Lining load development due to time–dependent rock mass behaviour

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Summary

Time–dependent rock mass behaviour can cause problems as far as the support system of a tunnel is concerned. If not discovered in the primary support stage, long–term creep induces significant loads in the secondary support system, causing a decrease in the factor of safety and pushing the system to the limit of serviceability. Numerical studies were performed to identify the consequences of time–dependent rock mass behaviour, assuming two types of long–term behaviour. The results showed that low but only slightly decreasing convergence rates exert more influence on the loading of the inner liner than initially high but rapidly decreasing rates.

Introduction

Since the ground in which excavations are built is both load and building material, the knowledge of its behaviour is crucial for the design of underground works. The ground behaviour exerts influence on the choice of the support measures and also on the schedule for the support installation. A specific aspect on the ground behaviour is its time–dependency; whether the ground exhibits creep or swelling and how high the long–term strength is compared to the Uniaxial Compressive Strength, just to mention three properties.

In conventional tunneling, the support is divided into a primary support which is installed immediately after each excavation step and the inner liner which is installed some time after the excavation works. If the deformations cease to increase and a stable state is reached during the primary support stage, theoretically the inner liner is only loaded by its own weight, whereas in case of time–dependent ground behaviour the load redistribution processes will continue even after the installation of the inner liner. In order to be able to quantify the long–term loads for the design, numerical studies were performed with the finite difference program $FLAC$ 4.0 and the results are described in this paper. A phenomenological approach is chosen in order to keep the number of parameters as low as possible and to be able to calibrate the model.

Modelling of the ground behaviour

In order to be able to apply a simple rheological model in the simulation of the rock mass, one has to simplify the ground behaviour with emphasis on the dominating behaviour. The two opposites are either the solid body or the viscous fluid. In reality, the behaviour of the ground will be a combination of both. To model a compressible viscous fluid, an elastic viscoplastic rheological model, the *Maxwell*–Model, is chosen. An elastic viscoelastic model, the *Kelvin–Voigt*–Model, is used to simulate the behaviour of a solid. The rheological elements of the used models are shown in figure 1.

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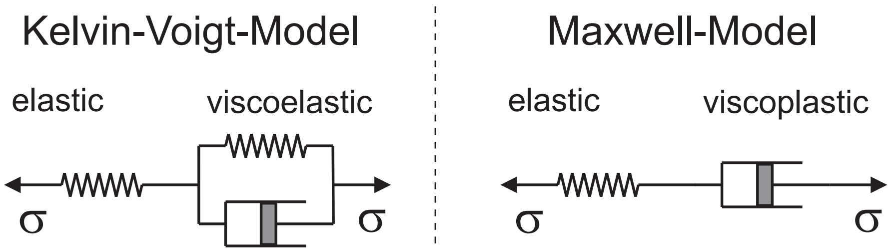

Figure 1: Rheological elements of the used models

Geometry and properties of the system

The geometrical symmetry enables the use of a half–system with the according boundary conditions. The depth of the tunnel axis is assumed to be 200 m and the radius of the circular excavation is $R_A = 6.5\,\text{m}$. The primary stresses in the region of the tunnel can be calculated via the density of the rock mass $\rho = 2.6\,\text{g/cm}^3$ and the tunnel depth, leading to a vertical stress $\sigma_v = 5.2\,\text{MPa}$ and by assumption of hydrostatic conditions ($K_0 = 1.0$) to a horizontal stress $\sigma_h = 5.2\,\text{MPa}$. The elasto–plastic properties of the ground can be seen in table 1.

Table 1: Elasto–plastic rock mass parameters

ρ	E	ν	UCS	f_{ten}	φ	c	ψ
g/cm^3	MPa	–	MPa	MPa	^{o}	MPa	^{o}
2.6	5000	0.25	1.9	0.1	35	0.5	1.0

The time–dependent properties of the ground are defined via the viscosity η resp. η_K and, for the viscoelastic model, also via the shear modulus of the *Kelvin*–element G_K. The variations of these parameters give five viscoplastic sets and 20 viscoelastic sets. Table 2 outlines the used values for the parameters.

Table 2: Time–dependent parameter values

viscosity	shear modulus
50’000 MPa d	100 MPa
100’000 MPa d	250 MPa
250’000 MPa d	500 MPa
1’000’000 MPa d	1000 MPa
10’000’000 MPa d	–

The primary support is simulated with elastic beam elements which represent a 20 cm thick shotcrete shell with a strength of about 30 MPa; to approach a linear–elastic perfect plastic behaviour using these elastic elements, the Young's modulus is sharply reduced from initially 5000 MPa to 10 Mpa when the normal stress reaches 29.5 MPa. The inner concrete liner is simulated with elastic solid elements. The Young's modulus is chosen to be $E = 30'000$ MPa, the Poisson's ratio is $\nu = 0.2$ and the liner is assumed to be 50 cm thick.

Simulation of the excavation process

Basically the excavation of a tunnel is a process in four dimensions – three dimensions in space and the time as fourth dimension. The used code is a 2 D finite difference program; thus one dimension in space is neglected and its influence has to be brought into the calculation otherwise. One means of modelling the advance of the tunnel face through the examined circumsection is to substitute the reactive forces exerted by the excavated material with support forces which are reduced stepwise. The value of the load reduction factor must be correlated to the excavation rate if the analysis is to be performed time–dependent. In this case, the reduction factors are derived from a purely elastic 3D simulation with an assumed round length of 3 m. It is furtheron assumed that the advance rate is one round per day. The development of the deformations in the elastic 3D calculation lead directly to the value of the load reduction factor for each day as given in equation 1.

$$\alpha_t = 1.0 - \frac{u_n}{u_{end}} \tag{1}$$

The activation of the primary support takes place about half a day after the excavation; from then on, further reductions are performed according to the time steps. When the reduction factor reaches zero – that means that any supporting influence of the advancing tunnel face has gone in the examined circumsection – the development of the time–dependent deformations is simulated by letting the analysis time increase. 164 days after the excavation has gone through the circumsection the inner lining is installed and the time is increased further. The calculations end after the time has reached 50 years since the installation of the inner liner.

Analysis of the primary support stage

The analysis of the results of the primary support stage – the first 164 days – comprised the settlement of the crown point and the stress development in the shotcrete lining. The settlement of the crown point is taken from the point when the shotcrete has been installed, so that the predisplacement is neglected. However these data contain the same amount of information that can be retrieved by optical displacement monitoring on site. The development of the axial force in the shotcrete lining is analysed to see whether the shotcrete reaches its ultimate load and how the system behaves after the change of modulus.

The ultimate load of the primary support is reached for the viscoplastic ground behaviour with $\eta = 50'000$ MPa d and for the viscoelastic ground behaviour with a viscosity of $\eta_K = 50'000$ MPa d and the shear modulus $G_K = 100$ MPa and $G_K = 250$ MPa. The development of the crown settlement allows to distinguish between the different parame-

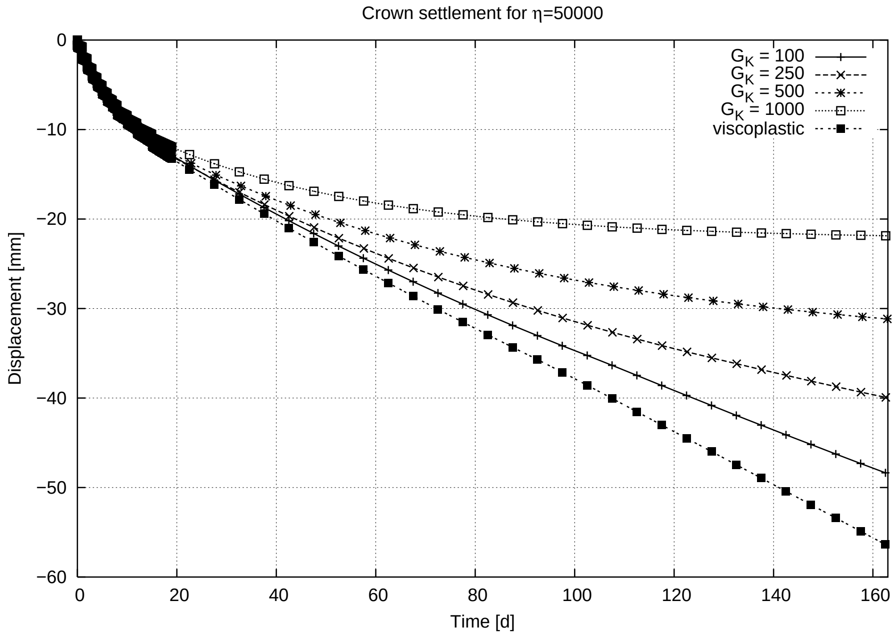

Figure 2: Crown point settlement for the same viscosity

ter sets as long as the viscosity lies between $\eta = 50'000$ MPa d and $\eta = 100'000$ MPa d. Simulations using parameter sets with higher viscosities give nearly identical settlement curves. Figure 2 shows the crown point settlement for $\eta(_K) = 50'000$ MPa d with varying shear moduli ($G_K = 0$ for the viscoplastic case).

Analysis of the load development in the inner lining

The axial forces of the inner lining and of the shotcrete shell are taken at 110 days, one year, 5, 10, 25 and 50 years to reflect the load development. It can be stated that for the simulations with a viscoplastic rock mass behaviour the support pressure tends to the primary stresses – the viscosity influences the point of time when both the primary lining and secondary lining together are loaded according to this state of stress. The distribution of the loads between shotcrete shell and concrete inner liner depends also on the viscosity; a high viscosity stands for low displacement rates in the primary support stage and therefore the loads are attracted by the stiffer secondary support.

The simulations with the viscoelastic model show that the value of the shear modulus G_K has some influence on the loads of the inner lining, but the value of the viscosity η_K exert again a strong influence on the distribution of loading between primary and secondary support. It can be stated that the higher the value of viscosity, the higher the contribution of the inner liner to the sum of loads. This can be easily explained by the fact that a higher viscosity means a lower crown settlement rate in the primary support phase, leading to

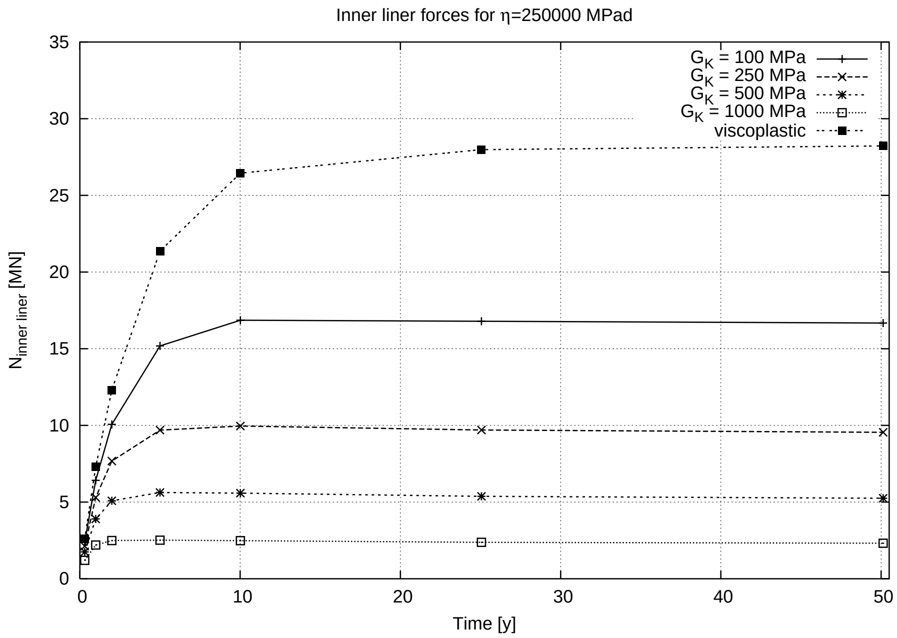

Figure 3: Development of the inner lining forces

lower forces in the softer shotcrete shell. In table 3 the axial forces in the inner liner after 50 years can be seen and figure 3 shows the development of the inner liner forces.

Table 3: Lining forces after 50 years

η [MPa d]	50'000	100'000	250'000	1'000'000	10'000'000
viscoplastic	28.35 MN	28.35 MN	28.35 MN	27.59 MN	9.32 MN
$G_K = 100$ MPa	13.45 MN	15.83 MN	17.51 MN	18.34 MN	8.60 MN
$G_K = 250$ MPa	5.38 MN	7.99 MN	10.40 MN	11.98 MN	7.65 MN
$G_K = 500$ MPa	1.46 MN	3.46 MN	5.95 MN	7.82 MN	6.37 MN
$G_K = 1000$ MPa	0.37 MN	1.08 MN	2.70 MN	4.45 MN	4.65 MN

Correlation between the crown settlement and liner force

Displacement monitoring during excavation works should give the engineer indications whether the rock mass shows creep or other time–dependent phenomena or not. It would be of interest, how the crown point settlements can be correlated to the axial forces of the inner liner.

In figure 4 two settlement curves are shown. Both curves exhibit a displacement rate of 2 mm/mon in the 5^{th} month after the excavation. One would assume that the greater the

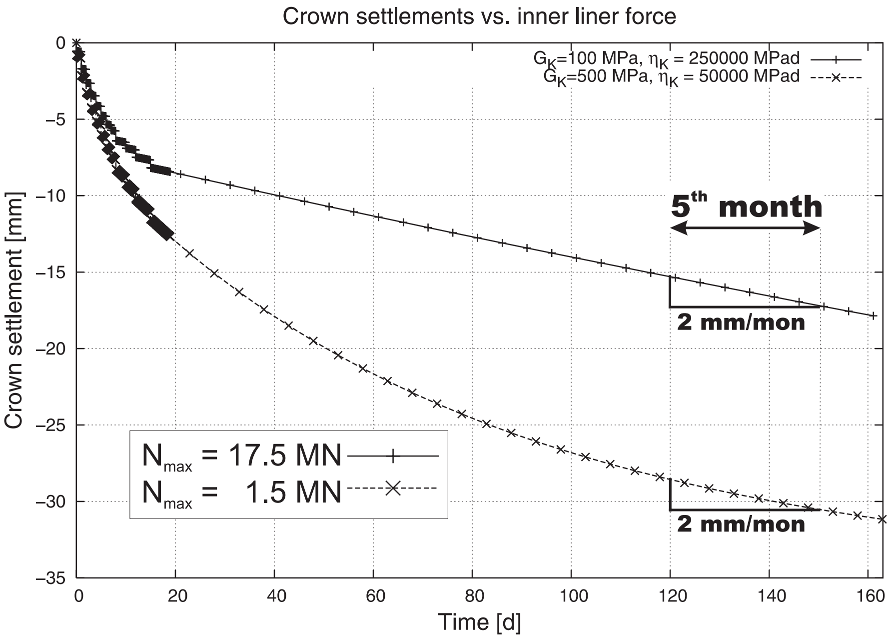

Figure 4: Crown point displacement and inner lining forces

absolute value of the crown point displacement, the greater the value of the axial force in the inner lining; this assumption is disproved by the results, also shown in figure 4. The quality of the two settlement curves indicates that the deceleration of the displacements is the key value for the long–term loading. The stronger the displacement rate decreases, the lower the loads for the inner liner.

Conclusions

It has been shown that time–dependent rock mass behaviour exerts influence on the loading of the secondary support system. This influence depends on the dominating behaviour type – whether the rock mass behaves like a viscous fluid or like a solid body with some delayed elasticity. As far as tunnel design is concerned, the fluid–like behaviour can lead to various difficulties in design which increase with overburden. The development of the settlement rates, especially their deceleration, is found to be a key parameter for the estimate of the long term loading and should be investigated further.

Reference

1. Krenn, F. (2004): “Zeitabhängiges Gebirgsverhalten und dessen Auswirkungen auf die Belastung von Innenschalen”, Ph.D. thesis, University of Leoben.