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Experimental investigation and theoretical modelling of soft soils from mining deposits

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ABSTRACT: Overburden soils from open pit mines of the company RWE Power AG are excavated with bucket wheels and transported with band conveyors to dump heaps. The deposition of the waste soils takes place within so-called regular sections. In order to perform stability computations one needs a geotechnical characterization of the involved soils, and in particular representative values of shear strength. Undrained shear strengths from laboratory and field testing are subjected to enormous scatter which makes the definition of the representative values difficult. Therefore, a concept for a reliable determination of the undrained shear strength of waste soils in mine heaps has been introduced. It is based on a hypoplastic constitutive model for fine-grained soils which takes into account the influence of soil state (in this case represented by consistency) on the undrained shear strength. The accompanying experimental investigations include extensive laboratory testing as well as large-scale field tests suitable for a statistical evaluation. The results of numerical calculations yield an experimentally justified statistical distribution of undrained shear strengths immediately after the soil deposition. A subsequent FE analysis of the consolidation behaviour provides the time-dependent values.

1. INTRODUCTION

Overburden soils from open pit mines of the company RWE Power AG are excavated with bucket wheels and transported with band conveyors to dump heaps. Several different materials are separated during this process. After extraction of gravel, sand and loess, two main soil classes are distinguished: M1 – soil mixture with fines content below 30% and M2 – soil mixture with fines content above 30%. With respect to soil consistency, the material class M2 is further separated into M2n (wet, with negligible angle of repose) and M2t (dry, with angle of repose greater than zero). The separation is performed by operators in the open pit mine and is based mainly on a visual judgement. The deposition of the M2 materials takes place in stages within heap regions defined by so-called regular sections (Pierschke 1995), see Figure 1. A realistic and theoretically sound determination of shear strength of the deposited soils is needed in order to judge the slope stability of the regular sections.

Soil is a not a material with constant mechanical properties. The latter depend on the soil state and are variable within certain limits. Thus, the distinction of the material classes based on granulometry must also take into account the soil state. In the geotechnical practice, e.g. in road construction or in landfill liners, soils can be treated in a way to fulfil the requirements. In case of the dump heaps is such an approach with a continuous adaptation of the deposited soils not applicable due to technological constraints of the mine operation.

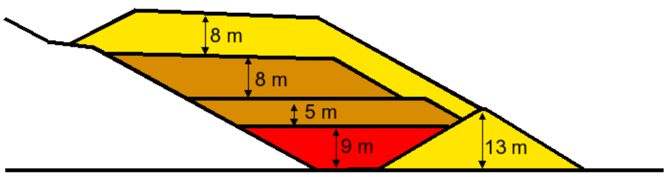

Figure 1. Example of a regular section

The Critical State Soil Mechanics (Schofield and Wroth 1968) introduced a concept of the state-dependent shear strength of soils. The crucial impact factors are the effective stress state and void ratio. For water-saturated soils the latter can be replaced by water content. Depending on boundary and loading conditions (drained or undrained) different shear resistances are obtained. They can be calculated from the soil state using a few clearly defined material paramaters. Time after deposition affects the changes of shear strength as well and, thus, is also important for the determination of slope stability.

In other works, relationships between compressibility and shear strength were investigated, yielding various useful correlations with index properties. In case of a hypoplastic model it was demonstrated for coarse grained soils (Herle and Gudehus 1999) that the parameters of the constitutive model can be estimated from granulometric properties or determined from simple laboratory experiments. A similar procedure is possible and meaningful also for fine grained soils.

In conventional geotechnics, two important parameters are used for the characterization of fine grained soils. Plasticity index I_p describes the soil sensitivity to changes of water content and soil consistency I_c the actual state (from liquid to solid). The soil saturation can change not only during the transport but also after the deposition. Consequently, water content and its changes with time (saturation and consolidation) play the crucial role in the evaluation of the slope stability of dump heaps.

Undrained shear strength c_u is primarly a function of void ratio and stress. Since void ratio in situ is difficult to measure, one can consider water content w and saturation ratio S_r instead. It follows from the Critical State Soil Mechanics that undrained shear strength depends also on compressibility of the soil. For the case $S_r = 1$, undrained shear strength c_u increases exponentially with water content.

2. SOIL INVESTIGATIONS AND DETERMINATION OF CONSTITUTIVE PARAMETERS

A significant scatter of undrained shear strengths in upper layers of a dump heap is depicted in Figure 2. The laboratory investigations were performed on undisturbed samples of the M2n material class from core drillings. The results provided information on the soil state after the deposition, in particular on density, saturation ratio and undrained shear strength c_u from unconsolidated undrained (UU) triaxial tests. The measured saturation ratio was between 0.75 and 1.0, although most samples of the material class M2n were practically fully saturated.

In Figure 2 one cannot recognize any trend with the depth which is usually typical for natural in situ conditions. It proves that the tested soil has not been consolidated yet after the deposition. Nevertheless, the results of the UU tests with the material from the core drillings manifest a substantially higher undrained shear strength than the

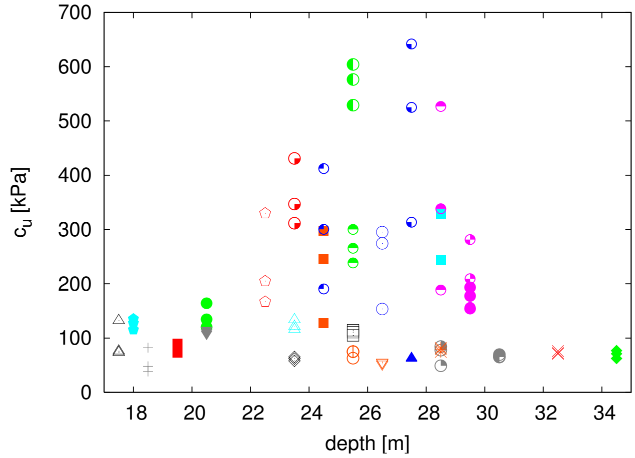

Figure 2. Undrained shear strengths of undisturbed samples from the material class M2n in dependence on depth

one of soils prior to dumping. In both cases the soils were reconstituted by transport on the band conveyors. A conventional averaging used often for the determination of a representative value of shear strength would result in large uncertainties since many measured values are below the average. In order to avoid a subjective selection of the shear strength values, a research programme was undertaken at the open pit mines of RWE Power AG in order to determine objective and representative values of the shear strength for slope stability calculations.

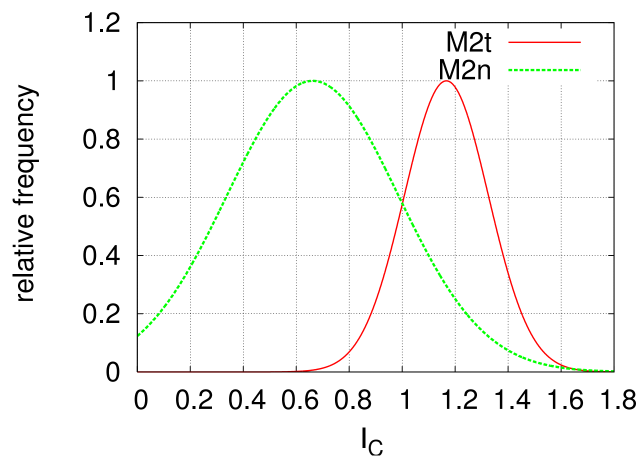

Figure 3. Normalized frequency distributions of the consistency index I_c for the material classes M2n and M2t

The first evaluation of the mechanical soil properties took place with data from investigations of the material collected directly from the band conveyors. It was proved that the distinction of the dumped material into the classes is based solely on the soil consistency. Subsequent investigations of a sufficiently large set of samples in the soil mechanics laboratory provided more precise parameters of the M2 material classes (mainly grain size distribution, water content and consistency index). For each class, these parameters were within a certain bandwidth which could be described with a frequency distribution. In a first approximation, the normal distribution defined through a mean value and a standard deviation was considered (Figure 3).

The classification tests also showed that the material class M2n is composed predominantly from a silt-sand mixture (Figure 4) whereas all possible grain sizes can be found in the material class M2t (Figure 5).

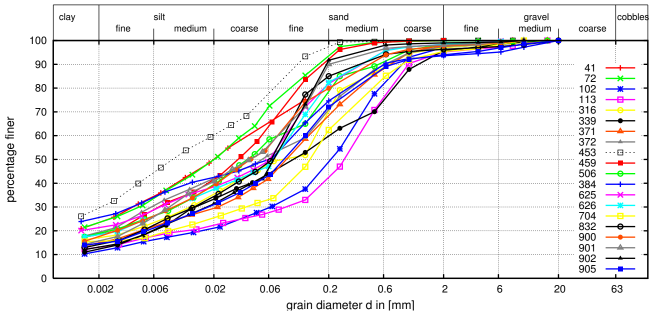

Figure 4. Grain size distributions of the samples from the material class M2n

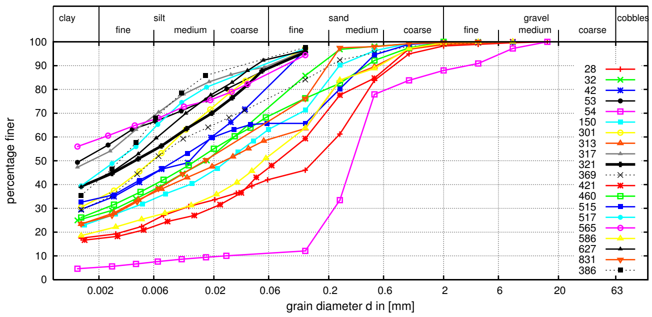

Figure 5. Grain size distributions of the samples from the material class M2t

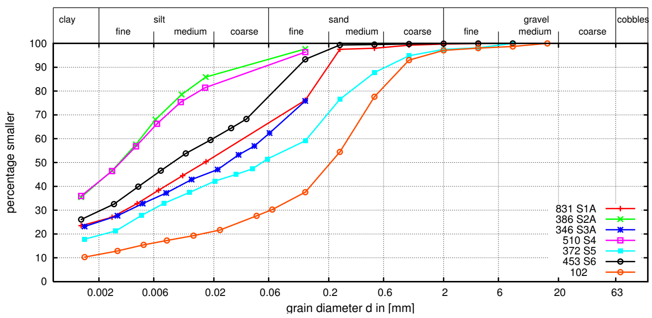

Figure 6. Grain size distributions of the samples investigated in detail

Seven representative samples from the band conveyors, covering the wide range of the grain distributions curves, were selected for the investigations of the mechanical properties of the dumped soils (cf. Figure 6). Using oedometer (Figure 7) and triaxial (Fig. 8) experiments, the parameters of the hypoplastic constitutive model (Mašín 2005) were determined. The suitability of the constitutive model for the description of the investigated material was validated by simulations of undrained traixial tests, see Figure 9.

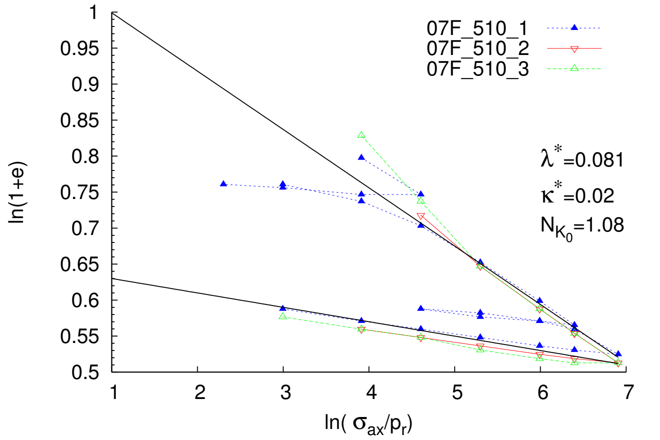

Figure 7. Example of oedometer tests on specimens of the reconstituted M2 material used for the determination of the hypoplastic constitutive parameters ($p_r = 1$ kPa is a reference pressure)

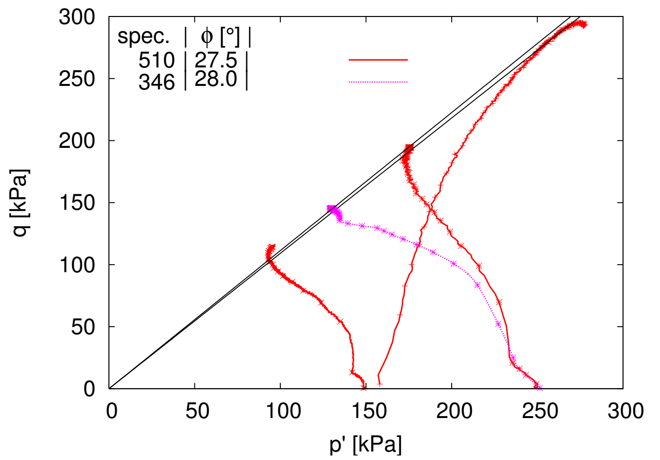

Figure 8. Stress paths from undrained triaxial tests on specimens of the reconstituted M2 material used for the determination of critical friction angle

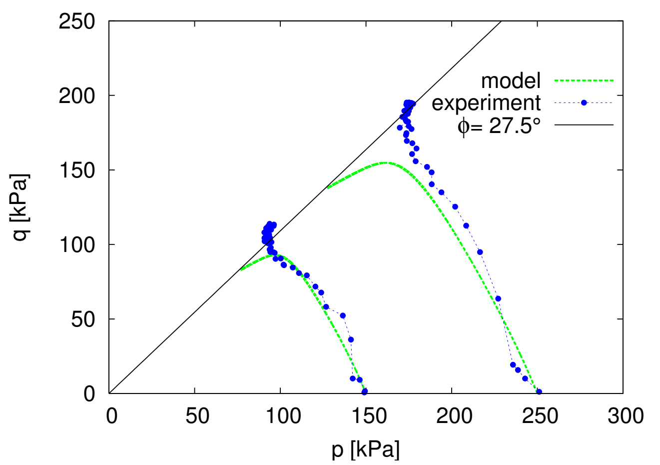

Figure 9. Validation of the parameters of the hypoplastic constitutive model in simulations of the CU triaxial element tests

The undrained shear strength of fine grained soils depends distinctly on water content and thus on consistency index I_c. Assuming a high degree of saturation S_r during the dumping (which was later confirmed), the undrained shear strength can be calculated from the hypoplastic constitutive model as a function of water content w. The results of these calculations for six representative samples are shown in Figure 10 as relationships between the c_u-values and the logarithmic consistency index

$$I_{clog} = \frac{\log w_L - \log w}{\log w_L - \log w_P}$$

(dashed curves). The c_u-values measured in the CU triaxial tests are depicted as points for comparison. A regression line of the calculated curves is shown as a full line. This line approximates the calculated curves well in the range $I_{clog} < 1$. For $I_{clog} > 1$ the linear approximation becomes less appropriate, i.e. in case of firm consistency the undrained shear strength should be preferably calculated from the constitutive model directly.

From a further sampling by core drilling in the M2t zones of the regular sections a number of undisturbed soil samples was additionally obtained and tested in unconsolidated undrained (UU) triaxial tests. The measured relationship between the undrained shear strength c_u and the logarithmic consistency index I_{clog} fits very well to the curves predicted by the hypoplastic constitutive model, see Figure 11.

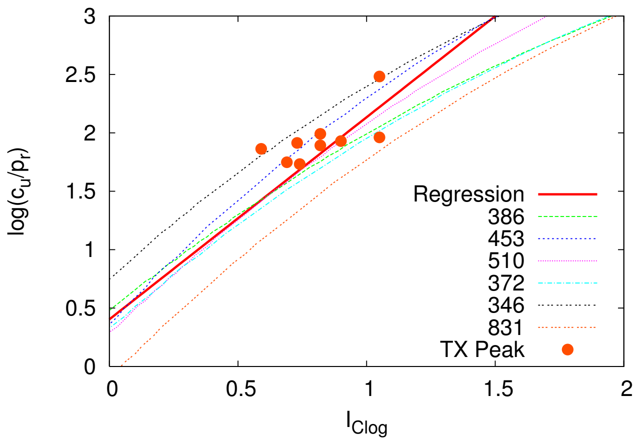

Figure 10. Calculated relationship between c_u and I_{clog} for normally consolidated soils ($p_r = 1$ kPa is a reference stress). Points represent the measured c_u-values of the reconstituted samples.

The grain size distribution curves of the samples investigated in detail cover the range of grain sizes for both, M2n and M2t material classes (Figure 6). Since the calculated relationship between c_u and I_{clog} is at closest to the regression line for the specimen No. 510 (see Figure 10), the hypoplastic parameters of the specimen No. 510 were considered as a representative parameter set for both material classes M2n and M2t. The classes differ solely by

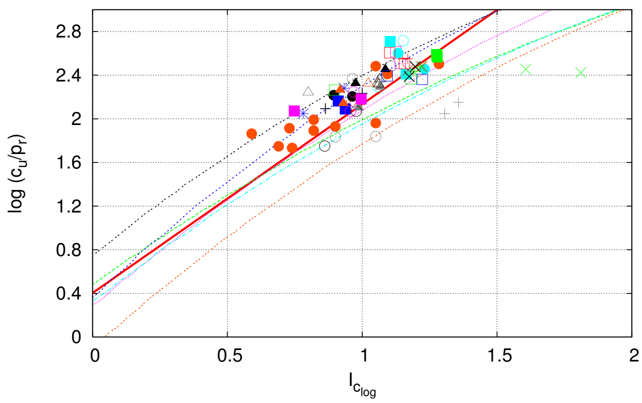

Figure 11. Calculated relationship between c_u and I_{clog} for normally consolidated soil (lines) compared additionaly with experimental values from UU testing on the undisturbed samples

soil consistency, whereas the parameters of the hypoplastic constitutive model are state-independent.

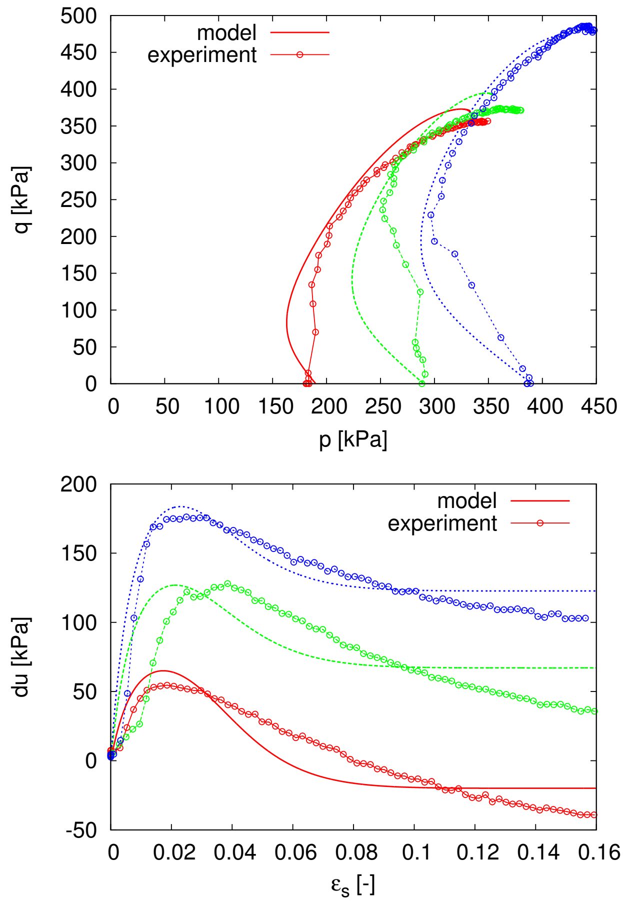

Figure 12. Calculation of the CU triaxial element tests for samples of the material class M2t using independently determined hypoplastic parameters (stress paths and evolution of excess pore water pressures)

The selected representative parameters of the hypoplastic model were also checked in element test calculations for several undisturbed samples from the material class M2t. It can be observed in Figure 12 that the material behaviour in consolidated undrained triaxial tests can be reproduced by the constitutive model very well. The assumption of a single representative set of the hypoplastic parameters for both material classes M2n and M2t was thus justified.

3. REPRESENTATIVE SHEAR STRENGTH

As shown in Figure 3, the normal frequency distribution of the consistency index was considered for both material classes M2n and M2t. Although these distributions were obtained for the samples from the band conveyors, they are also characteristic for the dumped soil as can be seen in Figure 13 for undrained samples from the core drillings.

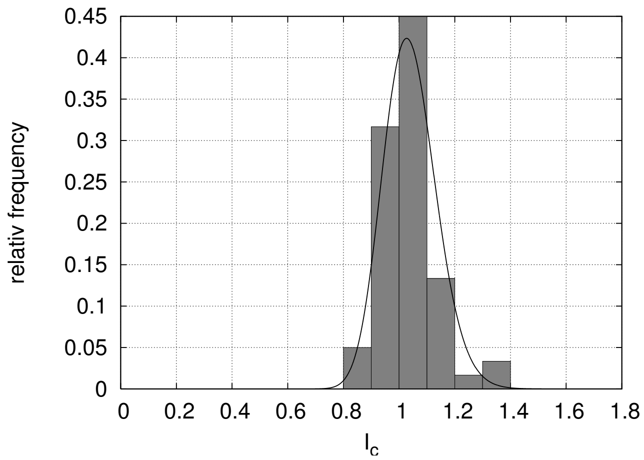

Figure 13. Relative frequency of the consistency index of undisturbed samples of the M2t material class

Since the undrained shear strength of normally consolidated soils is a function of water content (i.e. also of consistency), the calculated c_u values must be also statistically distributed. Due to the nature of the relationship between c_u and I_c, the frequency distribution of the undrained shear strength is logarithmic. Using the regression line from Figure 10 and the frequency distribution of I_c from Figure 3, the frequency distribution of c_u for the material classes M2 can be calculated, see Figure 14 for the M2n material.

One can define several characteristic values for such a log-normal distribution. The mean value corresponds to the average value (arithmetic mean) used also in a normal distribution, i.e. all values are summed and divided by the number of values. The median value is the middle value that separates the greater and lesser halves of the data set. The mode value is the most frequent value in the data set.

Let us consider a hypothetical example of the statistical evaluation of the results of 33 specimens from unconsolidated undrained triaxial tests. The measured c_u-values have been assigned to the intervals with a magnitude of 10 kPa (from 5 to 15 kPa, from 15 to 25 kPa etc), see Table 1. The frequency distribution of the c_u-values is depicted in Figure 15.

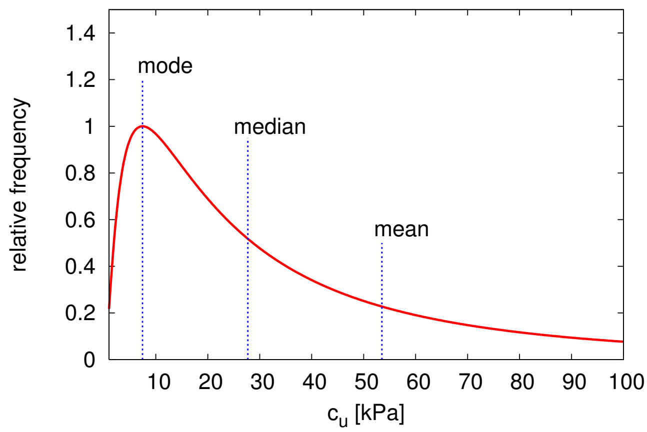

Figure 14. Calculated frequency distribution of the undrained shear strength of the M2n material class

Table 1. Example of measured c_u-values conforming to log-normal distribution

c_u kPa	10	20	30	40	50	60	70	80	90	100
freq.	10	6	5	3	3	2	1	1	1	1

The most frequent value (mode) is 10 kPa, which was measured in 10 cases. Regarding the total number of 33 specimens and the mean (average) c_u-value of 33 kPa, it would be extremely conservative to consider $c_u = 10$ kPa as a representative value. Approximately one half of the specimens yields $c_u > 20$ kPa which suggests that the selection of the median value would be more representative.

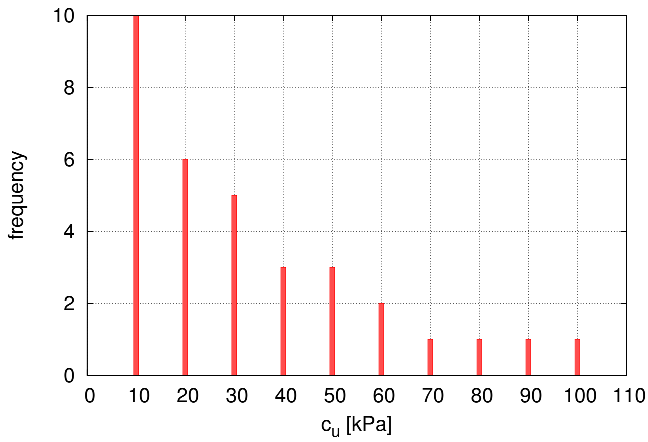

Figure 15. Frequency distribution of the values from Table 1

The frequency distribution of I_c for the material class M2t (see Figures 3 and 13) together with the regression line in Figure 10 can also be used for the estimation of the c_u-distribution for the material class M2t. This yields the mode value of 65 kPa, the median value of 154 kPa and the mean value of 237 kPa. A comparison with an experimentally obtained log-normal distribution of the measured c_u-values (Figure 16) suggests that the calculation results are conservative. However, one has to bear in mind that the tested samples were obtained from the heap several weeks after the soil deposition whereas the calculation is based on the behaviour of the reconstituted material typical for band conveyors. Moreover, the soils with higher I_c-values are usually less saturated which was not taken into account in the calculations.

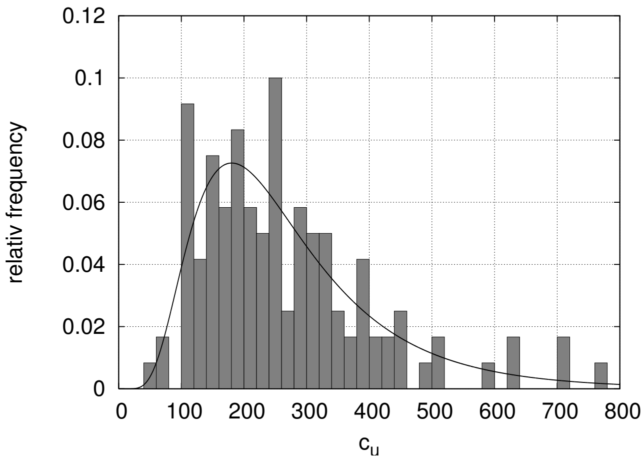

Figure 16. Frequency distribution of the measured undrained shear strength of the material class M2t

Consequently, it was decided to consider the median to be the most suitable c_u-value for the deterministic calculations of slope stability. The selection of the median as a representative value for deterministic analyses implies that the real c_u-value is located with the same probability below and above it, respectively. Taking into account that a potential sliding mechanism includes a large volume of the dumped material, the selection of such a representative value seems to be appropriate.

4. TIME EVOLUTION OF SHEAR STRENGTH

The undrained shear strength of a freshly dumped material does not remain constant but increases due to consolidation process with time. A rigorous consideration of the time evolution of the shear strength in case of mining deposits is difficult because the consolidation does not take place under constant load. The geometrical configuration of the dump heaps evolves within space and time which poses extraordinary requirements on numerical analyses and applied models.

Moreover, for the consolidation analysis the soil permeability plays an important role. Therefore, for the determination of the permeability coefficient not only the time-settlement behaviour of the oedoemeter specimens was considered, but additionally in situ permeability (infiltration) tests were performed. The laboratory experiments revealed also a strong stress-dependence of permeability.

The time-dependent compressibility of several soil layers was first studied in a simplified 1-D finite element model. It produced the change of void ratio of the dumped material with time. Then, analogously to the already described cases, using void ratio it was possible to calculate the undrained shear strength. Since the degree of consolidation changed depth, a volume averaging had to be performed for soil layers.

The predicted time evolution of the undrained shear strength is depicted in Figure 17. It can be clearly observed that the initial log-normal frequency distribution becomes skewed with time. The smaller c_u-values increase faster than the higher ones which changes the character of the frequency distribution.

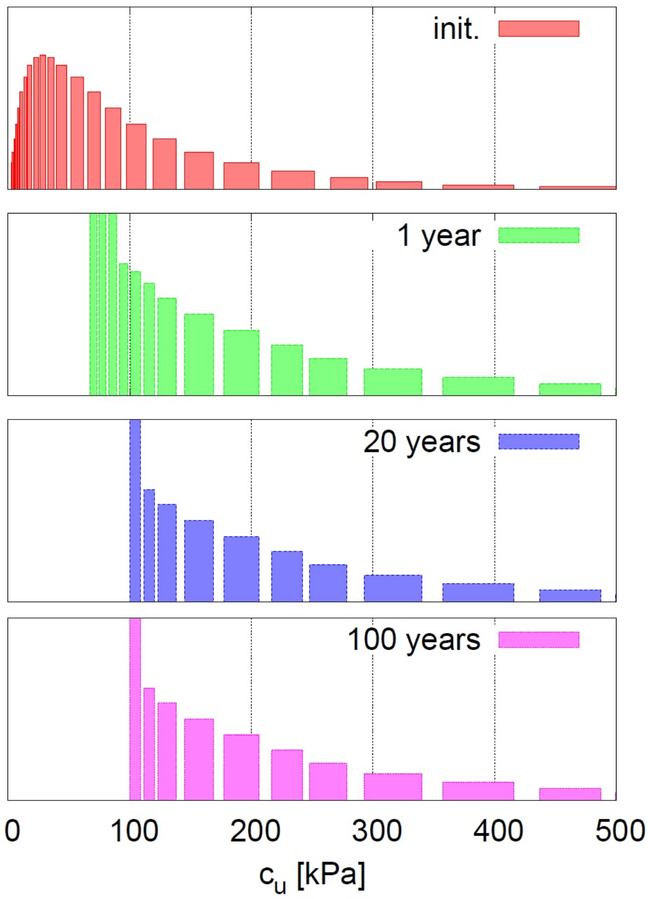

Figure 17. Time evolution of the frequency distribution of the undrained shear strength for the material class M2n (qualitatively)

Finally, the overall behaviour was studied in a two-dimensional finite element model of a regular profile. The simulation results were compared with deformations measured in situ (using hydrostatic surveying). The agreement between the numerical simulations and the measurements was satisfactory and confirmed the applied model of the consolidation behaviour.

5. CONCLUSIONS

The presented approach for the determination of undrained shear strength is based on fundamental concepts of soil mechanics which enable an objective selection of c_u. The undrained shear strength is not a soil constant but a variable changing with time and depending on the actual soil state. The hypoplastic constitutive model was calibrated from standard laboratory tests and its predictive capabilities were validated by numerous experimental results from undrained triaxial tests.

A scatter of soil properties and states is inevitably reflected in a scatter of undrained shear strengths. It has been shown that the c_u-values of a freshly dumped soil of the material class M2 are distributed along a log-normal frequency distribution which becomes skewed with time. It has been proposed to use a median value as a representative value of the frequency distribution, in particular for the studied case of mining deposits.

6. REFERENCES

REFERENCES

Herle, I. and G. Gudehus (1999). Determination of parameters of a hypoplastic constitutive model from properties of grain assemblies. *Mechanics of Cohesive-Frictional Materials 4*(5), 461–486

Mašín, D. (2005). A hypoplastic constitutive model for clays. *International Journal for Numerical and Analytical Methods in Geomechanics 29*(4), 311–336

Pierschke, K.-J. (1995). Standfestigkeit bei der Verkippung von nicht aufbaufähigen Mischböden im rheinischen Braunkohlerevier. *Braunkohle* (12), 5–12

Schofield, A. and C. Wroth (1968). *Critical state soil mechanics.* London: McGraw-Hill