Mitteilungen für Ingenieurgeologie und Geomechanik	Band 6	2nd Colloquium "Rock Mechanics – Theory and Practice"	p. 129-142	Wien 2004

Design of Tunnel Linings in Swelling Rock

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Abstract: Swelling rock often poses severe problems to design and construction of underground structures. Tunnels have been designed to withstand additional loads from swelling, but most of the broadly used methods of calculating the behaviour of swelling rock and the resulting structural response must be considered as purely empirical approaches, since they cannot appropriately account for the phenomenon in terms of a comprehensive continuum mechanical model. Although some finite element packages designed for geotechnical engineering provide material laws for swelling rock, designers often stick to the familiar simpler approaches such as the application of a uniform radial swelling pressure along the invert lining. The designer should be aware, though, that the application of a uniform radial swelling pressure entails the risk of an uneconomic or possibly even an unsafe design.

A viable approach to modelling swelling behaviour on a continuum mechanic level with an FE-program package without a prepared swelling feature is demonstrated. Comparison of the results with those obtained from the simplistic model of a uniform radial swelling pressure demonstrates the consequences on the quality of results.

1 Introduction

Various current tunnel projects are affected by the phenomenon of swelling rock. In the following, the focus is on swelling as encountered in case of expansive argillaceous rocks, rather than on swelling caused by the transformation of anhydrite to gypsum. In spite of different physico-chemical mechanisms, the mechanical description and the resulting conclusions are quite similar. In practice, swelling can result in inadmissible deformations, heavy loads acting on the lining, excessive stresses and the deterioration of ground strength parameters. Typical failure modes are spalling at the side walls, shear and/or bending failure at the transition from invert to side walls and bending failure at the centre of the invert. In the past, swelling rock often caused damage to tunnel linings, even though the swelling potential had been recognized in the ground investigation campaign.

2 Fundamentals

Essential prerequisite for any kind of swelling analysis are corresponding material parameters. Qualitative mineralogical tests reveal the presence of swelling clay minerals such as smectite and illite, but they do not give insight into the stress-strain-behaviour required for continuum mechanic analyses. Stress-strain relations can be obtained from a variety of rather simple mechanistic swelling tests as described and recommended by the ISRM [1]. Different from long-term testing of anhydrite specimens, swelling testing of argillaceous specimens is relatively quick and inexpensive. The most established testing set-up is the one according to Huder-Amberg [2]: This test is characterized by loading the specimen to a certain stress level, followed by unloading and reloading, then watering the specimen and, finally, step-by-step unloading of the water-saturated specimen. The uniaxial stress-swelling strain relation obtained from these tests can be approximated by a logarithmic swelling law. The swelling strain in axial direction is obtained in terms of the axial stress:

$$\varepsilon_{ax\infty}^{sw} = k_{sw} \ln\left(\frac{\sigma_{ax}}{\sigma_{sw0}}\right) \qquad \text{for } \sigma_{ax,min} \leq \sigma_{ax} \leq \sigma_{sw0}$$

$$\varepsilon_{ax\infty}^{sw} = \varepsilon_{ax\infty,max}^{sw} \qquad \text{for } \sigma_{ax} \leq \sigma_{ax,min}$$

$$\varepsilon_{ax\infty}^{sw} = 0 \qquad \text{for } \sigma_{ax} \geq \sigma_{sw0}$$

Swelling strains $\varepsilon^{sw}_{ax\infty}$ decrease with increasing axial stress σ_{ax}. No swelling strains develop at compressive stresses exceeding σ_{sw0}, or vice versa: σ_{sw0} is the final maximum stress for $t \rightarrow \infty$ at rigid confinement. The largest swelling strains are measured at zero confinement (or at some defined minimum compressive stress). Time dependent behaviour is not a matter in this test: the resulting swelling strains are valid for a given stress level which is maintained at a constant level until the observed swelling strain rate has dropped to a negligible value. The parameter k_{sw} is the slope of the stress-strain relation in the semi-logarithmic diagram. It can be calculated from measured values of $\varepsilon^{sw}_{ax\infty}$ and σ_{sw0}.
Typical swelling anisotropy due to foliation of clayey rocks can be assessed by varying the orientation of specimen placing relative to the axis of the testing apparatus.
There are more elaborate test set-ups which provide insight into true three-dimensional behaviour [3]. Since appropriate testing devices are scarcely available, these set-ups are more of academic interest, rather than of practical relevance.
If any test results are available at all, they will most likely originate from uniaxial tests such as the Huder-Amberg test. Nevertheless they are of high value, as three dimensional swelling laws can still be derived from these data.

3 Modelling Swelling behaviour - Overview

A variety of analysis models can be found in the literature [4, 5]. The approaches range from straightforward stress-dependent models (e.g. [6, 11]) to more sophisticated rheological multi-phase models (e.g. [7]). Despite the progress in modelling swelling behaviour in terms of comprehensive continuum mechanic constitutive laws does not reflect in everyday design practice.
Any continuum-mechanic model reverts to results from tests described above. For the most simplistic model, where some "swelling pressure" load is chosen, σ_{sw0} can be considered as an upper bound of the possible stresses. However, σ_{sw0}, being the maximum stress level at total confinement, will hardly develop around an excavation since there is no absolutely rigid confinement even with relatively stiff support and/or lining. Another simplistic method is to apply constant "swelling strains" over a predefined region below the invert [5]. In analogy to the approach of a uniform pressure, $\varepsilon^{sw}_{ax\infty,max}$ can be considered an upper bound for these strains.
Since secondary stresses underneath and along the invert are not distributed uniformly after excavation, neither the gradually developing stresses due to swelling, nor the related swelling strains are uniformly distributed. This conflicts with the assumption of a constant radial pressure intended to represent the relevant effects of swelling

There are several reasons why engineers often keep to the simplistic model of a uniformly distributed radial pressure along the invert:

- The analysis tool on-hand does not provide a swelling law.
- A thorough investigation of swelling in addition to standard load cases is considered too time-consuming and of little practical relevance.
- The analysis tool does provide a swelling law but the set of parameters is not available from laboratory tests.
- No continuum-mechanic analysis is performed for the lining analysis at all. Hence, swelling can be allowed for only as predefined external load, anyway.

In the following example a superior, yet handy method is applied in order to bring forward the arguments to make use of this method or more refined constitutive laws.

4 Strain Tensor Enhancement Method

The swelling behaviour is introduced into a standard finite-element analysis by enhancing the strain tensor by an additional "swelling term". The additional portion of strains depends on the amount of stress relief due to excavation. It is presumed that the swelling strains depend on the normal stresses in the principal directions of the anisotropic material. Orthotropic swelling properties of argillaceous rock are characterized by a main swelling direction perpendicular to the foliation planes and smaller swelling potential in the foliation planes. Using a coordinate system with axes x_i coinciding with the material's principal directions, the chosen swelling law can then be written as:

$$\varepsilon_i^{sw} = \beta_i \cdot k_{sw} \cdot \lg\left(\frac{\sigma_i}{\sigma_{sw0,i}}\right)$$

The stress-strain behaviour is defined by three material parameters easily obtainable from aforementioned standard swelling tests. A factor of anisotropy, βi, is introduced in order to account for orthotropic swelling behaviour.

The availability of water, triggering the swelling process, is usually restricted to the vicinity of the invert. A single-phase analysis is performed (i.e. coupling of water saturation with stresses and swelling strain rate is not accounted for). Hydraulic conditions are indirectly considered by restricting the region of application of the enhanced swelling term to the part of the model below the tunnel. In reality, water is not necessarily available at this extent, but infiltrates along or close to the invert into the rock mass underneath. The spatial extent of zones of high saturation and swelling triggered by the water concentration is hence restricted. Seepage analyses performed for a number of different excavation shapes in unsaturated media have shown that these zones of maximum saturation roughly match the areas where swelling occurs due to stress-relief. It can therefore be concluded from this similarity that the lack of a two-phase analysis does not necessarily result in a significant error with regard to the spatial extent of swelling, provided that the region of maximum water saturation coincides with the region of stress relief.

5 Analysis of an Alpine Road Tunnel

The approach was used for 2D-FEM analyses of the Achrain Road Tunnel project in Vorarlberg, Austria. (The project has been presented in [8].)
The 5 cm shotcrete lining of a circular pilot drift (TBM, 3.9 m in diameter) suffered considerable damage resulting from excessive invert heaves of up to about 20 cm.
A section with pronounced swelling was chosen for an enlargement of the pilot tunnel to the size of the top heading of the main tunnel (12 m wide and 5 m high). It was supported with 20 cm of shotcrete along the invert and 30 cm shotcrete and rock bolts in the crown. In order to assess the swelling behaviour on a full scale, the invert was watered over a length of 6 m every other day during one month [8]. Deformations were monitored at five measuring sections. The measured invert heaves amounted to more than 25 cm and kept increasing at the time of the last measurements.
No mechanical swelling tests comparable to the Huder-Amberg test had been conducted on rock specimens of the Achrain Tunnel. The only data available for the definition of a swelling law were those from a free swelling test. During such a test swelling strains are recorded while a specimen is watered and loaded with a low pressure of 5 kN/m². The maximum measured value of the swelling strain was 1 %. Since the samples had been recovered by means of water-flushed drilling, it can be assumed that they had already undergone some swelling before arrival at the laboratory. Therefore, the maximum swelling strain was assumed as $\varepsilon^{sw}_{1,max} = 2$ % in the analyses. The remaining two parameters usually available from swelling test, $\sigma_{sw0,2}$ and β, had to be estimated. Regarding $\sigma_{sw0,2}$, being the maximum

swelling stress component in direction perpendicular to the foliation developing under constant volume conditions, several authors have shown that at deep tunnels its in-situ value is often about the size of the primary vertical stress, although under laboratory conditions σ_{sw0} may exceed the overburden stress by several hundred percent. For the present analysis, the value of $\sigma_{sw0,2}$ is assumed to be equal to the primary vertical stress at the invert. The parameters chosen for the analyses are shown in tables 1 and 2. The factor of anisotropy, defined as $\beta = \varepsilon^{sw}_{2,max} / \varepsilon^{sw}_{1,max}$, is assumed as $\beta = 5$, which is in agreement with test series performed on clayey rock [9]. The applied swelling law in the direction perpendicular to the foliation planes of the material is plotted in figure 1.

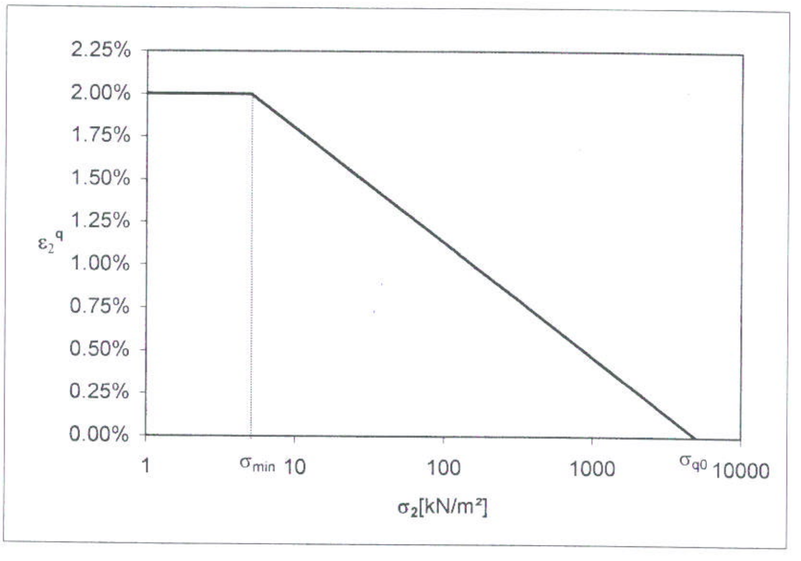


Figure 1: semi-logarithmic swelling law in direction perpendicular to foliation with $\varepsilon^{sw}_{2,max} = 0.02$, $\sigma_{sw0,2} = 4900$ kN/m² and $\sigma_{min} = 5$ kN/m² (compression)

Table 1: Material properties for the analyses

ground		shotcrete	
Young's modulus	5000 MPa	Young's modulus (green shotcrete)	5000 MPa
friction angle φ	40°	Young's modulus (hardened shotcrete)	25000 MPa
cohesion *c*	0.5 MPa	long-time-stiffness	10000 MPa
Poisson's ratio ν	0.3	Poisson's ratio ν	0.2
specific weight	26 kN/m³		

Table 2: Parameters chosen for the swelling law

swelling parameters	
maximum swelling strain	$\varepsilon^{sw}_{2,\max} = 2\ \%$
maximum swelling pressure	$\sigma_{sw0,2} = -4900$ kN/m²
factor of anisotropy	$\beta = 5$

Since the foliation dips insignificantly and strikes almost parallel to the tunnel alignment, it is assumed horizontal. A transformation of the stress tensor for the calculation of swelling strains is thus not required, because the material's principal directions of orthotropy coincide with the global coordinate system. The finite element program MSC.Marc 2003 [10] was chosen as analysis tool. The ground is modelled with isoparametric quadratic plane-strain elements assuming Mohr-Coulomb behaviour with associated flow. The lining, modelled with linear elastic three-node Timoshenko beam elements, is coupled to the ground by means of rigid gap elements which allow detachment of the lining and limited sliding between ground and lining (friction parameter μ=1.0). The finite element mesh in the vicinity of the opening is shown in figure 2.

An iteration scheme for the swelling stresses and strains is initiated in a calculation step following the excavation. The iteration scheme provides a steady and monotonous approach of the stress and strain fields to the final state. It can be regarded as a rudimentary consideration of the development of swelling over time. After a number of iteration steps the changes in strains and stresses become sufficiently small and the calculation is terminated. The assessment of the development of swelling over time is not aimed at, since hydraulic conditions can be complex, sometimes practically irreproducible in jointed rock. Based on in-situ observations and the analysis results, it is assumed that the most unfavourable conditions prevail at $t \rightarrow \infty$, at the end of the swelling process. The investigation of intermediate time steps is therefore usually not required for an ultimate limit state-design.
The pilot drift and the enlarged section are analysed separately (i.e. the prior excavation of the pilot drift) is neglected for the analysis of the enlarged section.

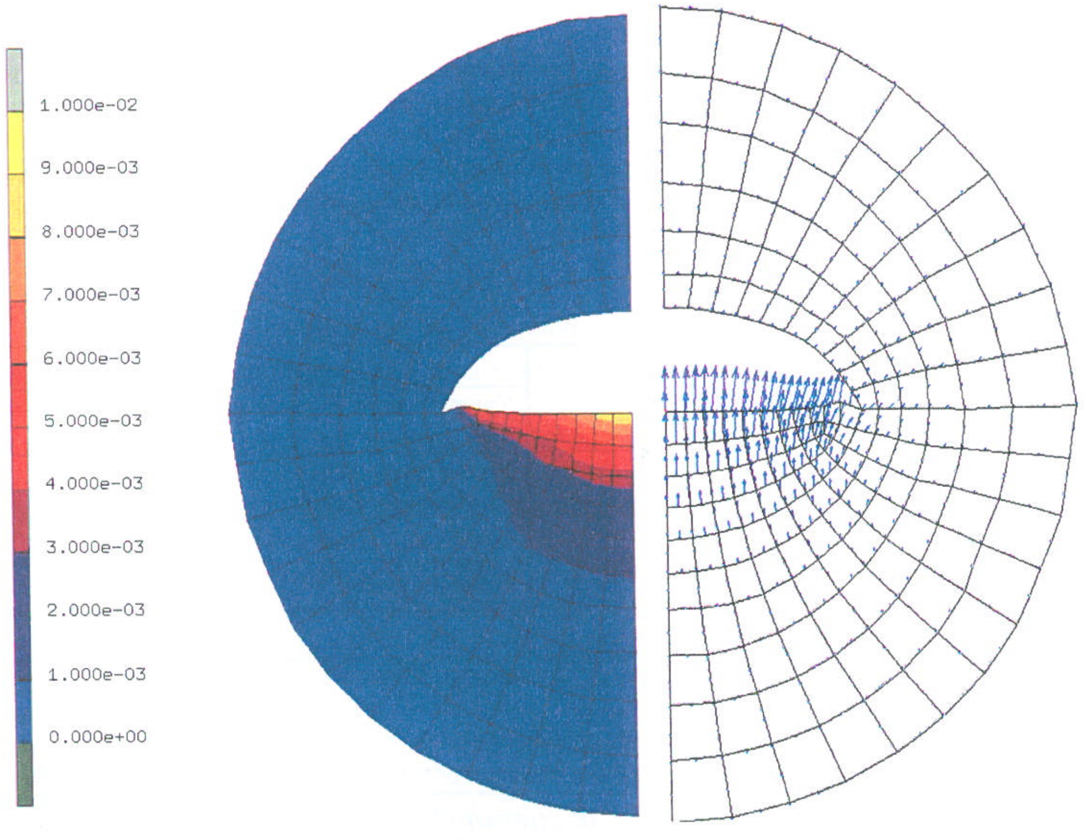


Figure 2: vertical swelling strains ε^{sw}_2 underneath the enlarged section (left) and displacement vectors due to swelling (right)

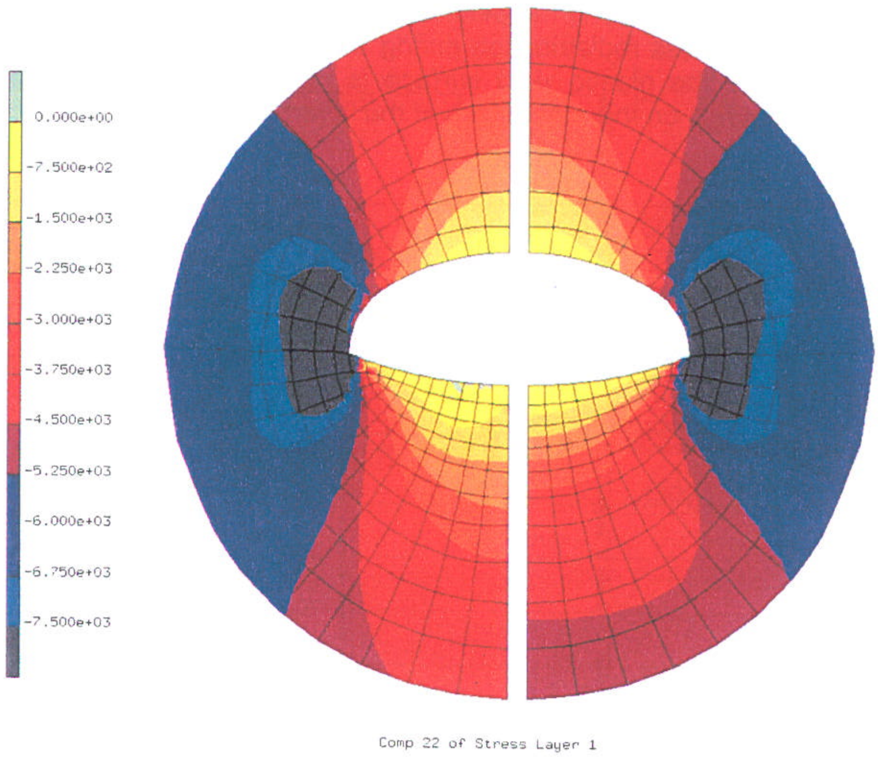
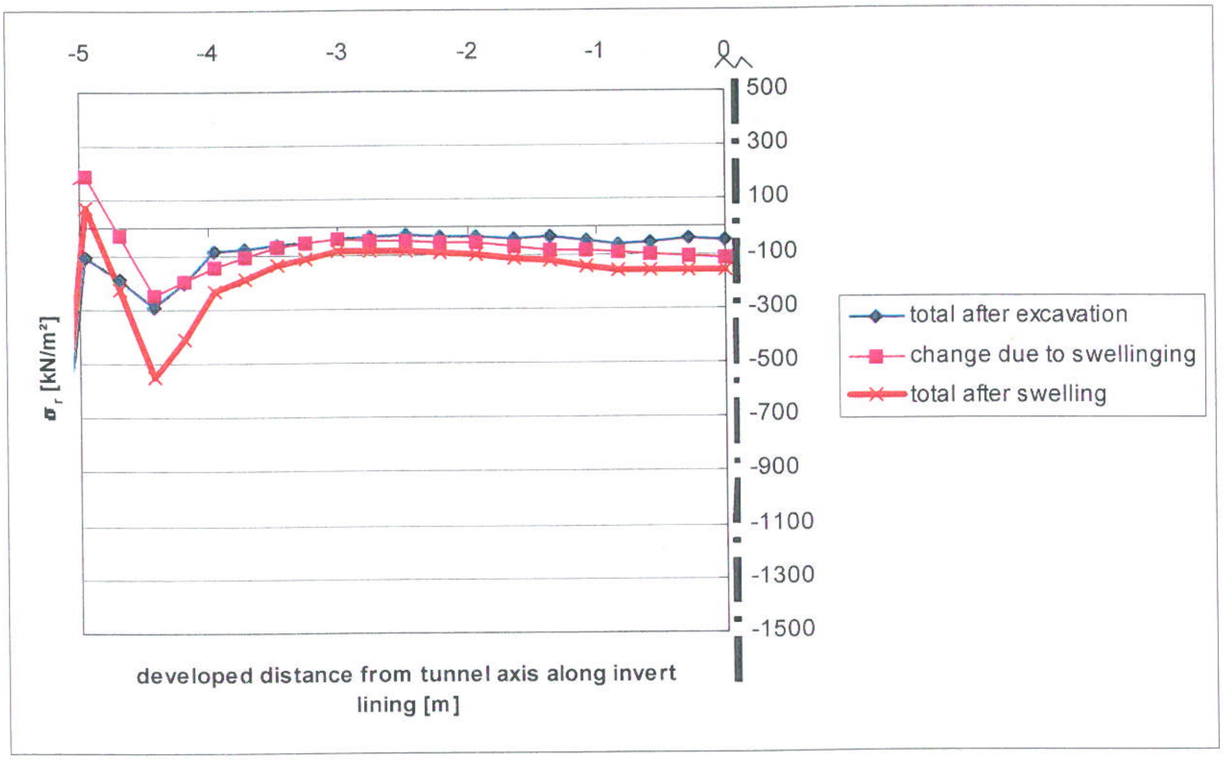


Figure 3: vertical stresses σ_{22} [kNm/m]around enlarged section, after excavation (left) and at the end of the swelling process (right)

6 Results

Figure 2 confirms that swelling causes substantial subsequent deformations. For the enlarged section an invert heave of 17 mm due to swelling after excavation has been calculated. As swelling strains are partly confined, the stress level underneath the invert rises during the swelling process (see figure 3). Final stresses increase with support stiffness up to the limiting case of a totally rigid confinement and stresses approaching $\sigma_{sw0,2.}$
It is obvious that the installation of a resilient lining produces smaller pressures on the invert than a stiffer lining with a high resistance capability and all structural consequences (i. e. excavation shape, concrete thickness, reinforcement). A technical approach which has been applied to various projects is to exchange the ground underneath the invert with bloated clay, thus admitting some unconfined swelling heave before pressures develop onto the invert lining. Theoretically, final pressures can thereby be reduced considerably. A shortcoming of this method is that it requires some certainty about the swelling potential in-situ. The necessary prognosis of absolute invert heaves due to swelling is not easy and very risky. The thickness of a deformable zone should therefore be chosen in a rather conservative way. The economic aspects, such as the additional excavation of the deformable zone, the risk of higher invert heaves and pressures resulting in inadmissible deformation, require particular consideration, especially in high speed railway tunnels. For this reason, this concept is rather applied in swelling anhydrite, since swelling pressures arising there are at times too high to be taken up by any economic support system. In argillaceous rock, stiff support systems have frequently been found to be the more economic alternative [11].

When it comes to the structural design of a tunnel lining subject to swelling behaviour, the distribution of any additional load due to swelling is of particular interest. Figures 4, 5 and 6 illustrate that the amount of additional radial stresses acting on the invert is not distributed uniformly, but varies substantially along the perimeter. It can even become negative at some points in the course of the stress redistribution during the swelling process. In particular this has been observed where the plastic yield surface is reached and the ground plastifies underneath the crown footings on the outer sides of the invert lining.



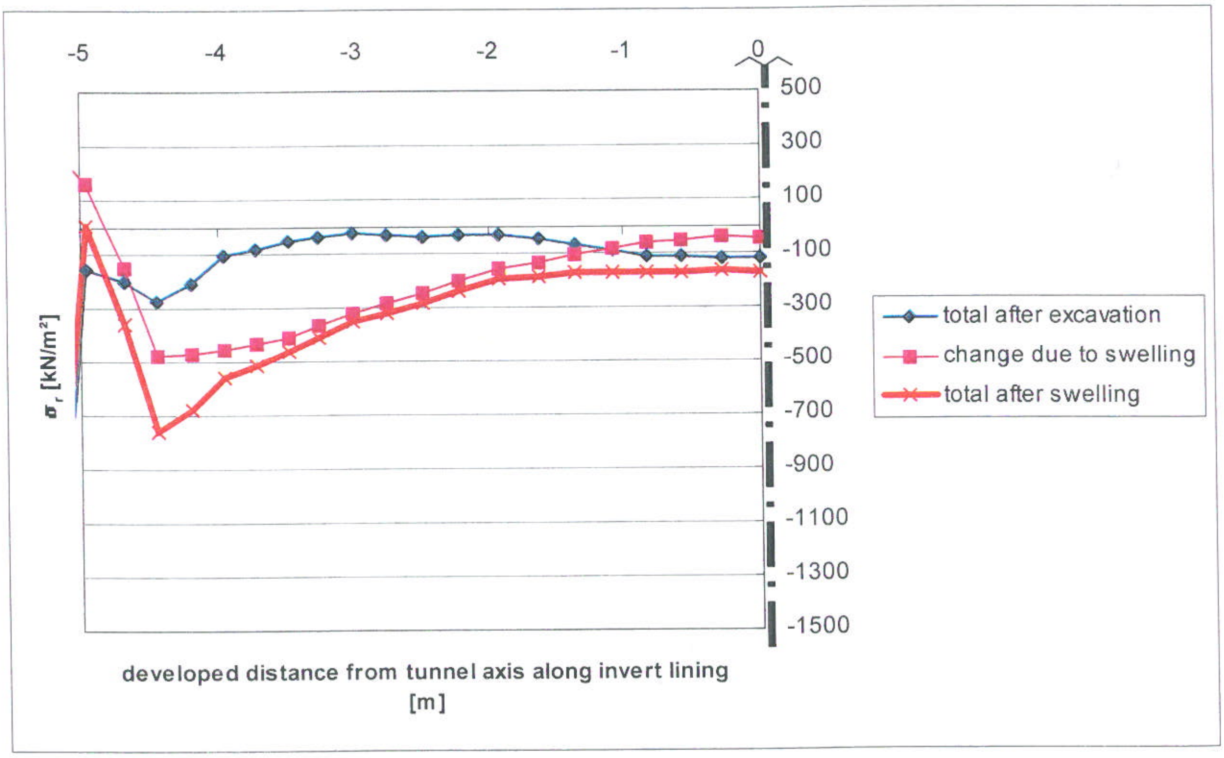


Figure 4: radial stresses σ_r acting along invert lining of enlarged section starting from centre point (=0 m) for softer lining (top) and stiffer lining (bottom)

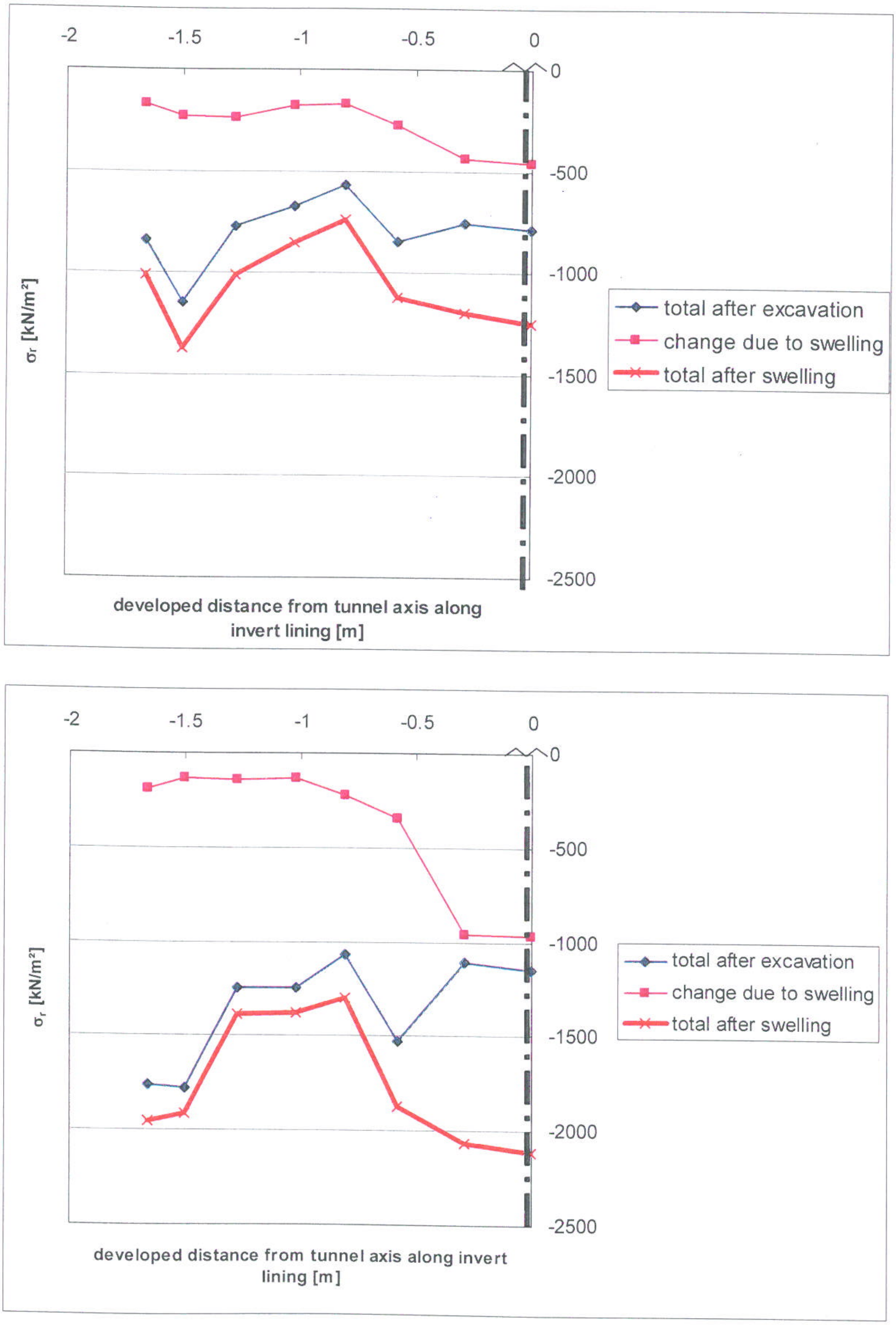


Figure 5: radial stresses σ_r acting along invert lining of circular pilot tunnel starting from centre point (=0 m); for softer lining (top) and stiffer lining (bottom)

The final stress and strain state for $t \to \infty$ results from the interaction of the underground structure with the ground. As stiff structures resist swelling strains, they attract larger swelling pressures. Softer structures give way to swelling strains and typically undergo large invert heaves. The development of swelling pressure does not only depend on the global structural stiffness but also on local stiffness conditions. This is one reason why the swelling pressure basically varies along the invert line. Measurements carried out at other projects confirm that in general there is no evenly distributed "swelling pressure" in reality [11].
Figures 4, 5 and 6 also show that swelling pressure distribution depends on the cross sectional geometry and support stiffness (i.e. lining thickness).
Figure 4 (top) displays additional radial stresses acting, i.e. "swelling pressure" on the invert lining, for the enlarged section with the primary support (20 cm shotcrete in the invert). Increasing the stiffness to a degree common for permanent support (total lining thickness up to 85 cm in the invert and 55 cm in the crown) results in a more pronounced deviation from a uniform swelling pressure distribution (figure 4, bottom).
The swelling pressure distribution along the invert of the circular pilot tunnel differs significantly from the enlarged section. It has been calculated for the soft lining as built (d=5 cm, figure 5, top) and a stiff lining (d=55 cm, figure 5, bottom).
Swelling pressures decrease with increasing distance from the centreline. The difference might originate from a combination of causes:

- The secondary stress field of the circular opening is significantly different from the one of the enlarged section. The extent and amount of stress relief is larger underneath the circular tunnel.
- Apart from the lining thickness, the shape of the circular pilot tunnel alone entails a generally stiffer lining behaviour, whereas the unfavourable shape of the enlarged section and the changed stress field due to swelling result in additional plastic strains. Consequently, radial stresses acting on the invert at such points even decrease during the swelling progress.
- The material's principal direction is radial at the centreline of the opening because of the horizontal foliation. Contrary to the enlarged sections, radial and principal directions deviate from each other quickly with increasing distance from the centreline of the circular section, resulting in less swelling potential in the radial direction of the swelling pressure.

Results for the cross section of the main tunnel are given in figure 6. (For a close-up of the FE-mesh around the excavation see figure 7.) The final lining is installed after about one half of the final invert heave has developed. This latter analysis is based on somewhat different ground properties. (E=20 GPa, φ=33°, c=2 MPa, $\varepsilon^{sw}_{2,max}$=3%; shotcrete thickness invert d_S=40 cm, invert lining thickness max. 230 cm). Again, it can be seen that the amount and distribution of current swelling pressures substantially depend on the shape of the opening and the stiffness of support and ground. A "swelling pressure" cannot be derived only from laboratory tests without the due regard of these leading factors. Nevertheless, at times geotechnical reports give values of "swelling pressures" to be calculated with.

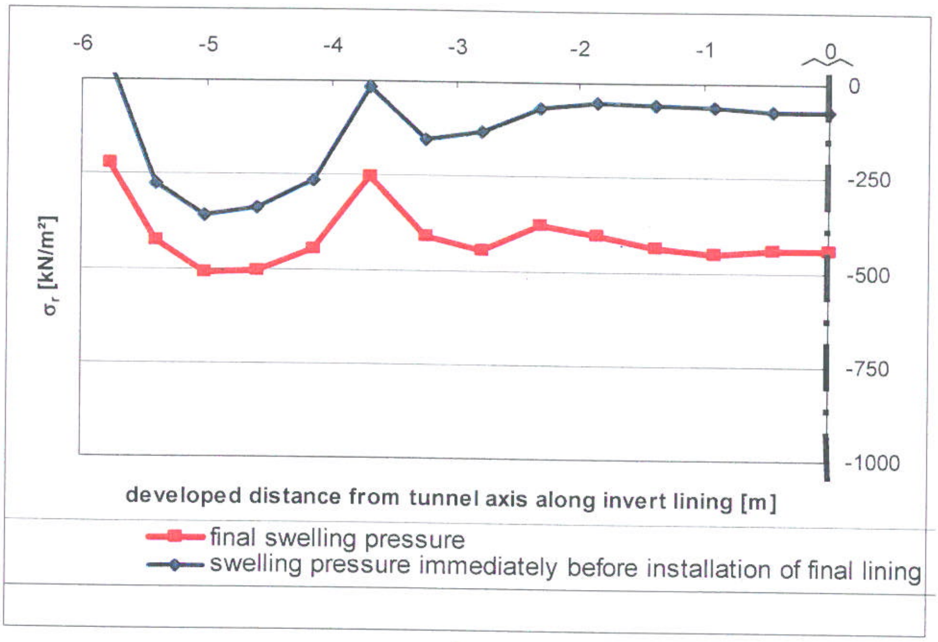


Figure 6: swelling pressure on main tunnel, i.e. changes in radial pressure sr due to swelling immediately before installation of final lining and at the end of the swelling process

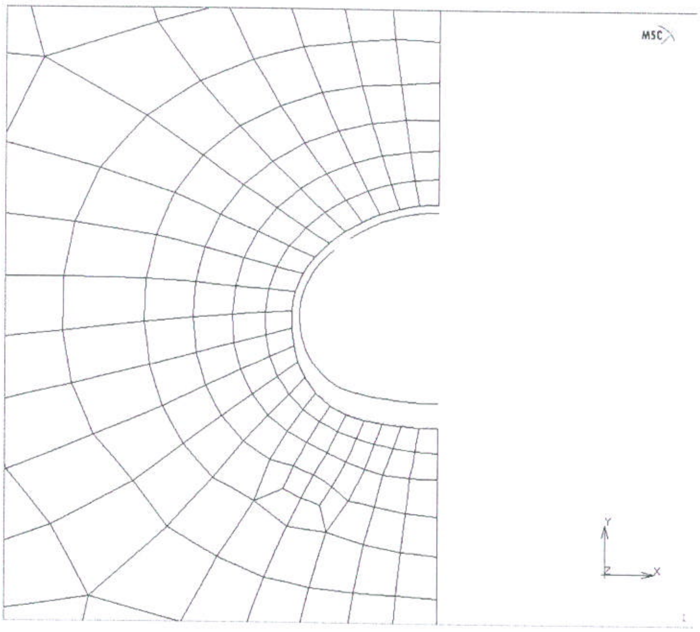


Figure 7: close-up of finite-element mesh around main tunnel

Figure 8 sums up the importance of a realistic pressure distribution. Bending moment distribution resulting from the analysis with the strain tensor enhancement deviates significantly from the one obtained after application of a uniform radial pressure of p=250 kN/m² (being more or less a mean value of the uneven distribution in figure 4) along the invert. Using such differing values of sectional forces as basis of subsequent ultimate limit state design will result in rather unlike distributions of reinforcement.

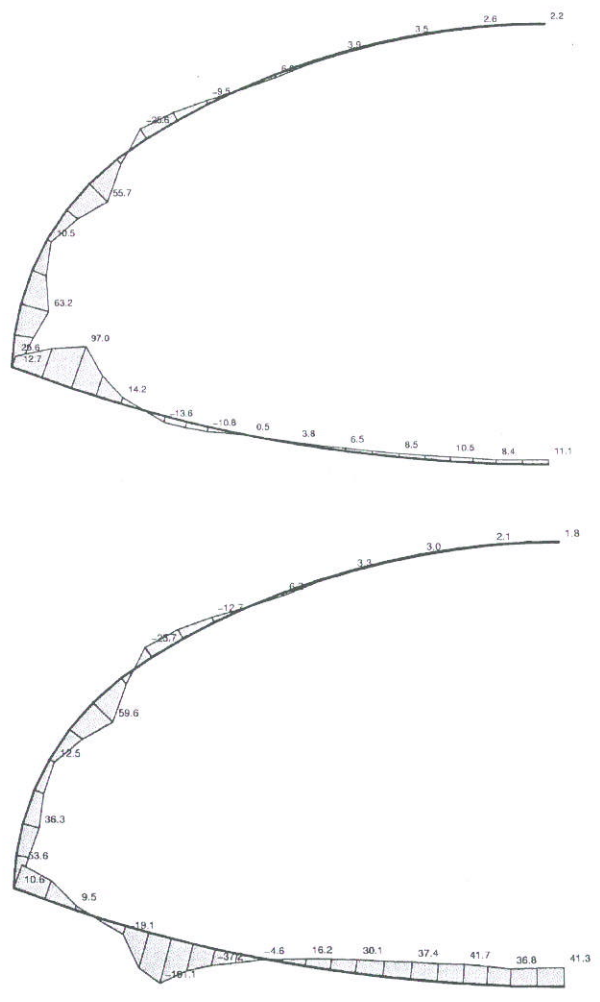


Figure 8: bending moments [kNm/m] in the shotcrete lining of the enlarged section, at the end of the swelling process (top), and after application of constant swelling pressure (bottom)

By means of the strain tensor enhancement approach, the location of the failure of the invert linings, both of the circular and the enlarged sections, could be simulated in good agreement with the observed damage in reality [5].
The model fails, however, to predict the excessive invert heaves measured at the enlarged section after the full scale swelling test. This may be caused by two factors:
First, the lining is modelled as being linear elastic. Non-linear post-failure effects such as an immediate drop of support stiffness are not accounted for. These effects would lead to a further increase of deformations.
Secondly, most expansive rocks such as the encountered marl tend to decompose and soften during water absorption. Additional invert heaves may be a result of mechanisms triggered by the softening. To this end, the softening was simulated in a further analysis by reducing the stiffness over a thickness of approximately 1.3 m underneath the invert rather radically from 5.000 MPa to 25 MPa, starting from the beginning of the swelling iteration. This brings about an additional heave of another 13 mm resulting in a total of 28 mm of invert heave caused by swelling which is still far less than the in-situ values. Realistic modelling of rock decomposition due to water uptake constitutes another challenge beyond the simulation of swelling behaviour.

7 Summary and conclusions

The results of a series of finite element analyses based on a simple enhancement of the constitutive law are compared with those obtained from the simplistic application of a uniformly distributed radial pressure along the invert in order to assess the benefits and shortcomings of either method with regard to its applicability and reliability.

Results show that

- the chosen swelling law predicts swelling strains in a region below the invert of about circular shape with a diameter of up to twice the tunnel span; the extent of this region is in good agreement with the prediction of water distribution resulting from unpressurised infiltration along the invert into initially unsaturated homogeneous ground;
- the resulting stresses (i.e. the "swelling pressure") acting on the lining of the tunnel are not uniformly distributed;
- the locations where the analysis predicts the most unfavourable stress states in the lining match very well with the locations where damage was observed at the construction site. Some of the so-called engineering approaches are not capable of appropriately predicting the critical limit state, thus implying the risk of an either unsafe or uneconomic design.

Simplified models require estimates of finally arising swelling strains or "swelling pressures" acting on the invert lining, respectively. Contrary to the additional parameters of the presented enhancement of the strain tensor, neither can be derived directly from simple laboratory tests, since they do not depend only on rock mass properties, but also on the shape and stiffness of the underground structure as a result of the interaction of lining and ground.
While it is almost impossible to judge the effect of design variants (for instance, the effect of a collapsible zone) using the most simplified models, the strain tensor enhancement method (and more advanced models) allows to quantify these effects.
Finite-element analyses are supposed to simulate the behaviour of the ground and its interaction with the structure. When making use of the simplistic method of a uniformly distributed radial swelling pressure along the invert lining, the results of such an FEM analysis are hardly better than those obtained from a bedded beam analysis. (This certainly concerns only the load steps of swelling, but these are usually the most critical.)
The above discussion of simple models and their shortcomings demonstrates that existing continuum mechanical models are a powerful and indispensable tool for an economic and safe design of tunnels in swelling rock. Especially when the finite-element package on-hand does not provide a swelling law, the designer might refrain from the self-made implementation because it is regarded a quite time consuming and tedious task. The presented example shows that this is not the case. In fact, the superior quality of results is worth the effort also in everyday design analyses.
When modelling time dependent swelling behaviour of argillaceous rock, the engineer must be aware of the major uncertainties caused by irreproducible hydraulic conditions in heterogeneous jointed rock on the one hand and the difficulties in modelling rock disintegration due to water uptake on the other.
If the engineer follows the simplistic approach of a uniform pressure along the invert, safety factors should be chosen in a rather conservative manner, since the computed sectional forces might be far from reality. Some basic variations of the pressure distribution according to local and global stiffness conditions (e.g. linear change of the pressure between centreline and side walls) seem advisable.

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