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Research results on soil liquefaction of mine dumps in Eastern Germany

Résultats de la recherche sur la liquéfaction des sols dans les décharges minières en Allemagne de l'Est

Wolfram Kudla, Tino Rosenzweig, Sebastian Szczyrba & Joerg Weissbach
Technical University of Freiberg - Institute of Mining and Special Foundation Engineering, Germany, Wolfram.Kudla@mabb.tu-freiberg.de

ABSTRACT: After the German Reunification in 1990, more than 30 open-cast lignite mining operations of the former GDR have been shut down and recultivated in Eastern Germany. In the course of shutdown, the groundwater level has mostly risen by 30 to 50 m in loosely bedded sand (dumped overburden material). This rise of the groundwater level has led to more than 125 soil liquefactions in inner dumps since 2006. Soil liquefaction is divided in flow liquefaction and surface collapse. Surface collapse can also lead to a rise of the terrain surface. Hazard maps have been prepared indicating the areas in which flow liquefaction events and surface collapses occur in case of soil liquefaction. The state of stress in dumps before the rise of the groundwater level was thoroughly determined within a large test field in an active open-cast mine. In overburden dumps, the state of stress cannot be described by the at-rest earth pressure coefficient alone. In another test field in the former Sedlitz open-cast mine, earth pressure measurements were carried out to determine that the horizontal stress in the area of the capillary fringe is reduced if the groundwater level rises. A geotechnical stability proof was developed for proof against balance deformations in case of surface collapse.

RÉSUMÉ: En Allemagne de l'Est, après la réunification allemande en 1990, plus de 30 mines de lignite à ciel ouvert de l'ancienne RDA ont été fermées et remises en culture. Au cours du démantèlement, la nappe phréatique a augmenté de 30 à 50 m dans le sable peu compact (matériau d'excavation). Lors de la recharge des eaux souterraines, plus de 125 liquéfactions du sol se sont produites depuis 2006. La liquéfaction du sol est classée en flux d'affaissement et en effondrement de terrain. Dans le cas d'effondrements de terrain, un soulèvement de la surface du sol peut également se produire. Des cartes de danger ont été élaborées, montrant les zones où se produisent les événements de flux d'affaissement et les zones où l'effondrement de terrain se produit en cas de liquéfaction du sol. Le niveau de tension sur les terrils a été déterminé dans un grand champ d'essai dans la mine à ciel ouvert de Reichwalde avant la montée des eaux souterraines. Dans les terrils, l'état de contrainte ne peut être décrit par le seul coefficient de pression terrestre. Dans un autre champ d'essai situé dans l'ancienne mine à ciel ouvert de Sedlitz, les mesures de la pression terrestre ont montré que la contrainte horizontale dans la zone de la frange capillaire diminue lorsque la nappe phréatique monte. Un contrôle de stabilité géotechnique a été développé pour la vérification contre les déformations compensatoires en cas d'effondrement du terrain.

KEYWORDS: Soil liquefaction, flow liquefaction, surface collapse, capillary fringe, geotechnical stability proof.

1 MINING-RELATED AND GEOTECHNICAL STARTING CONDITIONS

Until 1990, around 40 open-cast lignite mining operations were active in the former GDR (former Eastern Germany). The open-cast mining operations covered a surface between 2 km² and approximately 17 km². After the German Reunification in 1990, most open-cast mines have been successively shut down and recultivated in Eastern Germany. Today, four open-cast mines are still active in Lusatia (in the east of Germany) as well as three each in the Middle German and the Rhenish mining areas.

The open-cast mining operations closed in Eastern Germany after 1990 are being recultivated by the Lausitzer und Mitteldeutsche Bergbau-Verwaltungsgesellschaft mbH (LMBV) (100% subsidiary of the federal government). The four active open-cast mines in Lusatia are operated by the Lausitz Energie AG (LEAG). As lignite mining will no longer be subsidized in Germany due to climate policies, the open-cast mines currently still active will also be closed by 2038.

The open-cast mining operations already shut down after 1990 in Lusatia are concerned in this paper, as partly significant soil liquefaction events in the course of recultivation could only be observed there.

In Lusatia, the overburden material over the lignite is conveyed by means of overburden conveyor bridges over the lignite mining operation (see figure 1). The lignite is usually mined under the conveyor bridge using bucket-chain and bucket wheel excavators. The overburden material is dumped from the conveyor bridge and bedded without compression in bulk discs (cross-bedding l).

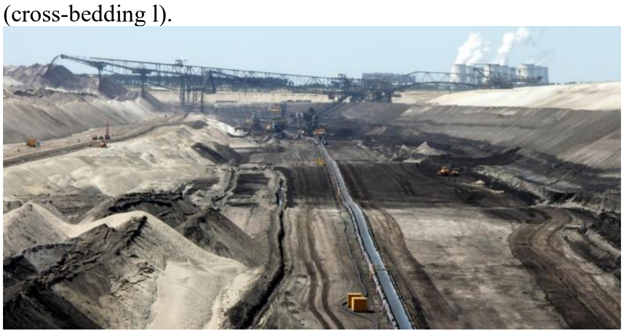

Figure 1. Conveyor bridge in an active open-cast mine in Lusatia.

In Lusatia, the overburden material usually consists of fine and medium sand, often with very low proportions of fine grain. The proportion is often between 2 % and 8 %, with a maximum of approx. 20 %. The fine sands are very narrowly graded (coefficient of uniformity C_u = 1.5 to 3). After dumping, the overburden material has a thickness of approx. 20 to 50 m (in active mines till 90 m) and is bedded loosely to very loosely. Due to the grain distribution and the very loose bedding, the sands are highly susceptible to liquefaction. Approx. 90 % of soil liquefactions occurred in the Northern region of Lusatia. Seese sands, marine deposits mostly found in the North of Lusatia, are particularly susceptible to liquefaction. The coefficient of permeability in north Lusatia is measured between 5.5×10^{-5} - 6.2×10^{-6} m/s. The rounding coefficient is between 0.83 and 0.86.

After the open-cast mine is exhausted, it is recultivated and in the course of this recultivation, the groundwater level rises (see figure 2).

Due to the mined lignite and the related deficit in mass, large cavities are formed and flooded when the groundwater level rises (post mining lakes). To avoid earth slides at the relic lakes, the shore around the relic lakes is compacted on a width of approx. 50 to 150 m by deep vibration or blast compression before the rise of the groundwater level (see figure 2). After rise of the groundwater level, the depth of the ground-water table in the area of the inside dumps is usually between 1 and 15 m, often between 2 and 4 m.

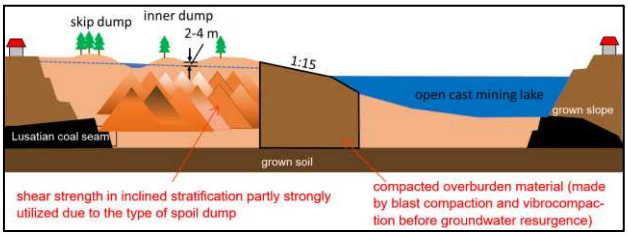

Figure 2. Schematic cross-section through a lignite open-cast mine after recultivation.

2 CLASSIFICATION OF LIQUEFACTION EVENTS

In the course of rise of the groundwater level, approx. 125 soil liquefaction events have occurred between 2006 and 2018 at the flat inner dumps in Lusatia with a total surface of approx. 0.01 ha to 1.79 km^2 (Weissbach 2020). In Eastern Germany, access to approx. 20,000 ha of dumps is fully restricted or may only be possible under strict conditions.

Almost all soil liquefaction events were initiated naturally. This means, they were not caused by vibrations (e.g. by compression equipment or blasting). The actual initiating origin of the liquefaction is unknown and therefore referred to as "natural trigger". This is static soil liquefaction. Liquefaction occurs spontaneous without any warning. Additionally, liquefaction has occurred in mostly horizontal areas of dumps with low inclination (slope angle mostly 0° to 8°; slope height of 0 m up to 16 m, mostly 0 to 6 m) (Kudla et al. 2017).

The liquefaction events with visible changes at the terrain surface are classified as follows:

1) **Type 1: Surface collapse** due to liquefaction (= terrain collapse - see figures 3)
   In case of surface collapse after resulting liquefaction, mainly vertical shifts occur at the terrain surface. In final condition, the horizontal shifts of 0 to 1m can be identified at the terrain surface (determined with orthophotos). Further distinction:
   Type 1a: Surface collapse due to liquefaction without rising of the dump surface (see figure 3)
   Type 1b: Surface collapse due to liquefaction with rising of the dump surface (sum from rising of the terrain surface and sedimentation build-up) (see figure 14)
   In case of a surface collapse of type 1b, considerable shifting occurs in the water-saturated dump. The terrain surface is lowered in the higher section and lifted in the lower section without horizontal shift of the terrain surface. The originally inclined terrain is levelled.
2) **Type 2: Flow liquefaction** at inner dumps (= flow slide - see figures 4 and 5)

The term flow liquefaction refers to soil liquefaction resulting in considerable horizontal and vertical shifts due to the morphology of the terrain. The soil breaks into clods of earth which can be transported over distances of more than 100 m. The distinction between flow liquefaction and terrain collapses was made because the damage from a flow liquefaction is much greater than from a surface collapse. With the help of hazard maps, it is possible to distinguish in which areas flow liquefactions and surface collapses may occur (see Kudla et al. (2017) and Weissbach (2020)).


Figure 3. Surface collapse (path originally running horizontally, red line) at the former Schlabendorf-South open-cast mine.


Figure 4. Sand craters after flow liquefaction at former Schlabendorf-South open-cast mine.

Figure 5. Flow liquefaction with breakages at former Schlabendorf-South open-cast mine in 2008.

3 RESULTS OF VARIOUS RESEARCH PROJECTS

Since 1990 and considerably since 2011, the LMBV and LEAG have assigned various research projects to the Technical University of Freiberg and various engineering offices with the goal to identify the causes and framework conditions for soil liquefaction. In particular, the goal of LEAG was to adjust their planning of open-cast mining operations in such a way that flow liquefaction events in approx. 20 years after shutdown and recultivation of still active open-cast mines are prevented.

As the shearing behaviour of loose sands under undrained conditions is generally known (i.e. Ishihara (1996), Jefferies & Been (2006) and Foerster (1998)) and bedded sands in all open-cast mines in Lusatia are generally susceptible to liquefaction, execution of soil-mechanical laboratory tests had a lower priority in research.

3.1 *Stress condition in dumps*

The overburden dumps in Lusatia, where soil liquefaction occurred, were filled using a conveyor bridge. This way, the sand was deposited in cross-bedding (see figures 1 and 2). Figures 6 and 7 indicate the ribs and valleys that are caused by a conveyor bridge.

Figure 6. Installation of stress measuring stations and cross-bedding of dump.

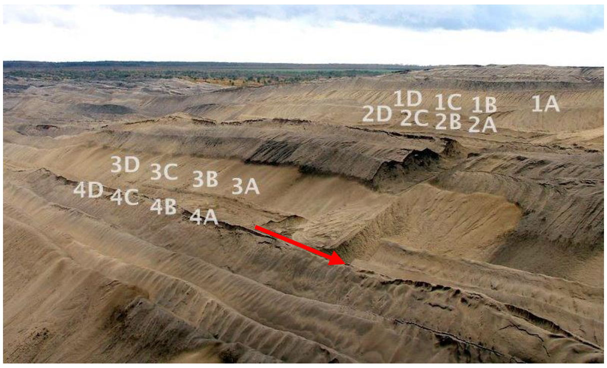

Figure 7. Cross-bedding of the dump with measuring cross-sections A, B, C and D (arrow in stope direction).

In the end, the top ribs of the bulk filling were covered with a spreader dump. The spreader dump is levelled afterwards. Due to cross-bedding, the shear strength of the soil is already almost fully utilized as the sand is bedded according to its natural bulk angle, which corresponds to the friction angle at a vertical stress of zero. Additionally, some segregation occurs as more course material deposits collect at the lower end of the slope.

As the shear strength of the soil is almost fully utilized due to the type of bulk filling and therefore the stress condition is highly anisotropic, only low deviator stresses under undrained conditions can be absorbed after rise of the groundwater level until the breaking strength is reached. Additionally, the deformation (compression) ε_f is very low before the breakage condition is reached. From laboratory tests, it is known that this value is often only approximately less than 0.1 % of the sample height at loosely bedded sands in Lusatia. Under undrained conditions, the sand shows brittle behaviour. Geotechnical calculations (for example, slope stability calculations after Krey-Bishop or Morgenstern-Price), however, require material with ductile behaviour. In brittle material, load redistribution is not possible or can lead to immediate breakage.

For experimental recording of the stress condition in dumps, a total of 16 stress measuring stations in 4 measuring cross-sections (A, B, C, D in figure 7) were installed in a large test field in an active open cast mine in Lusatia. Each stress measuring station consists of 3 earth pressure cells and one inclination sensor (see figures 6 to 9). Stresses have been measured since 2011. Inclination was measured at the beginning during dumping.

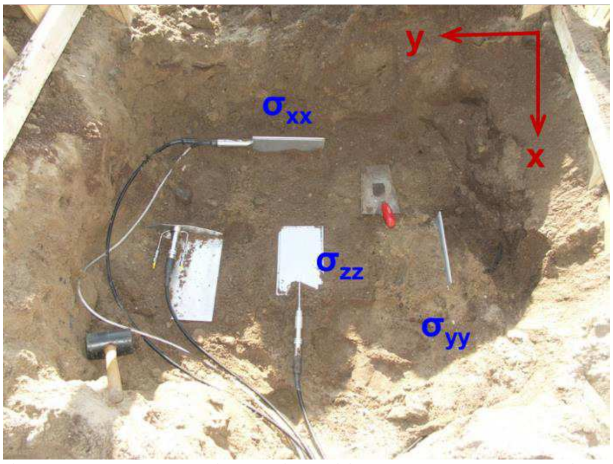

Figure 8. Stress measuring station with 3 pressure sensors and one inclination sensor.

Afterwards, the stress measuring stations were covered with overburden sand by multiple passes with the conveyor bridge. Figure 9 illustrates the measured vertical stress curve at 4 measuring stations in the measuring cross-section D over one year. From the measured stresses, the horizontal stress coefficients $K_{xx}(t)$ in stope direction and the horizontal stress coefficients $K_{yy}(t)$ perpendicularly to the stope direction were calculated. In some years, the dump will be covered by a spreader dump. Afterwards, the groundwater level should rise. The events described in the following apply for a dump which is not yet water-saturated. The following events illustrate the stress measurements (Kudla et al. 2017):

1) Up to now, horizontal stress coefficients K between 0.28 and 0.71 were measured. The mean value is approx. 0.5. However, as the horizontal stress coefficients are subject to high fluctuations, a mean value cannot be assumed for geotechnical calculations as this would not be reliable. Until now, horizontal stress coefficients between 0.39 and 0.62 were determined at overburden heights of more than 15 m. This way, the horizontal stress coefficient can already be near the critical main stress conditions before the groundwater level rises.
2) The horizontal stress coefficients K_{xx} in stope direction showed a lower spread than the horizontal stress coefficients K_{yy} perpendicularly to the stope direction. In stope direction, more homogeneous conditions apply for material distribution as well as the stress.
3) Homogeneous stress conditions, which could be described by means of an at-rest earth pressure coefficient alone, do not apply in inner dumps.

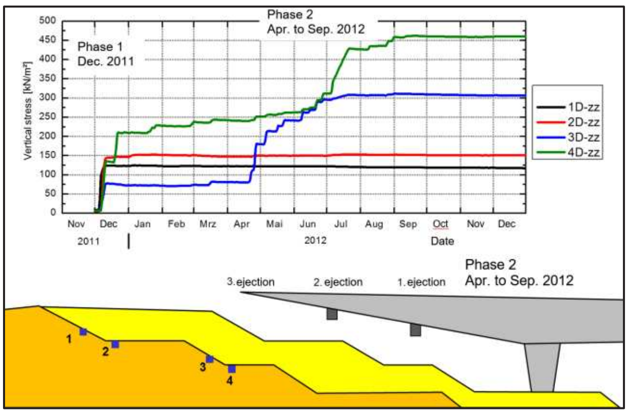

Figure 9. Measured vertical stress in cross-section D.

Afterwards, a dump made up of bulk discs was modelled by means of finite element calculations, which was calibrated by measurement data. Figure 10 illustrates the calculated utilization factor of shear strength if the bulk discs are activated successively one after the other according to production.

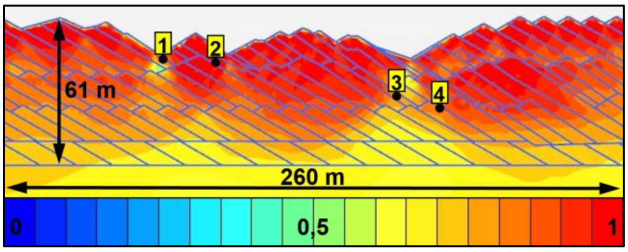

Figure 10. Utilization factor of shear strength from FE calculation with hardening soil model.

The calculated utilization factor of shear strength highly depends on the applied material model. In the calculations, the Mohr-Coulomb model and the Hardening Soil model were used. Figure 10 shows that the degree of utilization of the shear strength is for the most part between 90 and almost 100%, even with a nearly level dump down to 30 m depth. This is a big difference from geologically sedimented material.

3.2 *Sedlitz measuring field*

It could be determined that soil liquefaction occurs in inner dumps when the groundwater level sinks as well as when it rises (Rosenzweig 2022). From 2006 to approx. 2014, the groundwater level in numerous former open-cast mines in Lusatia has mostly risen due to targeted flooding of the post mining lakes. Measurements carried out by CPTU and dynamic probing also showed that the peak resistance is partly very low (0.5 MPa to approx. 1.5 MPa) at the capillary fringe in depths of approx. 2 to 8 m. It was assumed that the shear strength is partly exceeded due to redistributions of stress in the area of the capillary fringe, leading to the initiation of liquefaction under undrained conditions.

For further examination of this, a comprehensive test field was installed in 2016 in the former Sedlitz open-cast mine. In the next years, the groundwater level is going to increase by another 7 m. Figure 11 shows a photo of the test field. In the test field, a total of 10 pore water pressure sensors (PWD), 6 tensiometers (Ten), 2 groundwater levels (GwM), 7 rod extensometers (StE) and 2 settlement levels (SeP). Additionally, the test field includes a weather station, GNSS sensor and corner reflector.

Also 8 earth pressure cells (EDG) were installed at the end of 2016 in depths of approx. 2, 4, 7 and 10 m (slightly over the groundwater level) for measurement of the horizontal earth pressure stresses, the results of which are presented below.

The test field contains loosely bedded sands with a fine grain proportion of approx. 2 to 8 %. At the start of the measurement (02/2017), the groundwater table distance was approx. 10 m. Currently (05/2021), the groundwater level has risen by approx. 2.4 m.

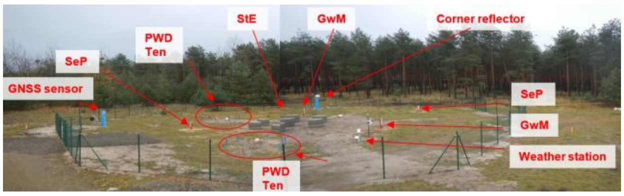

Figure 11. Sedlitz measuring field with measuring equipment.

Figure 12 shows the changes in horizontal stresses at both earth pressure sensors in a depth of 10 m since start of the measurement. Additionally, the groundwater level rise (GWL) measured by means of the water pressure sensors (air pressure-compensated) is illustrated. Multiple abrupt increases in total horizontal stress levels can be clearly identified in figure 12 (red arrows) with the horizontal stress slightly decreasing afterwards. This is caused by grain redistribution when the capillary fringe is passing through the sensor level from bottom to top. Additionally, the total horizontal earth pressure stress σ_h is slightly dropping at EPC-07 as of approx. 11/2018 and at EPC-08 as of approx. 03/2019 although the groundwater level is continuously rising. As of approx. 09/2019, the horizontal stress levels are rising again. While the horizontal stress was dropping, the groundwater level has risen by approx. 0.5 m. This drop in horizontal stress levels is caused by the redistribution of grain at decrease of the capillary cohesion by saturation of the sand in the capillary fringe as the sand is bedded loosely to very loosely. At sands with medium-dense or dense bedding, the horizontal stress level in the capillary fringe does not drop if the groundwater level rises.

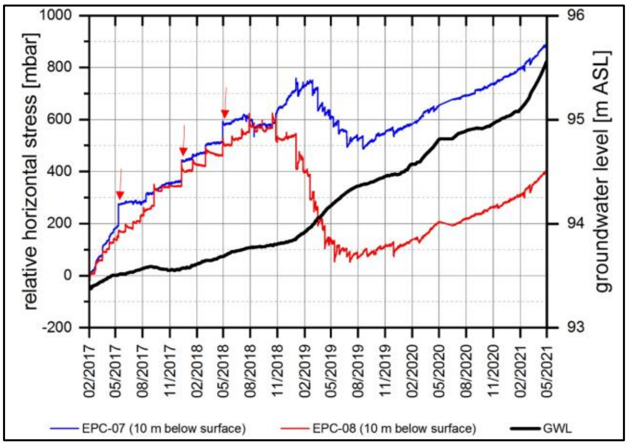

Figure 12. Increase/decrease of the horizontal total stress in a depth of 10 m at the Sedlitz measuring field at passing of the capillary fringe through the sensor level.

In technical trials as well, in a pit with the dimensions 2 x 2 x 1 m³ at the Technical University of Freiberg, a decrease in horizontal stress was also observed at groundwater level rise in very loosely bedded sands (Kudla et al. 2017 and Kudla et al. 2015b).

As the horizontal stress σ_x drops, the horizontal stress coefficient $K = \sigma_x / \sigma_z$ is reduced. The precise mechanism, however, could still not be determined and is further under research. As the horizontal stress coefficient drops, the critical main stress in the dump may be exceeded and liquefaction be triggered under undrained conditions. Against this assumption speaks, however, that an overpressure in the pore water at the capillary fringe could be reduced very quickly. However, as the dumps are not structured homogeneously and the bulk discs have partly different fine grain proportions, it cannot be excluded that the drainage possibilities on site are limited. Following this, it cannot be excluded that liquefaction events are locally initiated at the capillary fringe under undrained conditions due to local changes in stress during saturation. This would be referred to as "natural trigger".

3.3 *Model for proof against balance deformations*

The research project (Kudla et al. 2017) also included a discussion about potential models for proof of the stability against flow liquefaction or surface collapse.

Many geotechnical calculation models require sufficient ductility of the soil or the system of soil/construction. In compliance with DIN EN 1997 (Eurocode 7) Geotechnical design – Part 1: General rules, which is applicable in many European countries, as well as DIN 1054 Subsoil – Verification of the safety of earthworks and foundations – Supplementary rules to DIN EN 1997-1, it must be checked whether the soil subject to shearing stress shows sufficiently ductile behaviour. If the material shows brittle behaviour, the breakage occurs without warning, i.e. without previous deformation indicating a breakage. In the construction industry, loosely bedded sand is compacted for this reason, to ensure that the sand has a ductile behaviour. This can often be safely ensured if the critical pore number e_{krit} is fallen below.

However, if the strength of very loosely bedded, water-saturated sands under undrained conditions is exceeded, the sand shows brittle breakage behaviour and liquefaction occurs. In case of a brittle behaviour of the material, breaking strengths (friction angle φ_f and cohesion c_f) must not be applied for geotechnical stability calculation as the load cannot be redistributed from areas where shear strength is utilized to areas where the shear strength is not yet utilized (no redistribution of loads). The condition for geotechnical stability calculation is that the materials, through which the breakage is running at geotechnical calculation, have approximately similar stress-strain-curves and show ductile behaviour.

If the breakage is running through materials with brittle behaviour (e.g. in case of liquefaction), only the residual friction angle $\varphi_{u,R}$ or the undrained cohesion $c_{u,R}$ may be applied as shear parameters for geotechnical calculation.

The inner dumps of the former open-cast mines in Lusatia are often slightly undulated. The slope angles are approximately 4° up to 20° (often only up to 8°). For stability calculations, classical slip circles and the approach according to Krey-Bishop were applied, whereas the undrained residual friction angle $\varphi_{u,R}$ (or the undrained cohesion $c_{u,R}$) were applied for areas under the groundwater level, where liquefaction may potentially occur. Based on the slip circle calculations, the safety factor FS_{FL} (Factor of Safety in case of Flow Liquefaction) is calculated. As the undrained residual friction angle $\varphi_{u,R}$ is mostly between 0° and 5°, slopes must be realized with a very low angle.

For recultivation of open-cast mining operations, the LMBV is required to compact all dump materials and surfaces in such a way that flow liquefaction is prevented. Respectively, almost all slopes with slope angles of more than 4° must be compacted.

Surface collapses are generally distinguished by type 1a and 1b (see section 2). Although horizontal shifts at the terrain surface do not occur at type 1b, the terrain may drop (e.g. 5 m) or rise (e.g. 2 m) by several meters. The terrain drop and terrain rise are caused by horizontal shifting of large volumes of liquefied sand under the groundwater level during liquefaction. At the same time, the earth-moist covering is thick enough and at a very low surface angle, which is why it does not break into clods.

Figure 13 shows a top view on a height model from a surface collapse caused by soil liquefaction at the former Seese open cast mine along the stope direction with a length of 600 m (Kudla et al. 2017). Terrain drops of up to 3 m and terrain rises of up to 1.7 m have occurred there.

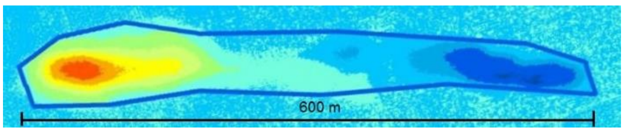

Figure 13. Surface collapse with a length of 600 m (measured vertical shift: max. 3.0 m (red) and min. -1.7 m (blue)).

Figure 14 illustrates a longitudinal section through the surface collapse and the terrain surface in 2005 before the surface collapse with an inclination of 1:40 on the left (south) and 1:90 in the right (north) as well as the terrain surface after the surface collapse in 2011. The groundwater table distance before the surface collapse was 4 to 10 m.

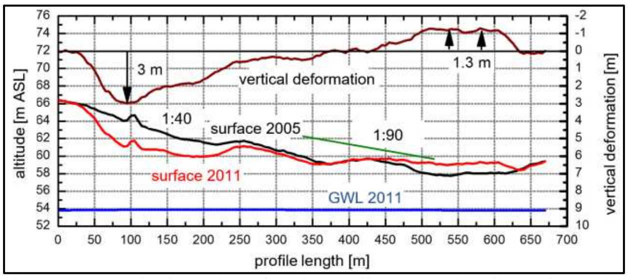

Figure 14. Longitud. section of surface collapse E23 (Kudla et al. 2017).

At this surface collapse, large volumes of liquefied sand have shifted along the stope direction under the groundwater level without any horizontal shift at the terrain surface. For the proof of potential occurrence of large horizontal shifts of the liquefied sand in case of surface collapse (without horizontal shift of the terrain surface), a model was developed (figure 15 and 16). In this model, it is assumed that a rectangular area with a height of h_{tfl} under the groundwater level is subject to homogeneous liquefaction. The sand in this area has an undrained residual shear strength of $\tau_{u,R}$ and a saturated specific gravity of γ_r. For this area, proof of the sliding resistance is obtained. For this, the total horizontal stress levels are applied on both sides.

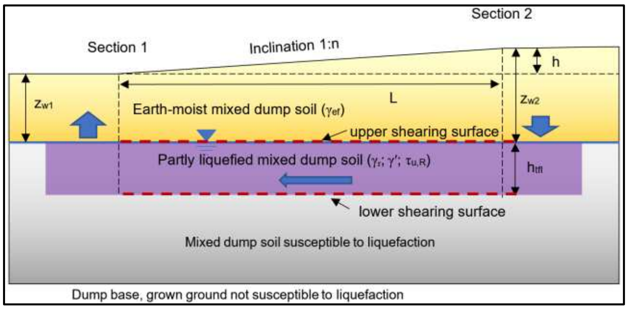

Figure 15. Initial situation at proof against balance deformation (Kudla et al. 2017).

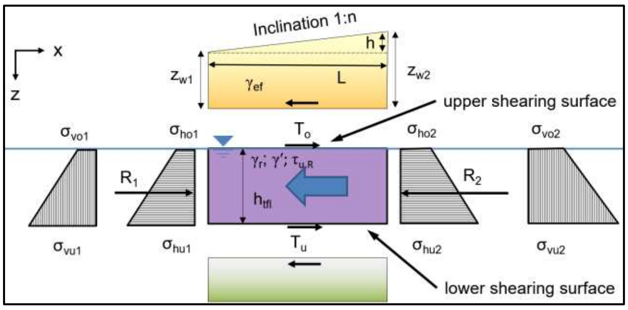

Figure 16. Model for proof against balance deformation (Kudla et al. 2017).

The level case is observed, in which the liquefied area, with a residual shear strength $\tau_{u,R}$, may be theoretically continued indefinitely in perpendicular direction to the image plane. For the proof, the slope is limited by the sections 1 and 2. Volume and shearing deformations in the liquefied area with a residual shear strength $\tau_{u,R}$ are ignored. Eventually, the liquefied area with $\tau_{u,R}$ is assumed as rigid body subject to holding and driving forces to establish a balance of forces (figure 16). The residual shear strength $\tau_{u,R}$ necessary to prevent horizontal shift of the rigid body is determined.

The driving forces are the horizontal total stresses in section 2, which are summarized into the resulting force R_2. This is countered (counter-held) by the horizontal total stresses in section 1, which are applied to form the resulting force R_1. The residual strength in the top and bottom sliding surface act as additional holding components and are combined over the length of the slope L in the resulting holding forces T_o and T_u.

This way, the approach corresponds to a proof of sliding resistance which is common in the field of geotechnics and complemented by the top holding force T_o.

The stability coefficient FS against balance deformations (BD) is determined according to equation 1.

$$FS_{BD} = \frac{existing\ residual\ shear\ strength}{required\ residual\ shear\ strength} = \frac{\tau_{u,R,exist}}{\tau_{u,R,req}} \quad (1)$$

For the dumps in Lusatia, the height h_{tfl} of the liquefied area is applied with approx. 15 m. This height corresponds to the liquefication depth of the surface collapses investigated in Schlabendorf-South.

An increase in the residual strength over the depth can also be considered. For variations of h_{tfl}, a minimum for FS_{BD} applies and an assumption of h_{tfl} is not required.

In the report (Kudla et al. 2017), it was proven that the force T_o can be derived by calculation from the ground resistance in the earth-moist covering, if the following applies: $FS_{BD} \leq FS_{FL}$. In case that $FS_{BD} > FS_{FL}$ is calculated, the safety against flow liquefaction is decisive for assessment of the stability. The stability against flow liquefaction is determined by means of slip circle calculations with application of the undrained residual friction angle $\varphi_{u,R}$ (or the undrained cohesion $c_{u,R}$) for the soil under the groundwater level.

For a (level) slope, the proof of the stability is generally established first by means of a slip circle (FS_{FL}) with application of the undrained residual friction angle $\varphi_{u,R}$ (or the undrained cohesion $c_{u,R}$) for the sand under the groundwater level. If this proof is established, the model illustrated in figure 16 is applied to check whether the proof can also be established against balance deformations.

One decisive condition for proof of stability with slip circles and proof against balance deformations using the undrained residual shear resistance is that the undrained residual friction angle $\varphi_{u,R}$ (or the undrained cohesion $c_{u,R}$) determined in anisotropic consolidated, undrained triaxial tests (CAU-pS) can be correctly estimated. The undrained residual friction angle $\varphi_{u,R}$ is determined in a triaxial test for a specific stress condition. Only if this stress condition is approximately complied with in the field, both proves may be applied. If it is to be expected that the motion during liquefaction eventually leads to widening (even only local) and the slightest increase in pore proportion, the undrained residual friction angle $\varphi_{u,R}$ drops (eventually to zero). This restriction must be taken into consideration in definition of shear parameters for stability calculations.

4 CONCLUDING REMARK

This article describes some of the research results reached over the previous years for LMBV and LEAG. The research work is continued on various topics in the field of soil liquefaction. The principle behind the soil-mechanical behaviour of loosely bedded sand under undrained conditions has been largely clarified in laboratory tests. Additional information can be obtained by field testing. However, considerable difficulties arise in field testing due to a lack of surveying possibilities as a large proportion of the dumps are restricted for safety reasons and cannot be accessed even for surveying with drilling equipment or cone penetration testing vehicles. As taking of undisturbed samples for testing from water-saturated loosely bedded sands is highly difficult and largely undisturbed sampling is actually only possible if (expensive) frozen samples are used, field testing is still recommended despite all above mentioned obstacles regarding access to the test sites.

5 ACKNOWLEDGEMENTS

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