Aspects Of Safety And Serviceability Of Heaps With Viscoplastic Materials Due To Man-Made Liners Or Natural Slip-Surfaces

Rolf Katzenbach[1], Matthias Seip[2] and Johannes Giere[1]

[1]*Institut für Geotechnik, Technische Universität Darmstadt, Darmstadt, Germany*
[2]*Ingenieursozietät Prof. Dr.-Ing. Katzenbach GmbH, Darmstadt, Germany*

ABSTRACT: The tailings of potash mining are piled up in huge heaps with heights of up to more than 200 m and a ground area of often more than a square km. The material behaviour of the salt is strongly visco-plastic, so that the slopes of the heaps are moving constantly at slow rates. As the strength of the salt is rate-dependent, structural analysis of the slope stability has to consider the deformations and deformation rates and the interaction of subsoil and slope. Due to the complex material behaviour the structural analysis is accompanied by an extensive measuring programme within the Observational Method.

The paper focuses on two slopes of tailing heaps, where huge deformations partly at accelerating deformation rates occurred, due to natural respectively man-made slip-surfaces in the subsoil. The deformation rates were critically high and deformations induced serviceability problems to infrastructure at the base of the slope. It is shown, how the restoration of the endangered slopes and infrastructure was established. The concept of restoration is based on both intensive measuring and numerical simulations.

1 INTRODUCTION

The potash minging in Germany yields large amounts of tailing. The tailing - roughly 50 to 75 % of the mined material - consists mainly of rock salt (Halite, NaCl), small amounts of various other salts and clay minerals (Beer 1996). Due to the lack of economially and technically reasonable processes for further use, the salt is dumped as tailing onto huge heaps, which have currently heights of up to 240 m and which dimensions in the ground view often exceed more than a kilometer.

As all structures these heaps have to be designed to meet the requirements concerning stability and serviceabilty. Due to their enormous dimensions they also affect the surrounding structures, such as buildings, pipelines, roads and railroads. When dealing with the design of the heaps and all connected structures special care of some typical aspects is required.

- With a density of up to approx. 1.9 t/m³ the heaps induce very high loads to the ground and especially to the base of the slopes. Stresses on all structures are considerably high and require a very robust design.

- A strongly visco-plastic material behaviour yields a permanent movement of the slopes which often lead to problems concerning the serviceability of adjacent structures.

- Rock salt is soluble in water. Precipitation solutes the salt from the heaps and washes it into the groundwater. To prevent the pollution of the environment with high salt concentrations, the water has to be gathered and channelled to ditches. Similar to base linings in landfills, a system of seals and ditches has to be installed underneath the bases of the slopes.

In this paper two examples of heaps are presented, where it was required to deal with these properties and conditions in special ways.

2 TIME DEPENDENT BEHAVIOUR

Structures consisting of materials with time dependent material behaviour show some special features in their load-displacement reaction. Compared to structures with no or neglegible time dependent behaviour they may be moving at a constant rate although they are stable (fig. 1). A collapse of the structure is typically preceded by an acceleration of the movements (type 3 in fig. 1). The time scale in this schematic view varies with the viscosity of the material. For rock salt the typical time scale would be days to weeks.

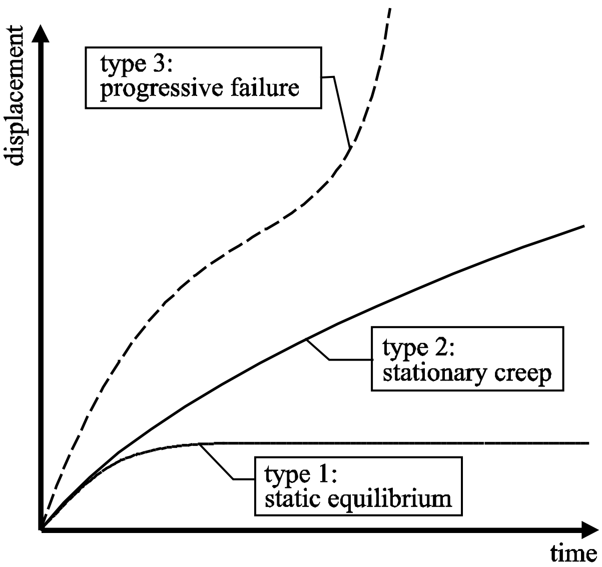

Fig. 1: Time-displacement for time-dependent material behaviour

Rock salt is due to its crystalline microstructure a strongly time-dependent material. When it is exposed to deviatoric stresses salt shows a deformation at a nearly constant rate. With time passing and under the influence of water and overburden, the granular rock salt with an initial density of $\rho = 1.4 \ldots 1.5$ t/m³ gradually transforms into a rather compact rock salt with a density of up to $\rho \cong 1.9$ t/m³, what is near to the original density of $\rho = 2.2$ t/m³.

The strength of rock salt depends on both the stress level and the deformation rate. Similar to the higher bearable deviatoric stresses at a higher stress level, bearable deformation rates increase with increasing stress level. This behaviour is illustrated in figure 2: The results of two strain driven triaxial tests on rock salt at different strain rates are given. The cell pressures in both specimens are the same, whereas the strain rates vary by the factor 1000. The specimen with the higher strain rate of $d\varepsilon_1/dt = 10^{-8}$ 1/s shows the typical behaviour of a dense granular material: It reaches a peak strength q_{fr} followed by a decrease in bearable deviatoric stress and macroscopic failure with a deterioration of the specimen. The second specimen was submitted to the same strains at a substantially lower strain rate of $d\varepsilon_1/dt = 10^{-5}$ 1/s. At this rate the specimen does not show macroscopic failure but creeps at a constant stress level q_{stat}. With reference to the different observed behaviour, the strain rates are called overcritical or undercritical respectively (Boley 1999).

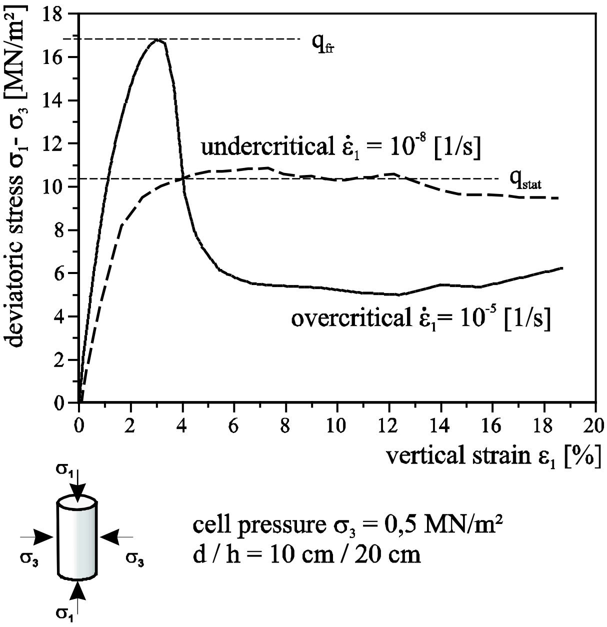

Fig. 2: Typical triaxial tests on rock salt

Creep is driven by stresses. Submitted to a deviatoric stress state, rock salt begins to creep at an initially high rate, slowing down asymptotically to a constant rate which is called the stationary creep rate ε_{cr}^{stat} (fig. 3). At this rate micromechanical creep mechanisms (dislocation creep, diffusion creep) take place, allowing the specimen to bear large strains without macroscopic failure.

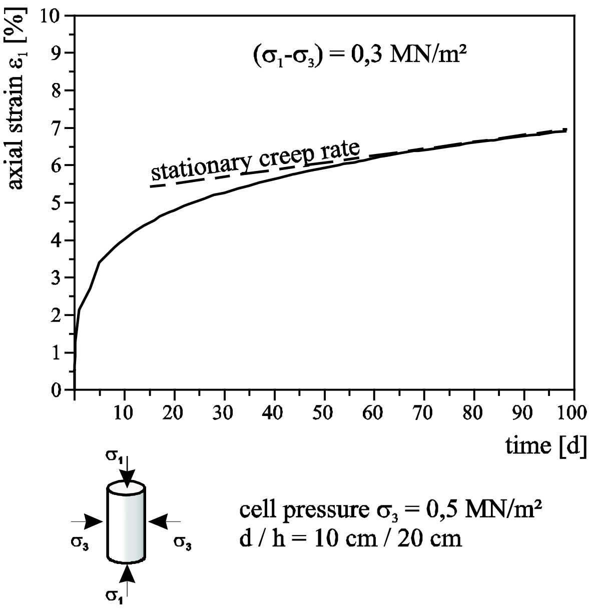

Fig. 3: Typical creep curve of rock salt

The formal framework connecting creep rates and corresponding stresses is given by creep functions, considering both volumetric and deviatoric creep mechanisms. Based on extensive experimental data from rock salt of several potash mines in Germany, Boley formulated constitutive equations describing creep and strain rate dependent failure of rock salt (Boley 1999). For a more detailed discussion of the material behaviour of rock salt see for example (Chumbe et al. 1996) (Fordham 1988), (Munson & Wawersik 1991), (Boley 1999).

The bearable acceleration of the structure and the distance from collapse depend on site conditions and parameters, which are hard to determine with appropriate accuracy. It is therefore crucial to observe the movements by geodetic and geo-technical measuring devices and to review the measuring results regularly. The theoretical models for prediction have to be validated and updated in order to assess the risk of failure of these structures and are applied within the frame-work of the Observational Method (Katzenbach & Hoffmann 2003)

3 MOVEMENTS DUE TO MAN MADE SLIP SURFACES

The first example deals with a 190 m high heap, which covers a ground area of >1 km² (fig. 4). It is situated in the German highlands. Dumping is performed at a yearly dump rate of about 2 Mio tons from the top of the heap. This dumping process lets the heap grow laterally at a constant height.

Fig. 4: View on tailing heap

At the time when the large displacements where observed first, dumping was performed on the western slope. Figure 5 schows a cross section of the western slope with a slope angle of approx. 37°. The western slope at that time followed a valley leading to NE. The height of the eastern slope of the valley is approximately 50 m with the steepest part near the bottom of the valley. The base sealing is situated in the bottom of the valley, consisting of a 50 cm thick mineral lining and a drainage layer of gravel. The base sealing is shown in light grey in the ground view (fig. 5).

The subsoil of the heap consists of Bunter Sand-stone. Under a thin cover of humus lies the heavily weathered Bunter Sandstone as a 2 m thick layer of silty sand with varying portions of stones. The intact Volpriehausen Sandstone is found in a depth of approx. 3 m. It consists of a layered Sandstone with thin layers of Siltstone and has a complex joint system. The strength of the rock is considered very high, given the absence of slope parallel joints.

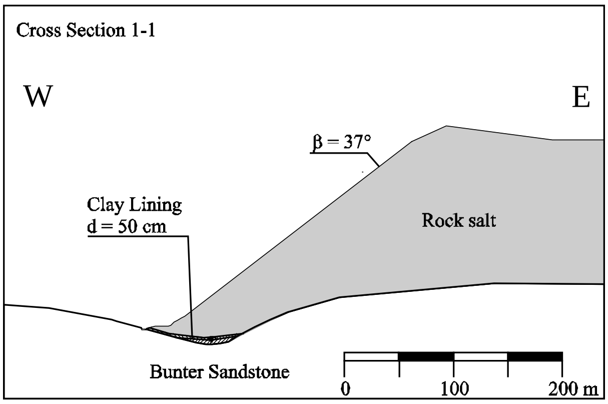

Fig. 5: Cross section

The strength parameters of the sand and the mineral lining as tested in triaxial tests or direct shear tests respectively are given in table 1.

Table 1: Strength parameters of sand and clay lining

	φ' [°]	c' [kN/m²]
Sand	32	0
Clay lining	20	10

2.1 Measuring programme to identify the moving slope

When large deformations of a dewatering ditch surrounding the heap indicated a possible move-ment in 1997, an extensive measuring programme was started to verify the observations and identify possible causes. Further investigations showed cracks on the top of the heap and fissures on the sides of the test pit (fig. 6), indicating a failure surface. The following measuring programme was carried out:

- geodetic measuring of 180 points adjacent to the footing of the western slope,
- two horizontally installed extensometers on

the top of the heap and
- one inclinometer with a depth of 22 m.

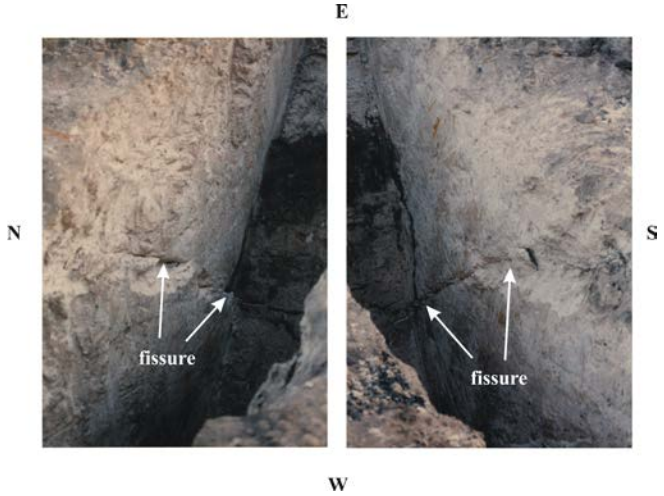

Fig. 6: Fotos of the test pit including fissures

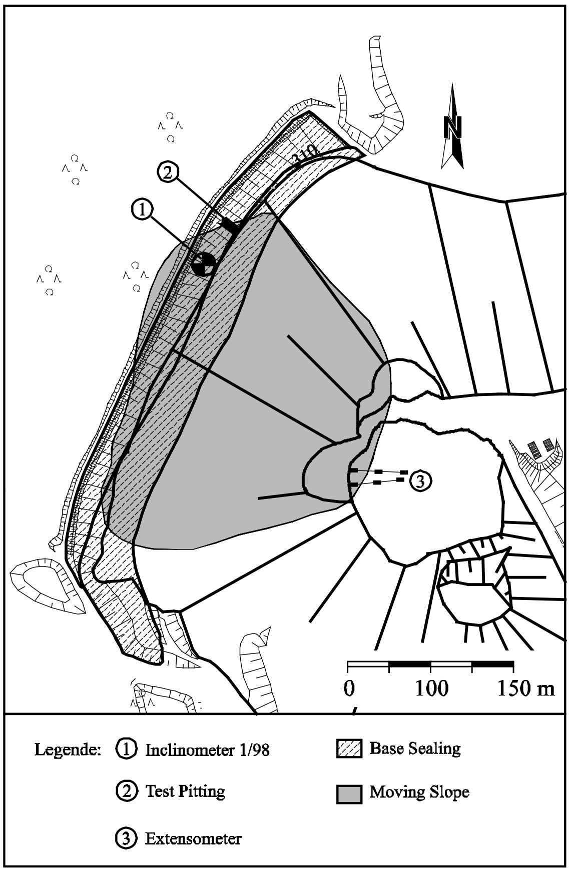

Fig. 7: Ground view of the moving slope and the measuring devices

According to the results of the geodetic measuring devices a major part of the slope moved at a rate of 2 cm/d. The spatial extension of the moving slope could be well defined and is shown in fig. 8. The mass was estimated to approx. 2 Mio. tons.

The cause of the observed movements could be identified clearly by the test pitting and the inclinometer. Both soil profiles showed a heavily remoulded mixture of clay from the base lining, pebbles and rock instead of the base sealing, which had obviously been destroyed by the already large movements. The inclinometer data confirmed the geodetically measured deformation rates and localized the movement to the heavily remoulded base lining (fig. 8). The obvious cause of the observed movements was thus identified as a failure of the base lining, establishing it as a slip surface and causing the western slope to change its balance towards a dramatically higher rate of deformation in the visco-plastic corresponding to type 3 of fig. 1.

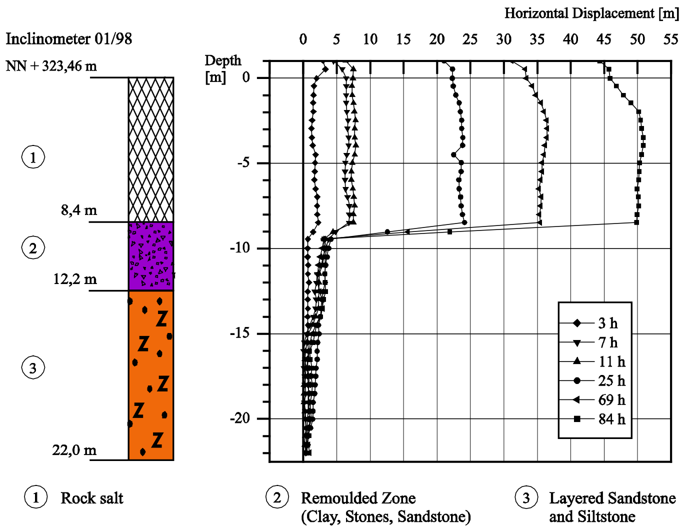

Fig. 8: Inclinometer Measurements

It is obviously impossible to assess the level of safety at that stage. Surely failure was close though and the remediation of the moving slope necessary. As precautional action therefore a large safety area at the foot of the slope was prohibited for public access.

2.2 Remediation and Monitoring

Several possibilities to remediate the heap slope were considered and evaluated. The conditions on the site were difficult due to enormous dimensions and the very steep slope. Furthermore no precedents of similar cases were known. Evaluating the various ideas to remediate the slope, the high risk of failure had to be taken into account; safety of all workers and structures surely had the highest priority.

Working on the slope was considered practically impossible (fig. 9). Due to the very high forces, supporting structures such as retaining walls, anchors and piles, as are known from moving slopes in mountainous regions (Brandl 1979), were considered not to be reliable enough, taking into account the highly aggressive milieu due to the dissolved salt in the brine. The lack of experiences with blasting in granular rock salt made the reduction of the height of the heap and thus increasing the load impossible.

The final remediation concept consisted of two elements:

- building a supporting heap at the base of the slope, using the ascending topography on the other side of the valley and
- exchanging the clay base lining by a material with higher strength and the same permeability in areas, where the slope had not covered it yet.

Taking into account the logistic problems connected with the transport of the enormous amount of material to be transported to the base of the slope and the minimization of personnel working in the endangered zone, the use of the heap infrastructure to install the supporting heap was chosen. The technique of the salt disposal from the heap top was adjusted, so that the salt was intended to come to rest right at the toe of the slope.

Fig. 9: View from the western slope

The remediation works were accompanied by an intensive monitoring programme and alarming system. In addition to the extensometers on top of the heap, two fully automatic geodetic tachymeters were installed, measuring app. 60 points on top of the heap and along the toe. Measuring data was sent online to the office to be evaluated. An automatic acoustic and visual alarm was installed to warn the personnel if the movement was accelerating more than allowed.

In search of an appropriate exchange material for the base lining several materials were tested in the geotechnical laboratories to meet the following requirements:

coefficient of permeability	$k \leq 1 \cdot 10^{-9}$ m/s
angle of friction	$\varphi' \geq 35$ °

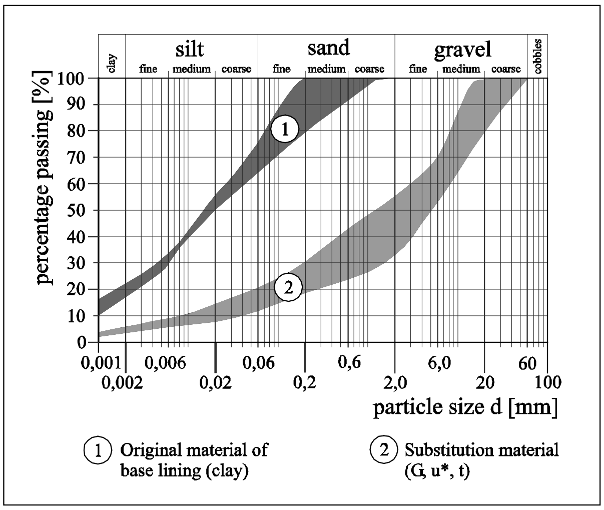

Fig. 10: Grain-size distribution of original base lining and substitution material of weathered basaltic rock "Steinerde"

Six months after the movements had been observed first, the basic clay lining was exchanged by a material of weathered basaltic rock called "Steinerde", which overexceeded the requirements. Its grain size distribution is close to the Fuller-Curve (fig. 11) with a reasonable part of silt and clay.

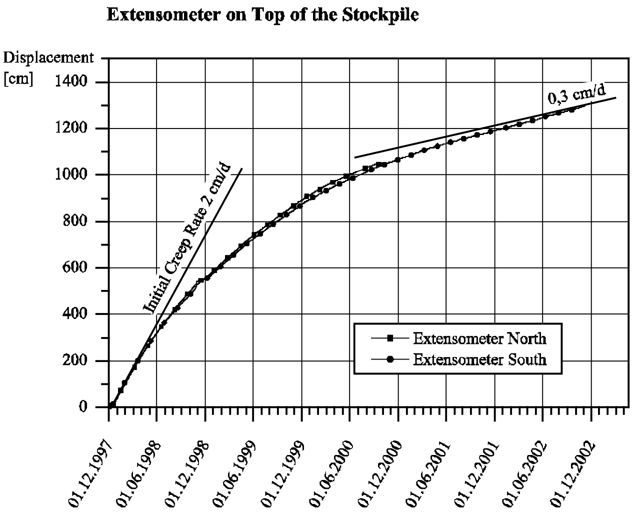

Fig. 11: Displacements measured with Extensometer

The chosen concept proved to be successful as the observed movements indicated by the extensometer show (fig. 12). Within 6 years the movement rates have decreased by the factor greater than 6 to 0,3 cm/d or approximately 1 m/a.

4 DEFORMATIONS OF A GAS-PIPELINE

4.1 Location and Problems

In the second case history the deformations of a high pressure gas pipeline are presented and discussed. The gas pipeline is situated within a distance of less than 25 m at the base of the slope of a heap a 190 m high. A picture of the heap is shown in fig. 13.

Fig. 13: View on the tailing heap

The base of the heap in figure 15 has the shape of a kidney. The length of the heap in southeast to northwest direction is about 1200 m. The width of the heap from northwest to southeast is 550 m. The gas-pipeline with a soil cover of 1 m and a diameter of 0.35 m passes along the northwest side of the heap (in red, fig. 14).

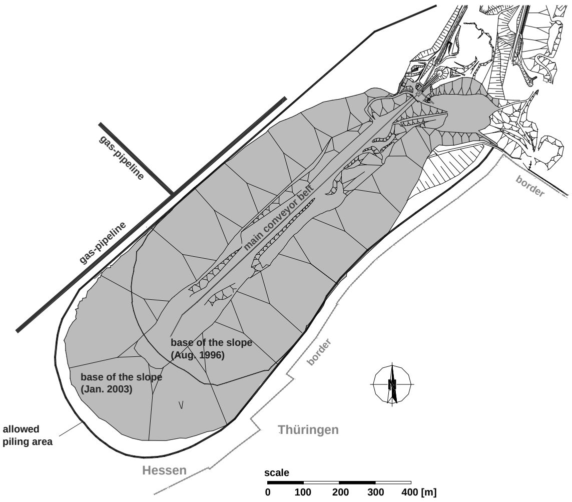

Fig. 14: Ground view of the heap

In the more detailed figure 15 the locations of the gas-pipeline and the measuring system is shown. According to the Observational Method the horizontal and vertical deformations of the gas pipeline are surveyed by several profiles of geodetic measuring points (profiles 1 to 7, fig. 15) and by inclinometers placed along the side of the tailing heap within a distance from 10 m up to 50 m (INK 1 to INK 10). The inclinometers have a depth of 30 m.

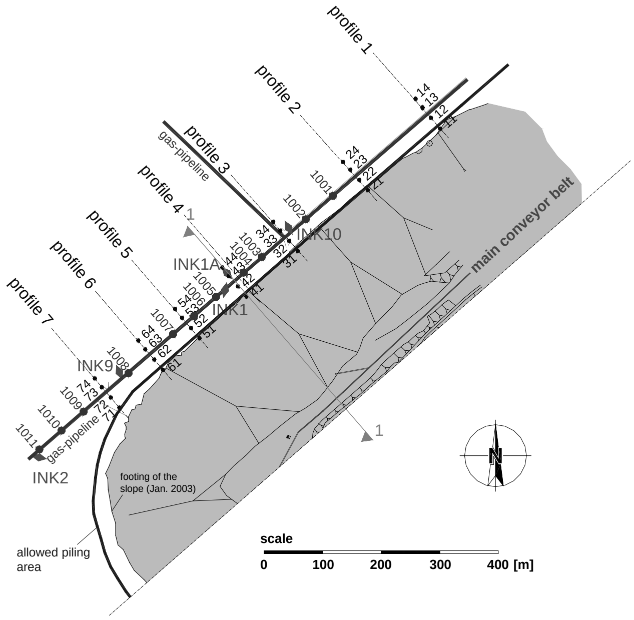

Fig. 15: Ground view (detail) with measuring system

Furthermore a profile of points in along the baseline of the slope is measured geodetically (points 1001 to 1019).

4.2 Deformations

The subsoil below the heap consists of quaternary loam (sand, silt and clay) down to a depth of 2 m. Under the quaternary follows the Bunter Sandstone with partly weathered layers and natural slip-surfaces in a depth of 5 m and 10 m.

Due to high loads caused by the waste in conjunction with natural slip-surfaces and weak zones respectively in the subsoil, large deformations, partly at accelerating deformation rates, occurred. The deformation may cause stability and serviceability problems of the gas pipeline in the long term.

In figure 16 some measuring results from the inclinometer INK 1 until the year 2002 are shown.

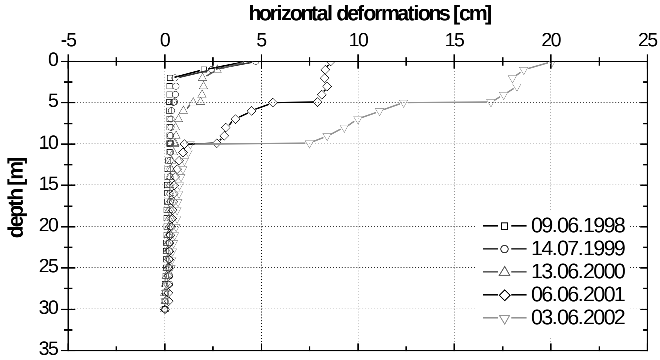

Fig. 16: Horizontal deformations measured in INK 1

The inclinometer shows the two natural slip surfaces in depths of 5 and 10 m.

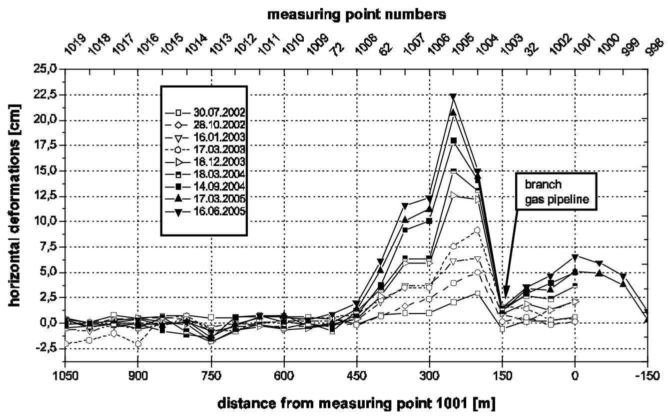

Fig. 17: Horizontal deformations along the baseline of the slope

Having a look at the horizontal deformations along the baseline of the slope in the points 1001 up to 1011 (fig. 17) the localization of the deformations can be noticed. The maximum of the differential deformations is noted between the points no. 1003 and 1004. The horizontal extension of the moving part of the subsoil parallel to the baseline of the slope can be estimated to approximately 300 m.

Taking into consideration all available data the observed movements have to be concluded as follows:

- The upper 5 to 10 m of the subsoil are moving at a relatively slow rate in direction NW, orthogonal to the baseline of the slope.
- The movements take place on two slip surfaces in the Bunter Sandstone.
- The movements are localized to an area of approx. 300 m along the baseline of the slope and to less than 100 m orthogonal to it.
- The deformation rates have reached a steady state at 10 cm/a.

4.3 Application of the Observational Method

With respect to the very high sensitivity of the gas pipeline to induced deformations a concept of safety as well as a monitoring and restoration concept has been developed. One component of the monitoring concept is the numerical simulation based on the results of the measuring programme to improve the prediction of deformations of the gas pipeline which has to be expected.

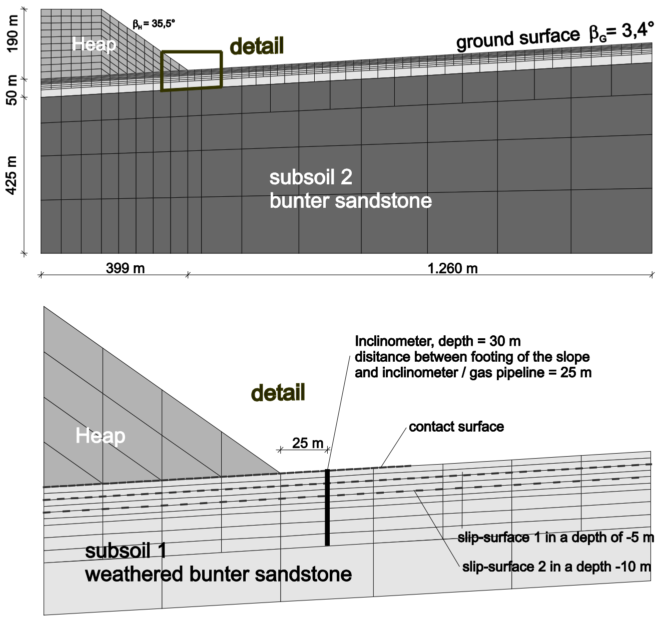

Fig. 18: Finite-Element-Model of the heap

The discretization of the Finite-Element-Model is shown in figure 18. Viscoplastic material behaviour for the salt has been considered using the constitutive equations of Boley (1999). Bunter Sandstone and weathered Bunter Sandstone are modelled using elasto-plastic material behaviour. The slip surfaces in a depth of 5 m and 10 m are modelled as contact pairs using the master-slave-concept to allow large deformations.

With the calibrated model numerical simulations for an improved forecast of the expected deformation have been executed. Fig. 19 shows the comparison between measured and calculated horizontal displacements in inclinometer Ink 1. The angles of friction φ_s in the slip-surfaces (S1, S2) have been back calculated. Both curves correspond well.

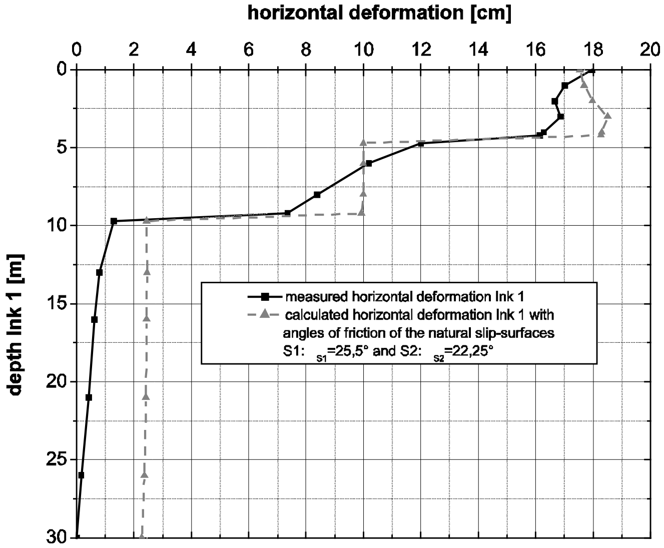

Fig. 19: Results of the back-analysis

With the calibrated and validated model a tool to predict the future deformations and to assess risk of failure is at hand. Along with the extensive surveying programme the induced deformations to the gas pipeline can be predicted. Thus a safe operation of the tailing heap and its adjacent structures is ensured in spite of the observed movements.

If the deformation of the gas pipeline reaches a critical range the following structural measures are provided:

- Uncovering the gas pipeline for stress relief and to reset the occurred elastic deformations.
- Installation of a bellow expansion joint at the branch line.
- Setting down the gas pipeline and ensuring the gas supply with an existing alternative pipeline system.
- Passing a new pipeline in a greater distance.

The critical deformation of the gas pipeline has been calculated against the distance of the measuring points. For this, the horizontal deformation must not exceed 60 cm within the range of 50 m (distance of two measuring points).

Up to now the maximum differential horizontal deformations between two measuring points (1003, 1004 s. fig. 17) reaches roughly 13,5 cm. To consider the deformations along the gas pipeline which not has been measured before July 2002 an additional differential horizontal deformation of 10 cm is add on the measured values. From this follows a maximum total differential horizontal deformation of 23,5 cm between the points 1003 and 1004. This value is underlying the evaluation of the serviceability of the gas pipeline. So nearly 40 % of the allowed maximum differential horizontal deformation of the gas pipeline within a distance of 50 m occurred right now.

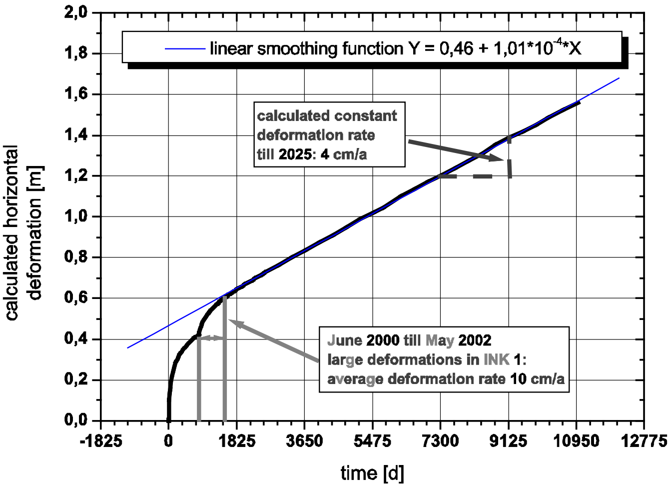

Fig. 20: calculated deformation forecast

In figure 20 the calculated time dependent deformations are shown. Based on the measured deformations between June 2000 and May 2002 in INK 1 (fig. 16) the deformations in this time step were calculated as an back analysis using the above mentioned angles of friction in the natural slip-surfaces. The calculated long term deformation rate is shown in figure 20 with an constant value of about 4 cm/a. This result fits quite well with the measured horizontal deformations in figure 17 where between March 2004 and March 2005 in point 1005 nearly 5 cm occurred.

Based on the numerical simulations and the measured deformations the gas pipeline will reach a critical range of deformations in 7 up to 9 years requiring a deformation rate of 4 to 5 cm/a.

REFERENCES

Beer, W. (1996): Kalilagerstätten in Deutschland. *Kali und Steinsalz* 12 (1), 18-30, Glückauf, Essen

Boley, C. (1999): *Untersuchungen zur Viskoplastizität und Festigkeit von Steinsalz*. Mitteilungen des Instituts und der Versuchsanstalt für Geotechnik der Technischen Universität Darmstadt (48), 1-236, Darmstadt

Brandl, H. (1979): Design of high, flexible retaining structures in steeply iclined instable slopes. *Proc. 7th ECSMFE* 3, 157-166, Brighton

Chumbe, D., Lloret, A., and Alonso, E. E. (1996): Creep and Permeability Tests on Compacted Granular Salt. The Mechanical Behaviour of Salt, *Proceedings of the Fourth Conference on the Mechanical Behaviour of Salt*, Montréal, 331-339, Aubertin, M. and Hardy, H. R. (Eds.), Trans Tech Publications, Claus-thal-Zellerfeld

Fordham, C. J. (1988): *Behavior of Granular Halite for Use as a Backfill in Potash Mines*. 1-181, Dissertation, University of Waterloo, Ontario, Canada

Katzenbach, R. und Hoffmann, H. (2003): Möglichkeiten und Grenzen der Identifikation und der Beherrschung von Massenbewegungen von Boden und Fels (Rutschungen). *Bauingenieur* 78 (7/8), 381-386, Springer, Heidelberg, Berlin

Munson, D. E. and Wawersik, W. R. (1991): Constitutive Modelling of Salt Behaviour - State of the Technology. *Proc. 7th Int. Congress on Rock Mechanics*, Workshop on Rock Salt Mechanics, 1797-1810, A.A.Balkema, Rotterdam