LONG TERM MEASUREMENTS IN UNDERGROUND OPENINGS AND THEIR INTERPRETATION WITH SPECIAL CONSIDERATION TO THE RHEOLOGICAL BEHAVIOUR OF THE ROCK

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1. INTRODUCTION

The first investigations involving the structural behaviour in underground openings have been based on an elastic behaviour of the rock during the excavation of the underground opening. This assumption is, in terms of history, quite easy to understand.

Even if this hypothesis may exceptionally be valid for the whole of the rock mass, these special cases are of little practical interest, due to the fact that the deformations are very small and cause no problems for the construction work nor for the stability.

Maillart, Fenner and other authors soon realised that in most instances the limit of elasticity or even the ultimate strength of the rock surrounding the opening would be reached or even exceeded.

Kastner defined these zones in which this occurs under certain assumptions but could not analyse the rearrangement of the stresses involved.

For the case of circular symmetry Talobre, among others, developed an elasto-plastic solution, which can be used under the limiting conditions made.

Thanks to electronic data processing it is now possible to compute elasto-plastic calculations of rock openings, taking into consideration every conceivable shape of the opening, the rocks dead weight, other forces, and, if wanted, the heterogeneous and anisotropic properties of the rock.

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The finite element method is the one most commonly used for calculations, although it may not necessarily be the most efficient nor ideally suited method for this problem. In any case there are many methods of calculations and a corresponding amount of publications in this subject area.

Elastic or elastoplastic investigations ignore the influence of all long-term effects and assume an instantaneous change from the natural to the secondary stress conditions at the moment the opening has been formed. Simple observations in tunnel construction show that this case happens very rarely. In fact, deformation takes place for a certain duration of time after the tunnel excavation is completed. Investigations were carried out years ago in an attempt to compute these long term effects and also to understand the rheological behaviour of the rock masses; for example Blend, Langer, Aiyer.

All these investigations were based on two-dimensional deformation-conditions and consequently neglected the three-dimensional stress-distribution which unquestionably occurs close to the tunnel face. The importance of this spacial stress-distribution is undoubtedly considerable and only by taking into account this phenomenon can one establish the relationship between the origin of the deformations of the rock and the same for the lining of the tunnel. Only in that way the indeterminate problem can be solved.

It is well worth repeating that the two most important problems in the theory of underground openings are those related to
- the spacial stress-distribution near the tunnel face, and
- the rheological behaviour of the rock

It is necessary at this place to remind the basic axiom which states, that:

"All the stresses in the lining or in the supports are induced and can be induced only by the movements of the surrounding rock mass which occur after these linings or supports are placed and begin to be statically effective".

Besides artificial influences and external forces, these secondary movements are caused by two factors

- continuation of excavation at the tunnel face, that is the progressive elimination of the rock nucleus in front of the face (three-dimensional state of stress), and

- changes of stress due to rheological behaviour of the rock around the opening (long term effects).
These two factors have independent time characteristics, which are influenced by
- the velocity of the excavation and
- the viscosity parameters

Depending on the relative length of the two characteristic times, they have to be considered either together or separately. The results of an investigation on this question are shown in Table 1. (See Paper 13)

<table>
<thead>
<tr>
<th>RELAXATION TIME</th>
<th>EXCAVATION SPEED</th>
<th>RHEOLOGICAL COEFFICIENT ( k = d/(T \cdot v) )</th>
<th>INFLUENCE</th>
<th>COMPUTATION METHODS</th>
</tr>
</thead>
<tbody>
<tr>
<td>large</td>
<td>high</td>
<td>( k &lt; 0.02 )</td>
<td>no</td>
<td>FACE+RHEOLOGY</td>
</tr>
<tr>
<td>short</td>
<td>low</td>
<td>( 0.02 \leq k &lt; 0.10 )</td>
<td>low</td>
<td>3-D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( k \geq 0.10 )</td>
<td>important</td>
<td>3-D</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( k \geq 0.2 \div 0.5 )</td>
<td>included in plasticity law</td>
<td>3-D</td>
</tr>
</tbody>
</table>

2.77

Figure 1. Ratio "relaxation time of the rock mass to excavation speed" and its influence on the rheological behaviour of the rock in the zone of the face. (\( d = \) Tunnel diameter, \( D = \) dimensional)

In summary, the importance of the long term effects in the rock mass must be underlined as well as the one of long term measurements in the underground cavity, which make it possible to give a correct interpretation of these phenomena.

2. THEORETICAL BASIC CONDITIONS

As in all fields of activity, measurements in underground works make sense only if an appropriate theoretical understanding exists about the phenomena to be studied.

The results of the measurements may show whether the applied theoretical principles are correct or not.

In our case, it is necessary from the beginning to choose rheological laws for the rock. The subsequent measurements may confirm the validity of those laws or may make it necessary to change them.
Due to the rather complex mechanical and rheological properties of the rock, the rheological model can only be approximated. An exact solution is not to be expected. It is usual to represent those laws with a model. For that purpose the following elements may be used together or separately:

- the element of elasticity (Hooke)
- the element of friction (St. Venant)
- the element of viscosity (Newton)
- the element of brittleness (failure)

The most important properties of these elements may be seen in Figure 2.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Name</th>
<th>Stress-Strain</th>
<th>Stress-Strain change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic</td>
<td>E</td>
<td>![Hooke Symbol]</td>
<td>Hooke</td>
<td>![Graph]</td>
<td>⚪️ = 0</td>
</tr>
<tr>
<td>Plastic</td>
<td>C</td>
<td>![St. Venant Symbol]</td>
<td>St. Venant</td>
<td>![Graph]</td>
<td>⚪️ = 0, ⚪️ = ∞</td>
</tr>
<tr>
<td>Viscous</td>
<td>η</td>
<td>![Newton Symbol]</td>
<td>Newton</td>
<td>![Graph]</td>
<td>⚪️ = 0, ⚪️ = ∞</td>
</tr>
<tr>
<td>Brittle</td>
<td>C_R</td>
<td>![Failure Symbol]</td>
<td>Failure (Rupture)</td>
<td>![Graph]</td>
<td>⚪️ = 0</td>
</tr>
</tbody>
</table>

Figure 2. The elements of the rock mass behaviour.

Using these simple elements it is possible to build up a large number of useful models. There is, however, a danger of making the models complicated, difficult to analyse, and often they may give only an apparent accuracy. For brittle rocks where friction and viscosity are the most important properties, the Bingham model can be a very good point of beginning (figure 3a). This model may be complemented with a failure-element to simulate a strength peak as show in figure 3b.

If a Hooke's element is introduced, an elasto-plastic-viscous behaviour of the rock mass can be represented, with or without
a strength peak, as in figure 3c.

<table>
<thead>
<tr>
<th>Model</th>
<th>Name</th>
<th>Symbol</th>
<th>Stress-Strain change</th>
</tr>
</thead>
<tbody>
<tr>
<td>a)</td>
<td>Bingham</td>
<td>$\equiv B$</td>
<td></td>
</tr>
<tr>
<td>b)</td>
<td>Bingham with failure</td>
<td>$\equiv BR$</td>
<td></td>
</tr>
<tr>
<td>c)</td>
<td>B-H</td>
<td>$BR-H$</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3. Simple models for the rock masses.

In order to improve the representation of rheological laws of the rock it is recommended to chain two Bingham models and an element of Hooke. With this arrangement it appears possible to simulate the progressive failure of a rock mass. Depending on the case, brittleness elements may or may not be used (fig. 4). It now shows that the most important numerical parameters for such models are the following:

- $E$ = module of elasticity
- $C$ = cohesion or shear strength
- $\eta$ = viscosity parameter

Obviously the models may be used only as an illustration, because in actual fact we are dealing with a three-dimensional body and not with a uniaxial stress pattern. On the other hand we usually are not dealing with theoretical plasticity, but frictional laws, similar to that of Coulomb.

These conditions may be shown in a simple way by means of the stress diagram using the Mohr's circle as seen in figure 4.
The two Bingham's models represent two limits which subdivide the stress plane into three fields corresponding to the equilibrium, the creep, and the failure. The relation between the two viscosity parameters must be chosen in order that the second one will allow faster failure movements than the first one will allow creep movements.

Actually the model is built up in such a way, that when the circle of Mohr is in contact with the second limit, the speed of the shear deformation is a constant one. This speed will decrease linearly with the radius of the circle of Mohr will disappear when the circle is just tangent to the first limit. On the opposite side of the second limit the speed will increase rapidly.

This model has been used for the following investigation. As we do not deal with a body with ideal plastic properties, we must consider changes in volume, that is some kind of dilatation.
3. METHODS OF CALCULATION

There are various methods which may be adopted when dealing with such rheological laws of the rock in order to solve the problem of the tunnel. In our investigation we have set up and used a method based on the so-called non-compatible elasticity theory.

Basically, it deals with the solution of the elasticity theory equations considering a disturbance vector which refers to the sum of the non-elastic distortions at the particular point, which occurred up to the moment considered.

At each time interval the expansion increments, which correspond to the viscosity and failure conditions of the rock, are computed as functions of the actual stress distribution and added to the previous increments in every component.

The equations of elasticity, including the compatibility equation, are solved taking into consideration the non-elastic deformations; this results in a new state of equilibrium.

A general computer program has been set up for the solution of the case of central symmetry. For simplification we consider first only the case of plane strain.

We will consider later the influence of the tunnel face.

4. EXAMPLE OF APPLICATION

There was an opportunity to verify this method of calculation on the basis of extended measurements in the field.

The northern section of the Gotthard highway tunnel had to be driven through a difficult area known as the Ursoren-Garvera or the Mesozoikum zone.

The construction and the measurements are described elsewhere.

The tunnel was built by means of the German Method i.e. successively driving the side adits, the arch, the bench and the invert. In the following only the arch drive will be described.

The upper half was excavated using a blade shield, which was supported by previously constructed concrete side walls. Steel ribs were erected behind the 4 meter long driving shield. Latter liner-plates were placed on the ribs and were fixed by timber wedges (figure 5.). Behind the liner-plates there was the usual annular overcut.

The rock in this area consists of a series of thin vertical strata of soft clayey-shale. Some moisture has been found. Investigations on the rock strength with shear, compressive and triaxial tests were carried out and enabled good average
results to be established for long term loads

- angle of friction \( \phi = 28^\circ \)
- residual shear strength \( C = 4 \text{ t/m}^2 \)
- modulus of deformation:
  - in undisturbed rock \( E = 1,000,000 \text{ t/m}^2 \)
  - in the fracture zone \( E = 500,000 \text{ t/m}^2 \)

Figure 5. Longitudinal section of the tunnel.

On the other hand peak shear strength values of \( C_s = 25 \text{ t/m}^2 \)
were measured. In the laboratory tests it was impossible to obtain values for the parameter of viscosity of the rock mass. It was intended to obtain this very important value in an indirect way by computations, using the results of measurements in the tunnel.
In order to gain the results, a sample cross-section was selected and fitted with measuring devices (Section a-a, figure 5.). These devices were fitted on two adjacent ribs. Hydraulic jacks were mounted to the base of each arch giving direct results of the arch reactions and on the total load on the arches (figure 6.). The deformations of the arch were measured at 9 lengths as movements of convergence.

Figure 6. Detail of the instrumented ribs.

An adequate adjustment of the results of movements and reactions of the arch allowed the determination of a very good distribution of the rock pressures on the arch and its changes with time.

The largest displacement in the surrounding rock is caused by the closing of the annular overcut and not from the deflexion of the arch itself. Short rock bolts (0.5 - 1.0 m) were therefore fixed around the perimeter of the arch. (Figure 7.) The movement between the anchor head and the steel arch was measured and found to correspond to the closure of the overcut and to be approximately 20 cm. 25% of this movement, correspon-
ding roughly to the thickness of the shield plates, happened without causing any pressure on the arch.

\[ a, b = \text{Measured distances} \]

**Figure 7.** Detail of the measurement rock bolts.

Afterwards the rock pressure increased as a linear function of the closing of the overbreak.

The stiffness was approximately 0.857 t/m² cm. (Figure 8.) As an additional check the stresses in the steel arch were computed by means of strain gauges. The measuring devices proved satisfactory, giving good and quite reliable results of the deformation and rock pressures in this tunnel section.

Figure 8. shows by means of the characteristic lines the equilibrium conditions of the cavity which are reached when the rock formation has obtained its long term condition. The line (1) refers to the cavity and has been computed with the characteristic values which correspond to the lower fringe, as shown
in figure 4. The point C1 defines the equilibrium at the tunnel face where the radial deformation has already reached 7.4 cm. An additional deformation of 6.8 cm occurs from the tunnel face to the section a-a, as shown in figure 5.

The support is set in this moment. The origin of the related characteristic line (2) therefore passes through point C2. This is as well the starting point of the calculation for the long term behaviour of the cavity. This line, (as shown in figure 8b.) has been deduced from the measured movements of the rock bolts.

The deformation of the arches is negligible in comparison with the decreasing of the void beyond the arch.

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**Figure 8.** Characteristic lines of cavity and supports.

The intersection point C3 of the two lines gives the final equilibrium state, which is obtained with a rock pressure of 12 t/m² and an average radial deformation of rock of approximately 33 cm.

The elasto-plastic-viscous deformations and the rearrangement of the stresses within the rock mass have been obtained by the previously mentioned method over a period of 125 days or 3000 hours. The results of the calculations are shown in figure 9. After 84 hours the deformation measured was 5 cm so that the linings began to be structurally effective. From this moment
the pressure increased to 12 t/m$^2$ and the radial deformation increased to a final value of 20.3 cm. Previous to the beginning of the calculation, a deformation of 14.2 cm has already occurred; thus making the final deformation about 34 cm.

Figure 9. Results of the computation - Deformations and rock pressure versus time.

In figure 9, the deformation, if the tunnel lining had been omitted, is also shown. The influence of the lining is clearly visible.

In figure 10., 11. both the results of the measurements and of the calculations for the pressure developments, as well as for the deformations are shown.

By choosing the ideal parameter for the viscosity the results of the measurements and those of the calculations were adapted to one another. The parameter of viscosity of the second fringe was computed to be $p = 0.6 \cdot 10^{-4}$ h$^{-1}$. A value, 10 times larger, was chosen above this limit. Obviously this parameter is valid only in connection with the chosen model.

The dilatancy during the rupture is shown to be about 1%o.
Figure 10. Computed and measured rock pressures.

Figure 11. Computed and measured radial deformations.
To clarify the rearrangement of the stresses which has taken place, figure 12 shows the stress distribution at the beginning and at the end of the calculations.

![Stress Distribution Diagram](image)

**Figure 12. Stress distribution at the beginning and the end of the computation.**

The proceeding extension of the fracture zone is clearly visible.

In figure 13, we see the stress circles corresponding to the points (1) to (5) (figure 12.) drawn for the state at the time $t = 0$, the time $t = \infty$ and at a time $t = 200$ hours.

It is now obvious from the calculations that the stress rearrangement occurs with a characteristic time of about 14 days. The speed of excavation in this section was approximately 1 m/day.

The characteristic index for the viscosity-extraction speed behaviour is:

$$K = \frac{D}{T \cdot V} = \frac{12.8}{14 \times 1.0} \approx 0.9$$

As seen in figure 1, the influence of the three-dimensional stress state at the tunnel face on the long term behaviour of the rock is not to be neglected anymore. This influence has not been considered in these calculations.
Figure 13. Mohr's circles for the points (1) to (5) at different times.

To show the influence of the tunnel face we made an analysis taking into account the three-dimensional stress state near the face. The results of these calculations are shown in the figures 14. and 15.

In figure 14. we see the computed radial deformations (1) without and (2) with the influence of the tunnel face. Curve (3) illustrates the only influence of tunnel face without taking into account the rheological behaviour of rock.

In figure 15. we see the computed rock pressures on the tunnel lining (1) without and (2) with the influence of the tunnel face. Curve (3) is the rock pressure when we neglect the rheological behaviour of rock.
Figure 14.

Figure 15.
5. CONCLUSIONS

The following important conclusions are drawn from the measurements carried out in the Mesozoikum zone of the Gotthard highway tunnel and from the related theoretical investigations:

- Observations and measurements have once more confirmed the existence of viscosity in rock, which is very important for tunnel construction. Whether this viscosity corresponds really to its theoretical meaning or if it appears to be more a sort of progressive failure is, in this respect, of subordinate importance.

- The axiom, whereupon the stresses in the supports are due to the deformation of the rock, which occurs after the mounting of the supports, has been once more confirmed also in an experimental way. These rock deformation can only be reduced, but not prevented by the supports.

- Thus the importance of the various stages of construction gets evident. In particular, the size of the final value of the rock pressure is considerably influenced or even defined by the moment in which the supports are mounted and by their stiffness.

- The extensive and complete measuring system used, proved itself beyond all doubt and supplied very useful data, in particular the closure of the annular void behind the tunnel supports. In our opinion this phenomenon has been too often neglected.

- The calculation method proved good and reliable even if it shall and can be improved upon.

- The effective values of the parameters can be obtained only by rock mechanical measurements. Whereas most of the strength and deformation values can be deduced from laboratory tests there are not known simple tests which allow the determination of the parameters of the viscosity of the rock by reasonable means. These parameters are obviously linked to the chosen model for the rock behaviour.

- The foregoing indicates the importance and the necessity of long-term measurements in the tunnel itself. These measurements seem to be the only way to allow the determination of the characteristic values, indirectly, by means of an appropriate method of calculation.

- Once these values are known, it is the possible by means...
of calculations to determine in advance the rock pressures in relation to a construction method, therefore confirming the most favourable construction method and the adequate and most economical tunnel supports.

As a concluding remark it is recommended that before tunnel construction proceeds, extended measurements should first be carried out, even by the means of previously driven test caverns. During the construction long-term measurements of all values involved should be carried out.
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