Remediation of the Uranium Mill Tailings Pond Dänkritz 2: Geotechnical Analyses

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ABSTRACT

The remediation of the East German uranium mining sites operated and used until 1990 started in 1991 and was funded by the German Federal Government. However, most of the numerous abandoned uranium mining sites of third party property that were distributed all over Saxony were not included. In 2003 the Federal Government and the Free State of Saxony agreed to jointly fund the remediation of those abandoned mining sites of high priority. Wismut´s department for Project Management of Wismut´s Abandoned Saxonian Mining Sites was given the responsibility for the project management. The Dänkritz 2 uranium mill tailings pond is one of those abandoned sites. Mill tailings were discharged during the 1950s surrounded by a max. 12 m high autostable ring dam, which was already reshaped during the 1960s to improve geotechnical stability. Dynamic stability calculations, applying the finite element program PLAXIS, were carried out by CDM Smith as part of the remediation design. They resulted in insufficient dynamic stability with respect to the maximum credible earthquake. In addition, soft fine-grained tailings needed to be stabilized to place a multi-layer type surface cover. Wismut designed the tailings remediation project. The paper will particularly focus on the advanced constitutive models used for the slope stability analyses of the embankment under dynamic loads and of the stability of the fine-grained tailings during interim cover placement.

INTRODUCTION

In Eastern Germany uranium mining lasted from 1945 until 1990. In total ca. 216,300 t of uranium were produced. The remediation of the uranium mining sites, operated or used until 1990 by the Soviet-German Wismut company, started in 1991 under responsibility of the newly founded Wismut GmbH funded by the German Federal Government. However, most of the numerous abandoned uranium mining sites of third party property, distributed all over Saxony, were not included. In 2003 the Federal Government and the Free State of Saxony agreed to jointly fund the remediation of those abandoned mining sites of high priority. Wismut´s department for Project Management of Wismut´s Abandoned Saxonian Mining Sites was given the responsibility for the project management. The Dänkritz 2 uranium mill tailings pond is one of those abandoned sites of high priority. The radiological, environmental and geotechnical site assessment proved the need for site remediation. Wismut designed the tailings remediation project and applied for the respective planning approval in 2015. The remediation includes water collection and treatment during remediation, interim covering of sandy and soft fine-grained tailings, dam reshaping, re-contouring of the pond area enhancing tailings consolidation by using vertical drains, placement of a multi-layer type final cover, and revegetation.

Geotechnical analyses where carried out by CDM Smith as part of the remediation design with the following aim:

- Prove stability of the ring dam (also taking into consideration earthquake loading).

- Investigate settlements due to consolidation of the tailings, under the load of the overlying re-contour layer and surface cover, to ensure the functionality for the planned runoff diversion system in finale state.

- Prove stability of the interim cover on the fine-grained tailings during placement under the load of the construction equipment.

SITE CHARACTERISATION

The uranium mill tailings pond, Dänkritz 2, is located in Germany, approximately 8 km northwest of the city center of Zwickau. The pond has an extension of ca. 250 m from east to west and ca. 400 m from north to south. Its area represents approximately 10.5 ha. The tailings pond is located in a former gravel quarry. Between 1955 und 1958 uranium mill tailings from soda-alkaline leaching were discharged into the abandoned open pit. Once the discharge of the tailings exceeded the pit capacity, the coarser particles were piled up to a surrounding ring dam. The heights of this ring dam range between 2 m and a maximum of 14 m. Slopes of the ring dam show a gradient between 1:2.5 and 1:3 (vertical to horizontal). The basin surrounded by the ring dam is today filled with tailings at thicknesses between < 2 m and 12 m. In total about 820,000 m³ tailings are currently contained in the impoundment, which is partly covered with a supernatant water layer up to a depth of 2 m. Figure 1 shows an aerial view of the Dänkritz 2 tailings pond in 2015.

The geogenic subsurface strata of the tailings pond Dänkritz 2 consists of the Upper Permian, bedrock which has a weathered zone of approximately 30 m thickness. The upper 2 m of the weathered zone includes 10 to 15% impermeable kaolinite. The Upper Permian is covered by tertiary gravels. On the eastern and southern side of the tailings pond a silt layer with a thickness of 3 m is enclosed in these gravels. The tertiary gravels were extracted during the working process of the gravel quarry so that there are just 2 to 10 m left.

Figure 1 Areal view on the Dänkritz 2 tailings pond in 2015 (view from north-west to south-east)

STABILITY FOR THE RING DAM

The seismic design has to be carried out according to the European standard Eurocode 8. Furthermore, for the seismic design of remediation ponds as sedimentation basins the German standard for the design of dam structures (DIN 19700-15) has to be considered. The representative safe-shutdown earthquake was derived in an expert seismological opinion for the Wismut locations from an analysis of historical events in the immediate vicinity of these locations and it corresponds to the progress of an earthquake that occurred in the North Italian region of Friuli in May 1976.

In order to analyze slope stability under the influence of earthquakes fully dynamic calculations were carried out. This method allows to extract acceleration curves (accelerograms) from expert seismological opinions, representative for the location. The calculations were completed applying a horizontal and a vertical acceleration of the safe-shutdown earthquake as a seismic input motion corresponding to the accelerogram on the base of a Finite Element Model. The calculations were repeated while reducing the parameters $\tan\phi'$ and c' until a continuous sliding surface, characterizing a slope failure, could be observed by analyzing plastic failure points. The factor of safety for the slope was determined by the ratio of shear strength, at this accordingly observed failure, to the available

shear strength using Fellenius's law (Fellenius 1936). The slope stability was analyzed considering both, drained and undrained conditions.

Figure 2 shows the numerical model for the slope on the eastern side of the ring dam. It is an extract of the numerical cross section 1 – 1', which is about 540 m long. In the numerical model the layer thickness of the underground is about 30 m. The bottom of the model implies the bedrock. The lower model boundary was coupled with a prescribed displacement on which the motion signal corresponding to the accelerogram was applied.
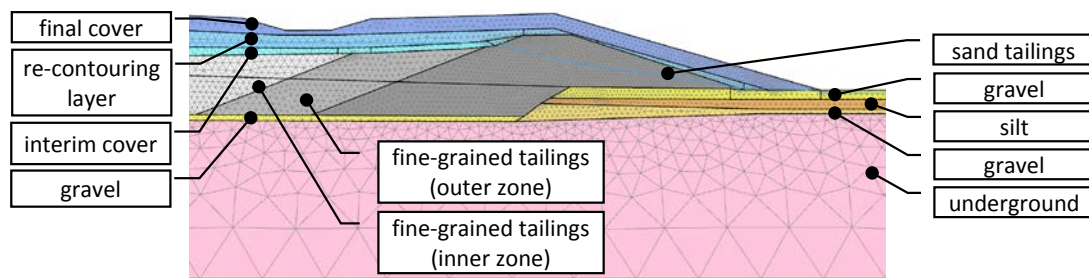

Figure 2 Extract of the numerical cross section 1 – 1', eastern side of the ring dam (showing the final state), not at scale

Soils have been modeled using the hardening soil model with small strain stiffness (HSsmall). The HSsmall model is based on the Hardening Soil model (Schanz 1999), extended with an elastic overlay model to take into account the high stiffness at small strain levels (Benz 2007; Benz et al. 2006). Brinkgreve et al. 2007 give an extended overview of the derivation of the hysteretic damping from the small-strain stiffness formulation in the HSsmall model. Here the dependency of the damping ratio from $\gamma_{0.7}$ and the G_0/G_{ur} ratio is shown due to varying $\gamma_{0.7}$ in a range of $1\cdot10^{-4}$ to $3\cdot10^{-4}$ and the G_0/G_{ur} ratio in a range of 2.5 to 10. Decreasing the value of $\gamma_{0.7}$ makes the material damping increase and even increases the ratio of G_0/G_{ur} (adequate to E_0/E_{ur}) and results in an increase of material damping.

In addition, according to Brinkgreve et al. (2007), Rayleigh-damping was postulated in the dynamic calculations while these damping parameters were set constant for all layers, which satisfied the requirements of simplification.

The history of the development of the tailings pond was modeled in order to generate the initial stress state. The ring dam did not show sufficient stability under seismic loading in this state. For this reason the slope of the dam was flattened and inter alia a multi-layer type surface cover was added to reduce the quantity of seepage water, which also had a positive impact on the stability of the slope. This multi-layer type surface cover consists from bottom to top of a 0.5 m thick sealing layer (hydraulic conductivity $k_f \leq 5\cdot10^{-9}$ m/s), an 0.3 m thick drainage layer (with a grain size distribution granting

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suffusion stability) and an 1.5 m thick storage and recultivation layer made of mixed-grained soil. Figure 2 shows an example of the potential failure mechanism in the static load case for the remediated southern slope of the ring dam of the Dänkritz 2 tailings pond. The calculated global factor of safety is η_{min} = 1.71. When applying horizontal and vertical acceleration of the safe-shutdown earthquake no continuous sliding surface could be observed by analyzing plastic failure points (refer Figure 4a) under provided shear strength. Due to reducing shear strength by 1.1 the calculation model showed an increase in failure points with a tendency towards a continuous sliding surface at the time step 6.6s (refer Figure 4b). Thus, sufficient stability of the ring dam, under seismic loading due to an earthquake, could be proven for the final state with a global factor of safety of η_{min} > 1.10.
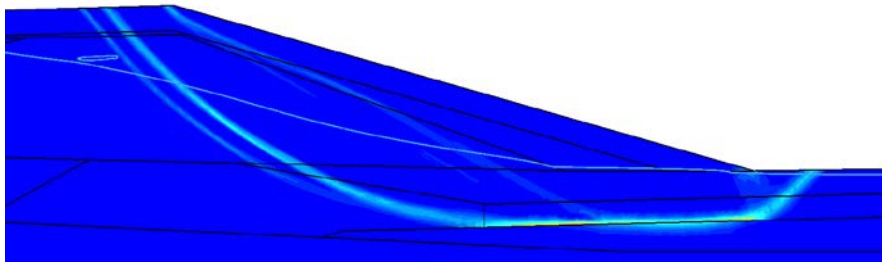

Figure 3 Concentration of deviatoric strain characterizing potential failure kinematics of the southern slope in the static load case (final state)
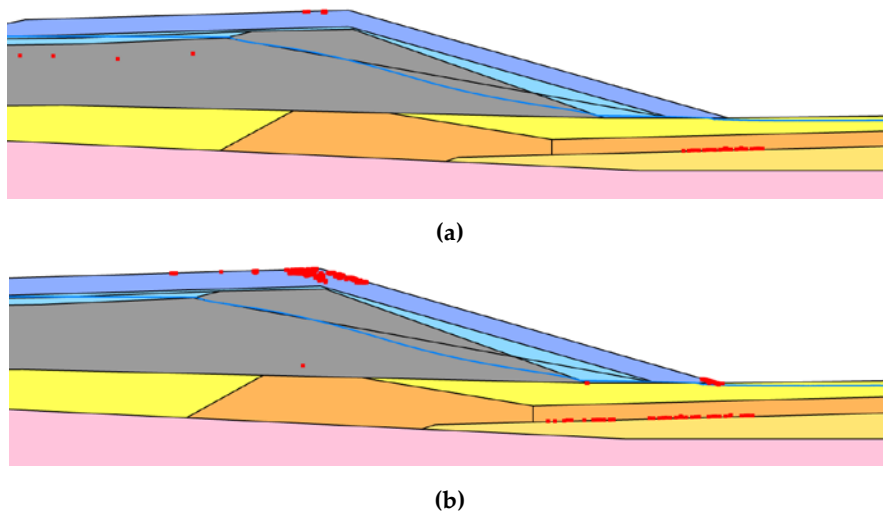

(a)

(b)

Figure 4 Development of plastic points due to reduction of shear strength (final state, crucial time step at 6.6 sec.): (a) plastic points due to seismic loading; (b) plastic points due to seismic loading, shear parameters reduced by 1.1

PLACEMENT OF A MULTI-LAYER TYPE FINAL COVER

Modeling of consolidation

The behavior of the soft fine-grained tailings is undrained. When the material is loaded by the multi-layer type surface cover an immediate increase in pore pressure occurs. This excess pore pressure dissipates as water drains from the material very slowly due to its low permeability. As the water drains from the material, change in volume occurs causing settlement (consolidation settlement). After placing the final cover layer with its water drainage system no more work for regulation shall be done. That means the cover has to be built with a higher level and a pre-defined contour that takes into account the settlement due to the consolidation process. While the interim cover has to be built from the ring dam towards the center of the pond, the re-contouring layers can be placed beginning from the center of the pond towards the ring dam, allowing the material in the center to consolidate more quickly. In addition, the dissipation of pore water pressure will be enhanced by means of vertical drains. The calculation has to consider all these conditions and the model should place Wismut into a position to design the re-contouring work in the right sequence and time, so that the final cover is correctly contoured and the water drainage system is working after completion of consolidation.

A three-dimensional finite element calculation using advanced constitutive soil models for the simulation of the non-linear and time-dependent behavior of tailings has been carried out. The placement of the layers has been modeled by splitting the sequence into calculation phases. The vertical drains and the geogrid have been modeled by means of structure elements. The vertical drains ended two meters above the bottom of the tailings to prevent drainage into the underground. The software PLAXIS was used to implement an undrained effective stress analysis, using effective strength parameters to model the material's undrained shear strength. The width of the modeled sections corresponds to the grid of the vertical drains using the symmetric conditions. Figure 5 is an illustration of a section from the calculation model.
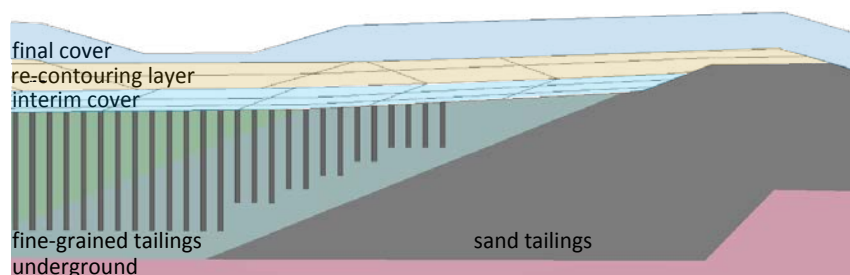

Figure 5 Section used for calculation (cutout — east side of Section 1 – 1')

The Soft Soil Model was used for this calculation. The model parameters have been derived from standard odometer tests conducted by Wismut. Based on of these tests Wismut provided the void-ratio-stress and the void ratio-permeability relations. Furthermore, the depth-dependent, undrained cohesion was measured along profiles by in-situ shear vane testing.

The surface settling process and consolidation tailings material due to discharge were verified analytically using a one-dimensional column model and a finite element column model to model the initial stress state of the tailings as close to reality as possible. The void ratio-depth profiles obtained were compared with one another in Figure 6 exemplary for the material of the inner zone of the fine-grained tailings. The void ratios measured in 2014, for depths down to 2m, are shown here as a reference. Finally, the tailings in the three-dimensional finite element model were divided into layers measuring one meter while the void ratio, the specific unit weight and the permeability were postulated in terms of depth here from the calculation of the initial state on the column model.
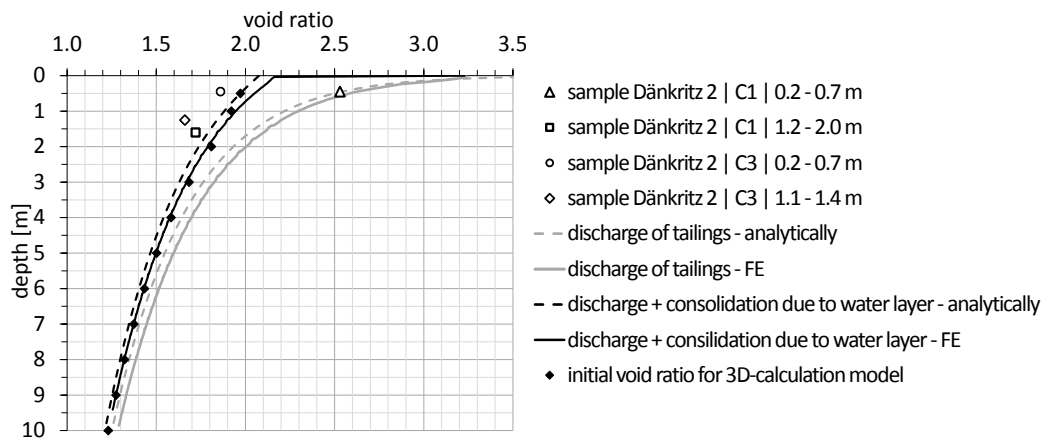

Figure 6 Void ratio depth profile of the inner zone of the fine-grained tailings due simulation of discharge of tailings

Consolidation was calculated taking into account the intermediate construction intervals (placement of interim cover, re-contouring layer and final cover). The modeled sequences followed this procedure with the aim to prove that the major portion of settlement due to consolidation will be completed at the end of the re-contouring work. This was essential for the strategy of the construction sequences and construction time.

With completion of the re-contouring layer a maximum settlement of 0.7 m was calculated in the center of the pond. The increase of contouring material for compensating this settlement was calculated at approximately 34,500 m³. The additional load of this material was considered in the calculation. Figure 7 shows the progression of the final cover surface after completing the consolidation exemplarily for the eastern side of the tailings pond. Position 0.0 m represents the center of the tailing pond. At position 90.0 m the ring dam begins. For the final layer a total settlement of 1.3 m was calculated at the time step approximately 21 months after completing final coverage. This settlement still ensured the functionality for the planned surface-runoff diversion system and for the drainage system.
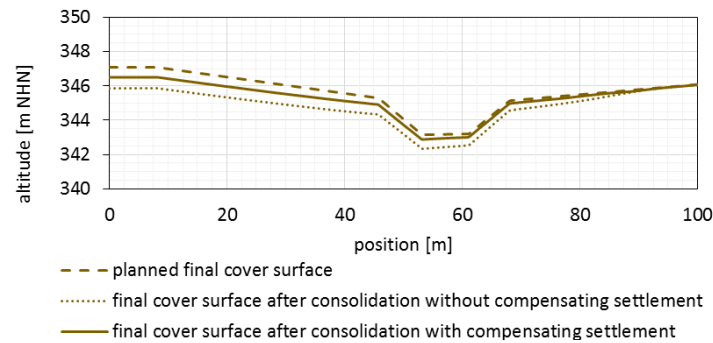
Figure 7 Progression of the final cover surface after completing the consolidation

Stability of the construction equipment

To ensure stability for the used construction equipment a geogrid will be laid out on the surface of the tailings. The aim of the stability analysis was to prove sufficient load-bearing capacity of the geogrid. This was done pursuant to Eurocode 7 for the ultimate limit state due to the failure of the ground while applying the finite-element method with factored loads. Stability is defined as proven if the finite element model converges in the calculation. The geogrid was designed for the tensile stress resulting in the calculation, which uses factored loads.

The stability of construction equipment on the interim cover was analyzed by using a three-dimensional finite element model (Figure 8), where the tailings were divided into layers measuring one meter while the void ratio, the specific unit weight and the permeability were postulated in terms of depth as calculated for the initial state in the column model. In addition, vertical drains were modeled with a triangular grid spacing of 1.25 m and a depth of 8 m including a geogrid on the surface of the tailings. The interim cover has been modeled with its three successively placed layers of thicknesses of 0.3 m (first layer), 0.5 m (second layer) and 0.7 m (third layer).
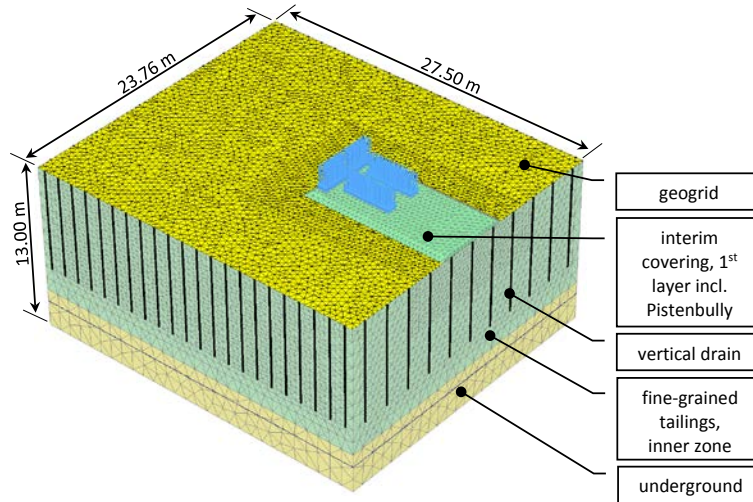
Figure 8 Calculation model for analyzing the stability of the construction equipment on interim cover

The Soft Soil Model parameters from the consolidation model were used (see Table 1 in Section 4.3). Figure 9 shows the distribution of the undrained shear strength resulting in the calculation model based on the effective strength parameters. It shows an excellent fit with the very low values measured due to in situ shear vane tests in the upper 3 m of the tailings, which are relevant for the stability.
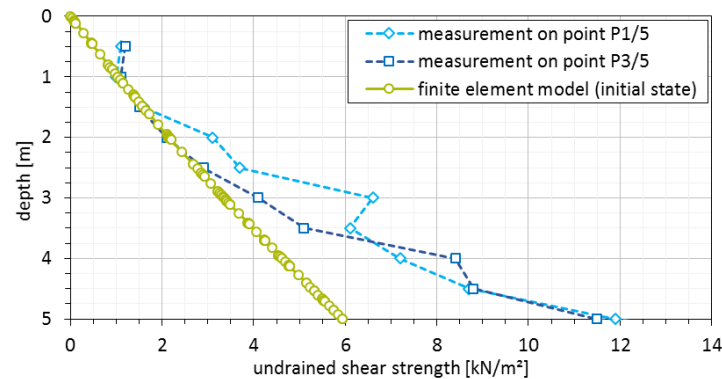

Figure 9 Progression of depth of the undrained shear strength in the fine tailings

An axial tensile stiffness of 700 kN/m was postulated in finite element modeling for the geogrid design on the surface of the tailings. This value is guided by the SS40 geogrid from Tensar International Limited (2003) that has a resistance to tension of 14 kN/m at a 2% strain in the longitudinal and transverse direction of the grid.

The analysis was carried out with the aim to prove the stability of a Pistenbully 300 (Moor caterpillar) and a CAT D3K2 chain dozer. In addition, a stack of soil material was placed in front of the

Pistenbully or Dozer considered to impose a surface load of 20 kN/m². Figure 10 shows the Pistenbully and the geometry postulated in the numerical calculation.
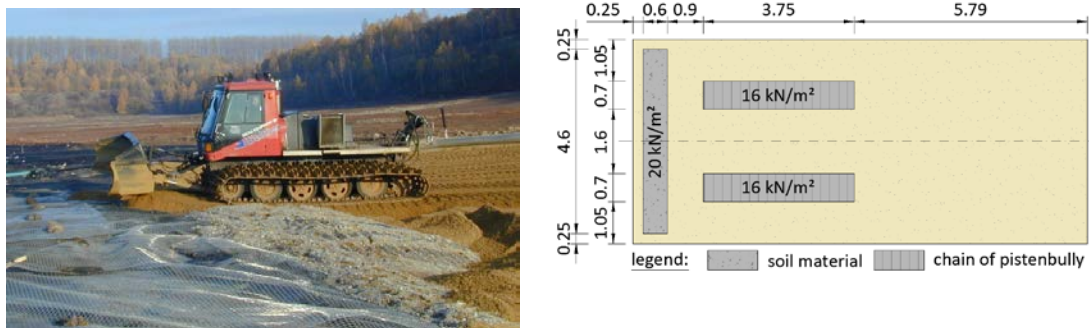

Figure 10 Pistenbully 300 and designed load

The assumption made in the calculations was that the entire geogrid and vertical drains had been placed at the beginning of building the 1st layer of interim cover material. Then, the 1st layer was consolidated before building the 2nd layer, which was included in the calculation with a consolidation time of 60 days. The finished 2nd layer of the interim cover was also consolidated for 60 days in the calculation before building the 3rd layer. The illustration of the excess pore water pressure shown in Figure 11 indicates the effectiveness of vertical drains during building up the 3rd layer of the interim cover.
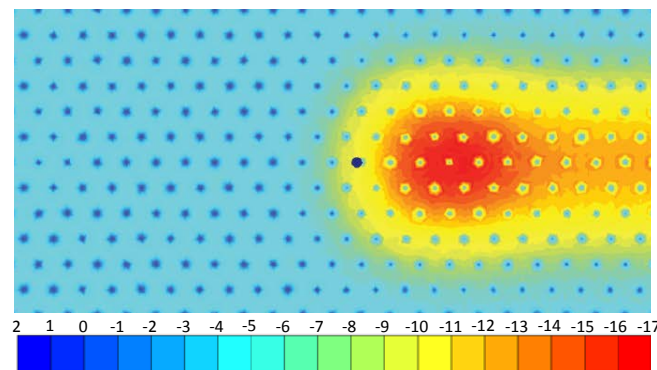

Figure 11 Excess pore water pressure in kN/m² during building up the 3rd layer of the interim cover
(horizontal cross section at 1 m depth below tailing surface)

As expected, the maximum tensile stress N_d in the geogrid resulted from the build-up of the 1st layer of the interim cover ($N_d \approx 11$ kN/m). The ground deformation showed a failure behavior at the 1st layer of the interim cover, but the calculation converged under factored loads and the analysis of the deviatoric strain (Figure 12) indicated reserves in resistance. The safety of equipment of the equipment could be proven.
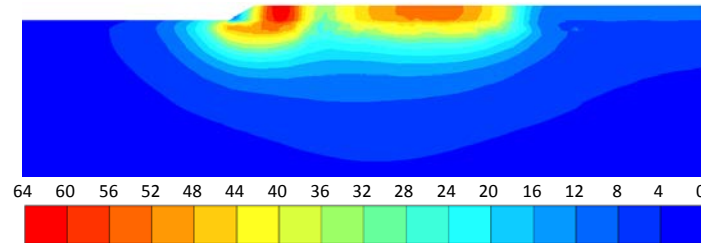
Figure 12 Deviatoric strain (10^{-3} m) due to Build-up of the 1st layer of the interim cover (vertical cross section at chain of Pistenbully)

CONCLUSION

The Dänkritz 2 uranium mill tailings pond is one of the abandoned uranium mining sites in Eastern Germany of high priority. The radiological, environmental and geotechnical site assessment proved the need for a site remediation. Wismut´s department for Project Management Saxonian Abandoned Wismut Mining Sites was given the responsibility to manage the project. It designed the tailings remediation project and applied for the respective planning approval in 2015. The remediation included water collection and treatment during remediation, interim covering of sandy and of soft fine grained tailings, dam reshaping, re-contouring of the pond area enhancing tailings consolidation by using vertical drains, placement of a multi-layer type final cover and vegetation. Geotechnical analyses were carried out by CDM Smith as part of the remediation design.

In order to analyse the stability of the ring dam under seismic loading CDM Smith performed a fully dynamic calculation using finite element models. Soils have been modelled using the hardening soil model with small strain stiffness, which implements a hysteretic damping. A Rayleigh damping was additionally defined. The calculations were made applying a horizontal and a vertical acceleration of the safe-shutdown earthquake. The factor of safety was determined by repeating the calculations while reducing the shear strength parameters and analysing the development of plastic failure in the system. The ring dam did not show sufficient stability under seismic loading due to an earthquake in its actual state. For this reason the slope of the dam was profiled. Additionally, a multi-layer type surface cover was provided to reduce the quantity of seepage water which had a positive impact on the stability of the slope. Sufficient stability of the ring dam under seismic loading could be proven for the final state with a global factor of safety of $\eta_{min} > 1.10$.

In order to place Wismut into a position to design the re-contouring work in the right sequence and time, such that the final cover lies in correct contour and the water drainage system is working after completion of consolidation of the tailings, CDM Smith carried out a three-dimensional finite element calculation using advanced constitutive soil models for the simulation of the non-linear and time-dependent behaviour of the tailings. The parameters used for the soil model have been calibrated in one-dimensional models based on results from laboratory and field investigations. The

modelled sequences followed the procedure of the construction with the aim to prove that the major portion of settlement due to consolidation will be completed at the end of the re-contouring works. This was essential for the design strategy of the construction sequences and construction time. For the final state a total settlement of 1.3 m was calculated at the time step approximately 21 months after completing the final surface cover. This settlement still ensured the functionality of the planned surface runoff diversion system and the drainage system within the surface cover.

It is necessary to prevent any ground failure when using construction equipment for successive placement of the interim cover on the tailings. This is accomplished by installing an underlying geogrid. The aim of the stability analysis carried out by CDM Smith was to prove sufficient load-bearing capacity of the geogrid. This was done pursuant to Eurocode 7 while applying the finite-element method with factored loads. Stability has been proven as the finite element model converged in the calculation. The geogrid was designed for the tensile stress resulting in the calculation, using factored loads.

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