiece. One end of the rods and protective sleeves are welded together to anchors to be set at the desired depths. The free ends of the rods pass into the headpiece and terminate about 25 mm short of the datum end (Fig. 1a). The assembly forms a unit which is fully grouted into the drill hole with the datum end exposed. The grouted sleeves and anchors provide maximum support against blast shock and a brass cap protects the head from flyrock.

For measuring movements in the rock around a tunnel or shaft, the B.E.'s are installed in

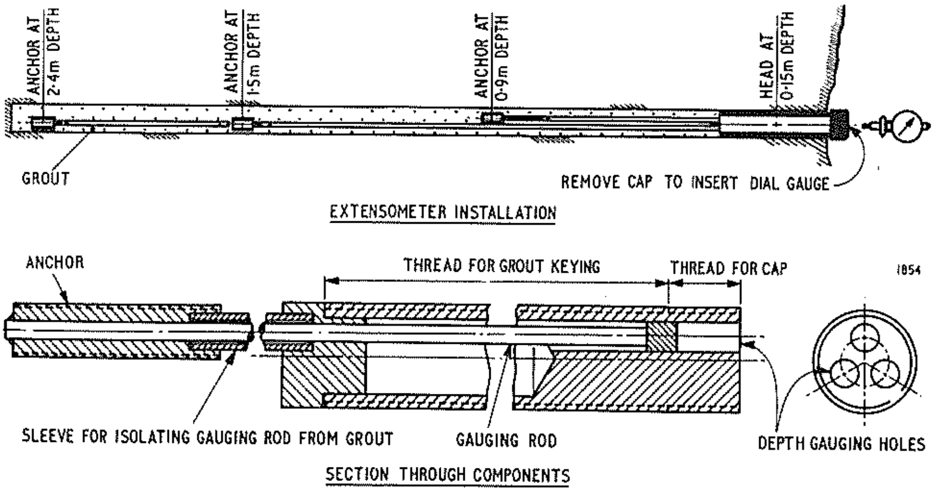


Fig. 1a Borehole extensometer

Percussion drill holes radiating out from the tunnel in a direction normal to the axis of the tunnel. The holes are drilled within 0.3 m of the face in sound rock, not drummy, and in areas where the surface is recessed rather than projecting into the tunnel.

The installations are normally performed at weekends when clear access to the face is available and sufficient time is allowed for the grout to cure. Readings are taken just prior to blasting the face, and between firings in some cases, and before and after the firing of subsequent tunnel advances until the fact is advanced several diameters. Readings are then taken at longer time intervals, up to several months if necessary.

The readings are taken with a dial gauge (Fig. 1b); however a depth micrometer is sometimes used where access is difficult. Both instruments give comparable accuracy of ± 0.01 mm.

The B.E.'s are relatively free from instrumental error caused by blasting and although they are installed within 0.3 m of the tunnel face, it has been found that only about 10 per cent of the installations are lost by blast damage. Possibly because of the unobtrusiveness of the instrument, vandalism is not a problem.

4 THEORY

For the early investigations (Ref. 6) a two-dimensional (2D) theory was used to reduce the data with a nominal factor applied to compensate for the movement that had occurred prior to installation. In the present tests the 2D movements are corrected by a factor determined from a three-dimensional (3D) finite element model of the end of a circular tunnel. The arbitrary loading axisymmetric method of Dunham and Nickell (Ref. 7) is used. For a circular tunnel, in plane strain the following 2D relationship was developed from Reference 8:

$$w = \frac{5Va^2}{2Er}\left[1+\frac{\nu}{5} - \frac{a^2}{r^2}\cdot\frac{(1+\nu)}{5}\right] - \frac{3Ta^2}{2Er}\left[1-\frac{\nu}{3} - \frac{a^2}{r^2}\cdot\frac{(1+\nu)}{3}\right] \quad (1)$$

where:

V,T = principal virgin rock stresses in the plane normal to tunnel alignment and here taken to be vertical and horizontal,

w = radial displacement of a point in the rock (taken to be in the v-direction) towards the centre of the tunnel, from before to after tunnelling,

r = distance of point from centre of tunnel,

E,ν = elastic modulus and Poisson's ratio of rock

a = radius of tunnel.

The equation for general orientation is given by Kruse (Ref. 4). For shapes other than a circle the 2D dilations are computed by studying the behaviour of a 2D finite element model with and without such a hole.

The 3D correction to the 2D solution is obtained from a 3D finite element model of the tunnel, for a circular hole with a flat end, under the actions of the V (vertical), T (horizontal, transverse) and Z (horizontal, longitudinal) virgin stresses. The correction factors (k) in Table I relate extensions between depth intervals at different installation distances x (as a function of tunnel radius a) from the face to extensions from before to after tunnelling.

The factors depend on Poisson's ratio and are given for ν = 0.2. These factors differ from the nominal correction assumed in the past, 75% (Ref.6).

5 DEFORMATION MODULUS METHOD

It is possible to estimate the deformation modulus of the rock around the opening to various depths provided the stress field is known or assumed,

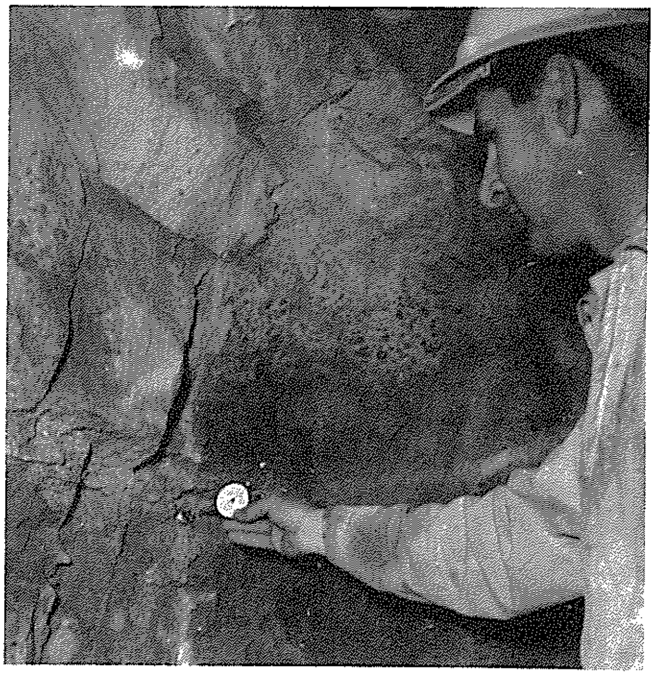


Fig. 1b Extensometer installed

TABLE I

Correction Factor, k (percent)
Anchorage Depths (m) (for a = 1.65m)

Primary stress field ratio	0.15-0.91	0.91-1.52	1.52-2.44	0.15-0.91	0.91-1.52	1.52-2.44	0.15-0.91	0.01-1.52	1.52-2.44
V, T, Z	x = 0.09a			x = 0.13a			x = 0.18a		
1, 1, 1	57	47	68	38	58	64	29	43	57
	x = 0.15a								
	Springline			Sloping 45°			Crown		
1, $\frac{1}{2}$, $\frac{1}{2}$	36	75	66	36	55	63	41	65	66
1, 0.33, 0.33	4	0	0	32	50	57	39	55	57

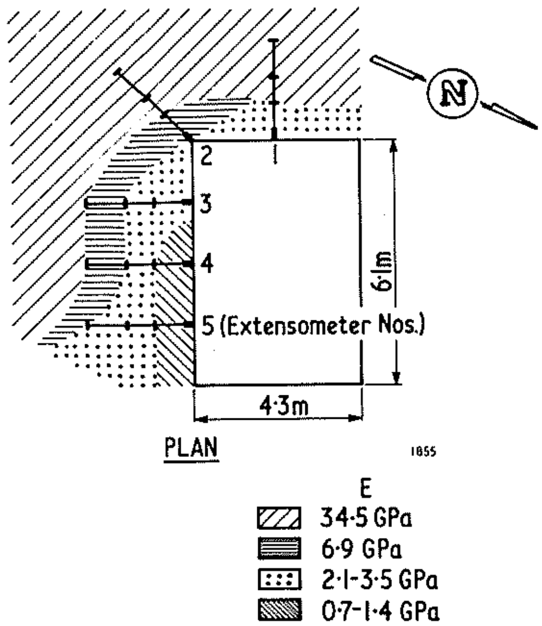


Fig. 2a Observations in access shaft, Ramu No. 1 Project - installation arrangement and pattern of deformation modulus.

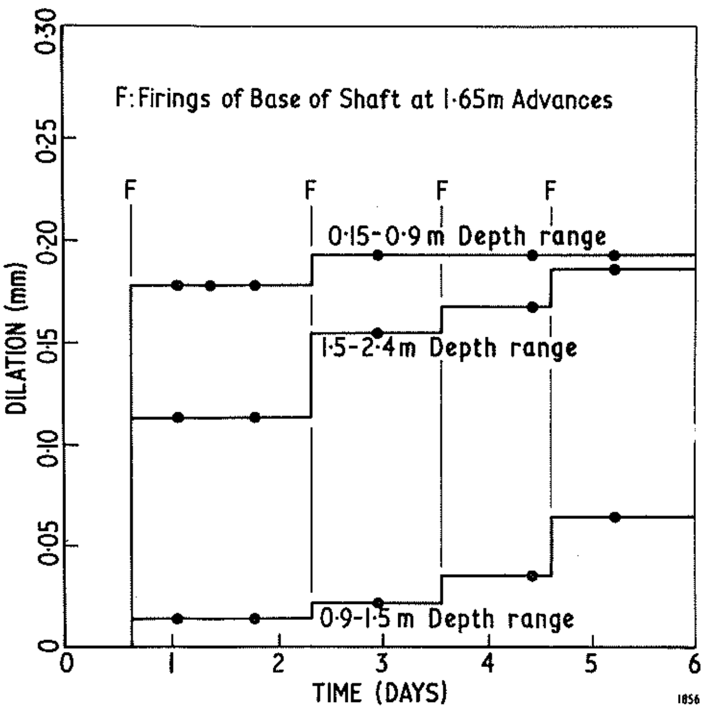


Fig. 2b Observations in access shaft, Ramu No. 1 Project - extensions versus time, extensometer No. 2

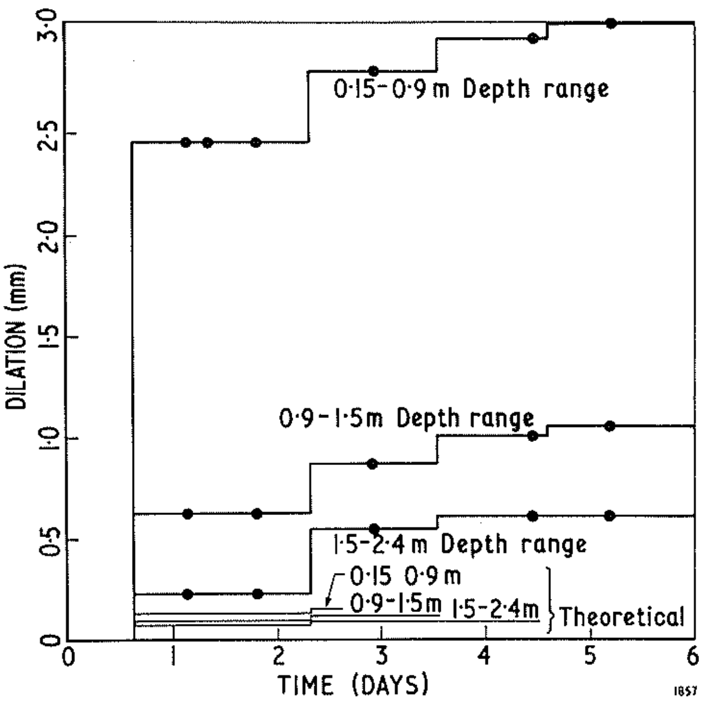


Fig. 2c Observations in access shaft, Ramu No. 1 Project - extensions versus time extensometer No. 4.

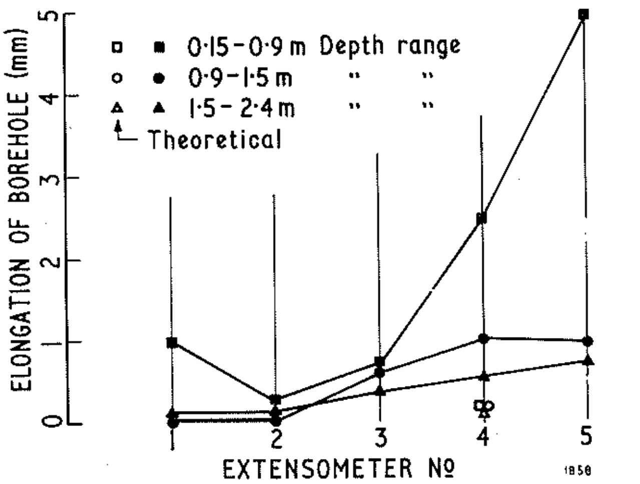


Fig. 2d Observations in access shaft, Ramu No. 1 Project - total extensions, profile around shaft.

and when the dilations measured are assumed to be caused by elastic stress relief. The opening of cracks and the creep movements are thus allowed for by the modulus. The investigations for the Underground Machine Hall of the Ramu I Project, Papua New Guinea included many B.E.'s which were analysed in this manner to provide a range of modulus values for a finite element study of the stability of the machine hall. Here the rock was mostly marble of mediocre to moderately good condition (Ref. 9). Joints were frequent, many with clay seams of a few mm thickness.

Typical plots from an installation in the access shaft (Fig. 2a) are shown in Figs. 2b, c. Deformations occurred as steps at the times of blast, with no change in between, and continued for about five advances, or 7.5 m, instead of being complete after the first, as in the elastic model. Thus blas shock caused an undetermined fraction of the movement at the first advance, and all the later movement. The profiles of measured dilation around the shaft are shown in Fig. 2d. The primary rock stresses were measured close by. It was found that the vertical stress and the north-south horizontal stress were of the order of the overburden stress. The east-west horizontal stress was uncertain. A hydrostatic condition of stress equal to the overburden pressure was used for computation of the theoretical movements for intact rock (Fig. 2c). (Shaft end-effect correction factors were estimated.) The deformation modulus values, found to give the dilations observed at different positions around the shaft, are shown in Fig. 2a.

From this it can be seen that the deformation modulus is an artificial or a 'pseudo-modulus' for the following reasons:

- The deformation is caused jointly by blast shock and stress change, and the deformation under stress change in the absence of blast is not known.
- The deformation of an inelastic body depends on the stress-change path, and not only on initial and final values. During advance of the shaft a stress concentration advances through the surround and can cause movements on discontinuities different from those due to stress change in a body of unchanged shape, to which the term 'deformation modulus' applies.
- The influence of surrounding rock with other values of modulus is not considered in the calculation.

6 EXCESS DILATION METHOD

For calculation of the expansion of the rock surround of a tunnel which is to be lined and subject to water pressure a different approach to the reduction of data is required. The deformation modulus is not considered sufficient to explain the rock behaviour.

On depressurizing a tunnel after long term pressurizing, there is a residual expansion of the rock surround which forms a gap behind the lining. This depends on the closure of cracks in the rock surround and the creep and changes due to moisture change in the rock during the loading (Fig. 3).

The excess dilation is defined as the difference between the observed dilation and the theoretical extension computed for an elastic rock surround having the modulus of intact rock. It is taken as a measure of the opening of cracks and of the creep and changes due to moisture change in the rock.

On long term pressurizing of the tunnel, the rock is considered to offer negligible support until the larger part of the excess dilation is taken up, and thereafter to resist movement with the modulus of intact rock.

The method is equivalent to the use of a deformation modulus which is not constant, as in the deformation modulus method, but becomes less at lower loads and longer loading periods.

Contributions to the gap come from the rock beyond the deptn range measured and from the rock at the immediate surface. The former was taken in relation to elastic movement at a reduced ratio, and the latter was found from plate bearing tests.

This method of analysis was used for the design of the steel lining of the Kangaroo Tunnel of the Shoalhaven Scheme. The Kangaroo Tunnel is located almost entirely in fresh rock of the Lower Member of the Berry Formation. The rock consists predominantly of weak, fine-grained silty sandstone. Fresh rock generally develops surface cracks on exposure to the atmosphere and clay mineral examinations showed the rock to contain a small amount of mixed-layer illite-montmorillonite clay minerals.

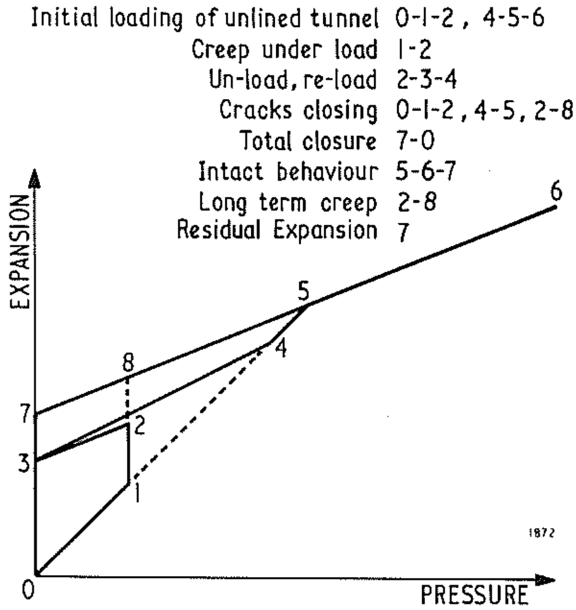
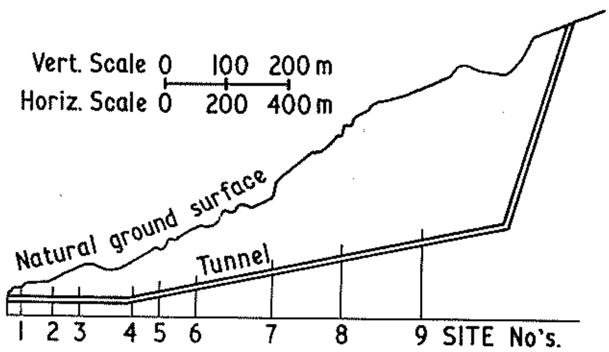


Fig. 3 Assumed effect of expansion of a tunnel under internal pressure and time on closure of circumferential cracks in rock surround



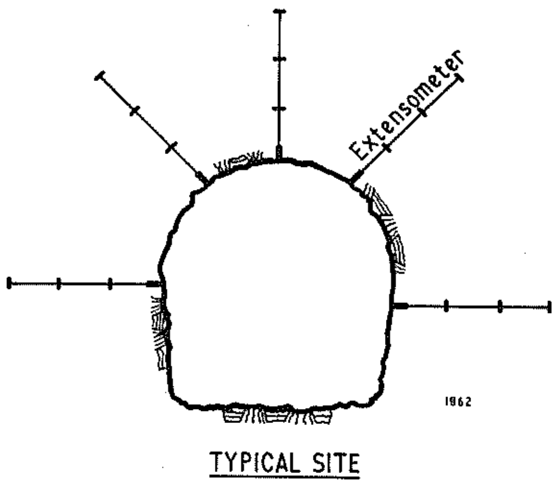


Fig. 4a Observations in Kangaroo Tunnel - arrangement of installation.

Generally, no rock support was required during tunnelling, except at sheared zones, and the rock condition was mostly moderately good to good (Ref. 9). The scheme of installation of extensometers in the tunnel is shown in Fig. 4a.

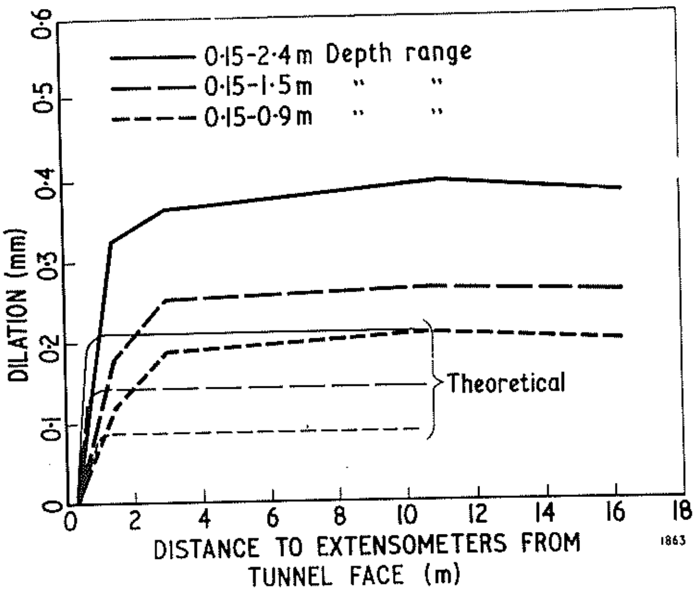


Fig. 4b Observations in Kangaroo Tunnel - extensions vs. tunnel advance, Sta KT5.

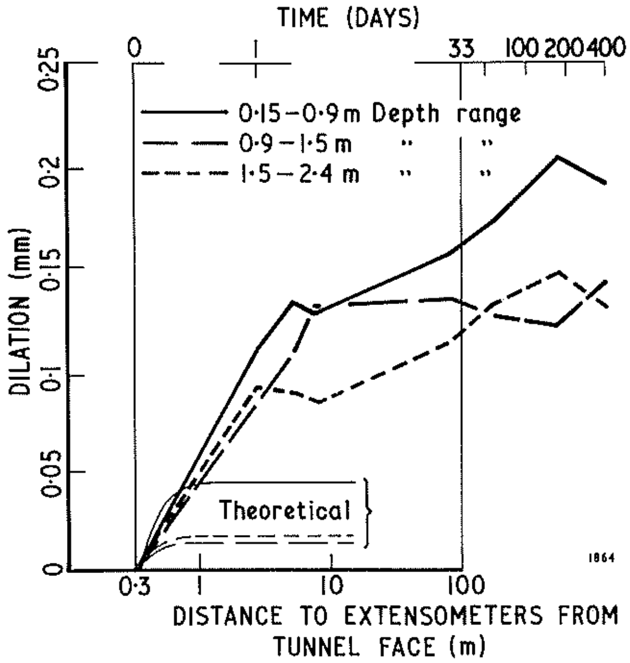


Fig. 4c Observations in Kangaroo Tunnel - extensions vs. log advance, Sta KT6

The rock stress and modulus values were measured in a test adit off the Kangaroo Tunnel. The rock stresses were measured by the United States Bureau of Mines borehole deformation method and by the flat jack method. The ratio of virgin vertical to horizontal stress normal to the tunnel alignment was found to be 0.5 and the primary vertical stress approximated the overburden pressure. The modulus of the rock was measured in several ways: flat jacks, biaxial tests on hollow cores from the borehole stress tests, uniaxial tests on cylindrical cores, and plate bearing tests were used.

The short term deformation modulus of intact rock and of compact rock (all cracks closed) was fairly uniform at 6.9 GPa (1 x 10^6lbs per sq. in.). However, marked time and moisture dependent properties were observed.

From the rock stresses and short term modulus values the theoretical dilations were computed with the aid of equation (1) and the correction factors given in Table I above.

Typical observed dilations are shown in Fig. 4 b, c, along with theoretical intact movements for the particular sites given.

The long-term observations (up to 540 days) were plotted on semi-log paper. The movements were found to be near linear with the log of time (Fig. 4c). The final values of the observed dilations developed up to the latest time of readings, (100 to 540 days) were plotted for all sites versus vertical overburden pressure. The results for the horizontal extensometers are shown in Fig. 5.

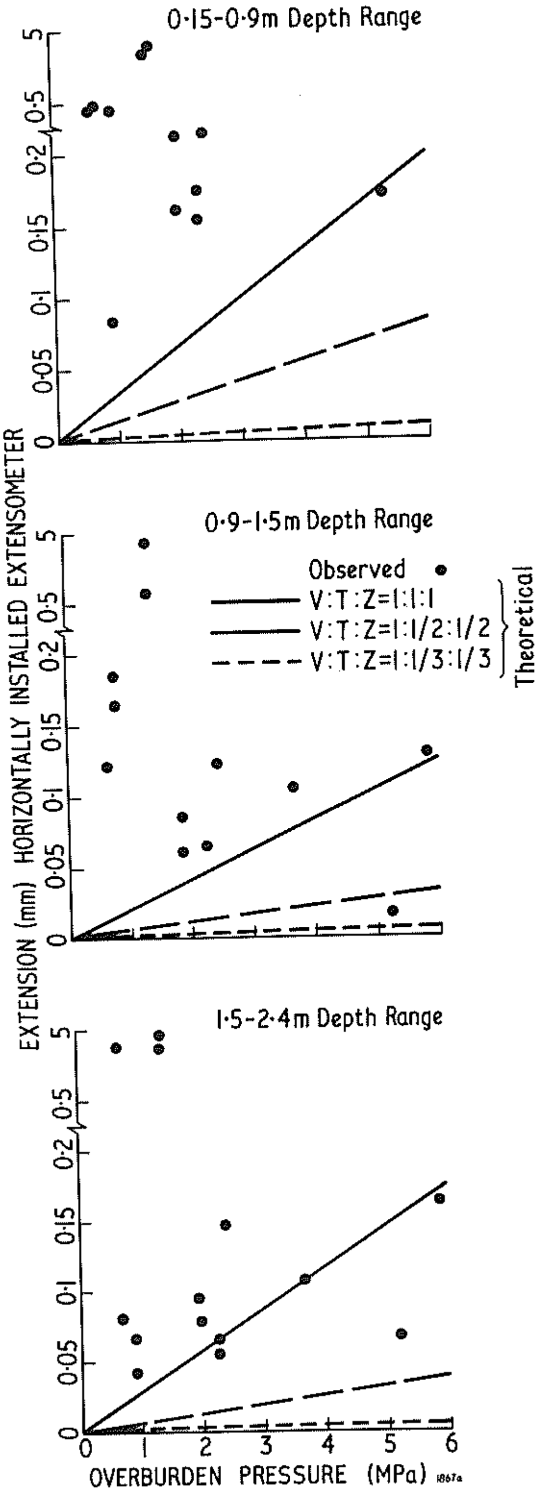


Fig. 5 Observed extensions vs. theoretical overburden pressure, Kangaroo Tunnel

It was apparent that the dilation was independent of orientation around the tunnel, and of overburden stress.

Hence the excess dilation generally diminished as the overburden increased. The theoretical dilation was a rough lower limit to the observations. This limit appears independent of virgin stress ratio in the first two depth intervals, but from 1.52 to 2.44 m appears to fit a ratio of ½.

The time changes indicated a very small in situ strain change compared with swelling and chrinkage strains observed in laboratory tests. Shrinkage (viz. negative dilation, or increase in tunnel radius) occurred only on a few sites. It seems probable that changes of tunnel radius due to drying were inhibited in the stressed rock surround by the development of an opposing stress state. Thus, no further general tendency for change in radius is expected when rock moisture content is restored after the tunnel is lined.

7 CONCLUSIONS

Appreciable movement often develops on the second and later advances of the tunnel due to blast alone, when theoretical elastic movement is complete. Hence the deformation modulus values depend partly on the blast conditions. The values obtained give an indication of rock conditions and their range and pattern may be considered in design studies of stress-deformation conditions around proposed larger excavations by the finite element method. However different conditions of blasting and stress change call for caution, as deformations due to stress change alone have not been measured.

It has been shown that the B.E. together with measurements of virgin rock stress can provide data on the crack openings in the deeper rock. These were used to calculate the equivalent gap for the design of the steel lining of a pressure tunnel. The equivalent gap was used with the intact modulus to calculate the long-term expansion of the tunnel under pressure.

The rock type and the specific use of the data determine the method of analysis required.

In addition to the above applications, the extensometers can give valuable information relating to

- the depth to which rock is disturbed and offers reduced support
- the support offered by rock bolt patterns with different length and spacing of bolts
- the observations of time effects over long periods in underground works and open cuts.

It did not appear possible to derive information on rock stress magnitude or direction from the extensometer data, in the tunnels at moderate depths investigated. Additional data for rock at the immediate surface and from deeper extensometers are required.

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