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Deep soil compaction as a method of ground improvement and to stabilization of wastes and slopes with danger of liquefaction, determining the modulus of deformation and shear strength parameters of loose rock

Michael Lersow *

BUL Sachsen GmbH, Germany, Spreetaler Straße 4, D-02979 Elsterheide, OT Sabrodt, Germany

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Abstract

For the stabilization of dumps with the construction of hidden dams and for building ground improvement, for instance for traffic lines over dumps, nearly all applied compaction methods have the aim to reduce the pore volume in the loose rock. With these methods, a homogenization of the compacted loose rock will be obtained too. The compaction methods of weight compaction by falling weight, compaction by vibration and compaction by blasting have been introduced, and their applications and efficiencies have been shown. For the estimation of the effective depth of the compaction and for a safe planning of the bearing layer, respectively, the necessary material parameters have to be determined for each deep compaction method. Proposals for the determination of these parameters have been made within this paper. In connection with the stabilization of flow-slide-prone dump slopes, as well as for the improvement of dump areas for the use as building ground, it is necessary to assess the deformation behavior and the bearing capacity. To assess the resulting building ground improvement, deformation indexes (assessment of the flow-prone layer) and strength indexes (assessment of the bearing capacity) have to be determined with soil mechanical tests. Förster and Lersow, [Patentschrift DE 197 17 988, Verfahren, auf der Grundlage last- und/oder weggesteuerter Plattendruckversuche auf der Bohrlochsohle, zur Ermittlung des Spannungs-Verformungs-Verhaltens und/oder von Deformationsmoduln und/oder von Festigkeitseigenschaften in verschiedenen Tiefen insbesondere von Lockergesteinen und von Deponiekörpern in situ; Förster W, Lersow M. Plattendruckversuch auf der Bohrlochsohle, Ermittlung des Spannungs-Verformungs-Verhaltens von Lockergestein und Deponiematerial Braunkohle — Surface Mining, 1998;50(4): 369–77; Lersow M. Verfahren zur Ermittlung von Scherfestigkeitsparametern von Lockergestein und Deponiematerial aus Plattendruckversuchen auf der Bohrlochsohle. Braunkohle — Surface Mining, 1999;51(1):39–47] improved a direct procedure, the so-called plate-loading test. With this improved procedure, it is possible to produce profiles of deformation parameters and shear-strength parameters of the loose rock. On this basis the settlement behavior and the bearing behavior of the ground can be described. The PDV-BS cone-penetration test and the pressiometer test are compared and the reliability of the soil mechanical indexes are assessed critically. The PDV-BS can be used as a calibration test for cone penetration tests as well as for the calibration of pressiometer tests. With the application of a PDV-BS and a pressiometer test in combination in a testing field, the anisotropy properties of the loose rock can be proved. © 2001 Elsevier Science Ltd. All rights reserved.

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1. Introduction

With the re-unification of both German states, the majority of brown-coal open pits in the regions "Mitteldeutschland" and "Lausitz" had to be closed. The amount of energy produced from brown coal was restricted for some reasons. The legal owner of abandoned, mining influenced areas became the Federal Republic of Germany.

Due to the "Bundesberggesetz" (mining Act), the owner of mining sites has to rehabilitate these abandoned areas. The areas have to be given back to public

* Tel.: +49-3564-389307; fax: +49-3564-389347.
E-mail address: lersowm@bul.de .

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use. This means that all connected dangers have to be removed. First then these areas are released from the supervision of the mining authority.

Dangers for the public arise from:

- abandoned open pits, dumps and slopes itself,
- the inclination of uncompacted dump-slopes (flow-slide-prone areas), especially in the Lausitz area,
- shock-waves into water-filled open pits, formed from great-volume slidings of dump slopes, induced by flow-sliding,
- the possibility of the failure of dams, for instance between two open pits,
- a disturbed groundwater balance in the brown-coal area and neighboring regions,
- the production of acid water after a re-rising of ground water.

Therefore, the following main tasks for the rehabilitation of abandoned mining sites arise:

- removal of the danger of landslides, to secure the sliding-prone areas, mainly by deep-soil compaction methods,
- recultivation and landscaping of former mining sites,
- establishment of a self-regulating groundwater balance with a suitable water quality, etc.

Therefore, the requirements for the deep compaction methods, which are used in these special case, are defined.

If former mining areas should be released from the supervision of the mining authority, and former dump areas from brown coal mining operations should be used for the construction of traffic lines, special requirements are established on the settlement behavior and/or strength properties of the building ground. The deep-soil compaction methods and geophysical testing methods have to be able to establish and to prove the required deformation and/or strength parameters. Differences in the general procedures do not arise, but the danger for men and equipment is often greater if dump slopes have to be secured, whereby more strict requirements on safety measures arise. However, it is advantageous to combine the process of rehabilitation and the establishment of a settlement-stable building ground, because the Federal Republic of Germany is the owner of these sites, released from the supervision of the mining authority and subsequent constructions of traffic lines will be paid by the state with public money.

As already mentioned, lignite open-pit mining activities in the Lusatian area as well as processing of lignite and electricity production have drastically changed the natural grown landscape. In many areas, a public use is not possible. For this reason, the slopes have to be stabilized. This is especially necessary on flow slide-prone dumps, summing up to a total length of 100 km in this region (Fig. 1). One stabilization method is the creation of so-called hidden dams. Furthermore, the construction of traffic lines (like highways and railway lines) is planned (Figs. 2 and 3).

A number of soil-improvement methods are used to guarantee the function and the proposed life time of the constructions and to reduce secondary costs.

2. Building ground improvement, stabilization of dumps

2.1. Objectives

Building ground improvement and the stabilization of slopes with hidden dams are necessary if the expected settlement is too high or if the load on the ground exceeds its load capacity. The load capacity of the ground can be improved to that amount that a shear failure of the ground can be prevented, but settlements can only be prevented gradually. The expense increases with the thickness of the sensible layer and it increases with the restriction of the allowed settlement or settlement velocity of buildings [1].

To stabilize dump slopes usually a four-step procedure is applied:

1. establishment of a supporting body by compaction by blasting,
2. stabilization of the zone in front with compaction by blasting,
3. decreasing of the slope angle for the construction of a plane for the compaction with vibration (RDV),
4. final slope shaping.

Fig. 1. Sliding, in 1998 (about 4.8 million m^3) on Koschen-dam near Senftenberg.

Fig. 2. Location of the "Lusatian-Ring" in the Lusatian lignite mining district.

Fig. 4 shows possible steps for the rehabilitation of a dump [2].

2.2. Stability analysis

Stability tests are necessary for the flow-slide-prone dumps. These tests have to be orientated to the necessary piece of work and to the technological order or the technological order is the result of the tests.

The safety of the workers and of the machines has to be guaranteed in all steps of the working process.

Therefore stability assessments will be carried out for the slope:

- before blasting
- during blasting
- after blasting
- and during subsequent works.

Calculation models for flow-slide-prone dump slopes will be established and different alternatives will be compared.

Fig. 5 [2] shows an example of a calculation model of a dump slope. A frequently used procedure is that established from JANBU. This procedures assume a balance of the forces and forced circular cylindrical failure planes. The safety is defined according to Eq. (1):

$$F_m = \frac{\sum[c'\cdot b\cdot R + (N - u\cdot b)\cdot R\cdot\tan\phi']}{\sum W\cdot x - \sum kW\cdot e \pm [D\cdot d] \pm A\cdot a} \quad (1)$$

where:

- R radius of the sliding plane
- N normal force
- u pore water pressure
- W weight of the segment
- x horizontal distance of the middle line of the segment to the pivot
- f rectangular distance of the normal force to the pivot
- kW horizontal dynamic force
- e vertical distance of the middle-line of the segment to the pivot
- D outer line-load
- d rectangular distance of the line load to the pivot
- A outer water-force
- a rectangular distance to the pivot
- b width of the segment.

The geometrical and mechanical relationships according to the introduced safety-definition [Eq. (1)] are shown in Fig. 6. The influence of the cohesion and the friction angle of the loose rocks on the safety definition of the dump slope, according to Eq. (1), can be seen clearly.

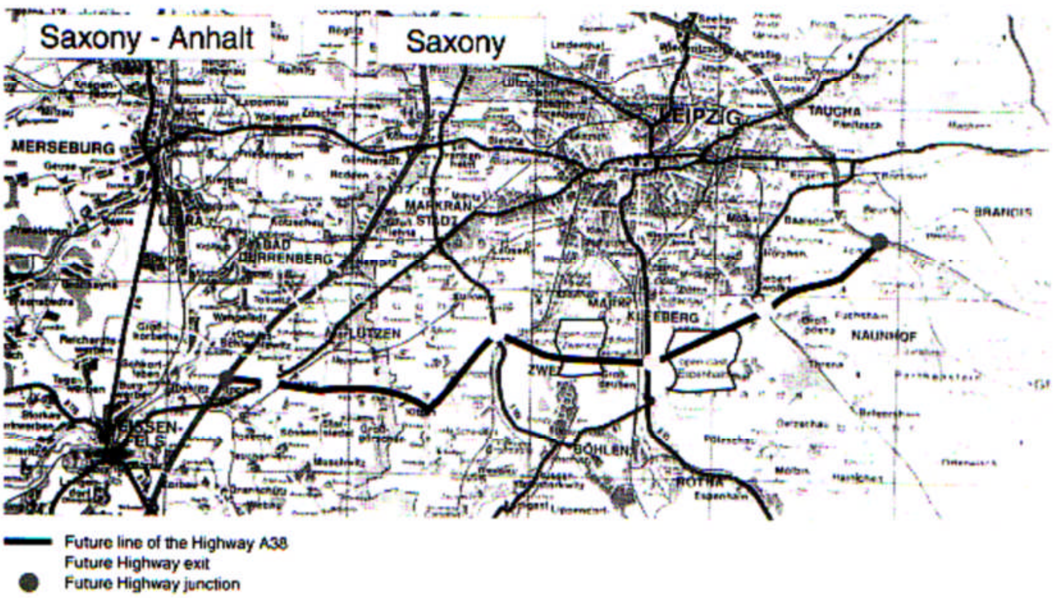

Fig. 3. Future line of the Highway A38, south of Leipzig.

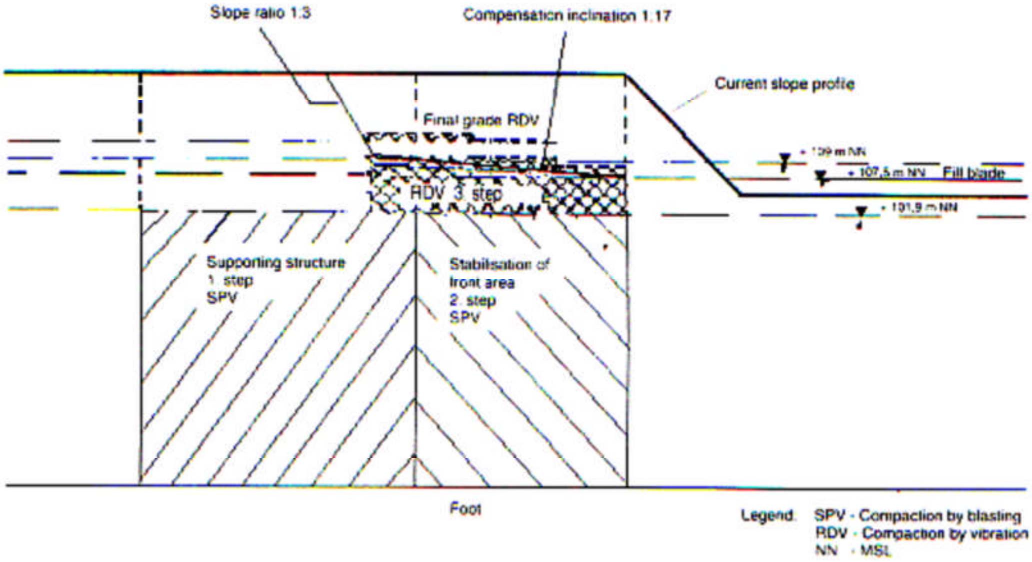

Fig. 4. Possible restoration procedure for the residual pit Burghammer [2].

Table 1
Strength parameters of different loose rocks from different dump slopes [3]

Number	Parameters of soil mechanics Loose rock	
1	Wet $\phi' = 37°$ $c' = 5$ kN/m^2 $\gamma = 18$ kN/m^3	Water saturated $\phi' = 37°$ $c' = 0$ kN/m^2 $\gamma_r = 18$ kN/m^3
2	Wet $\phi' = 30°$ $c' = 2$ kN/m^2 $\gamma = 18$ kN/m^3	Water saturated $\phi' = 30°$ $c' = 0$ kN/m^2 $\gamma_r = 18$ kN/m^3

Tables 1 and 2 summarize some soil-physical parameters of loose rocks.

2.3. Deep soil compaction methods

For the stabilization of dumps and for building ground improvement (for instance for traffic lines over dumps), nearly all applied compaction methods have the aim to reduce the pore volume in the loose rock. With the input of a load or an energy, the soil can be compacted this way so that the grains will be forced to seek a more dense arrangement and thereby its deformation tendency will be reduced.

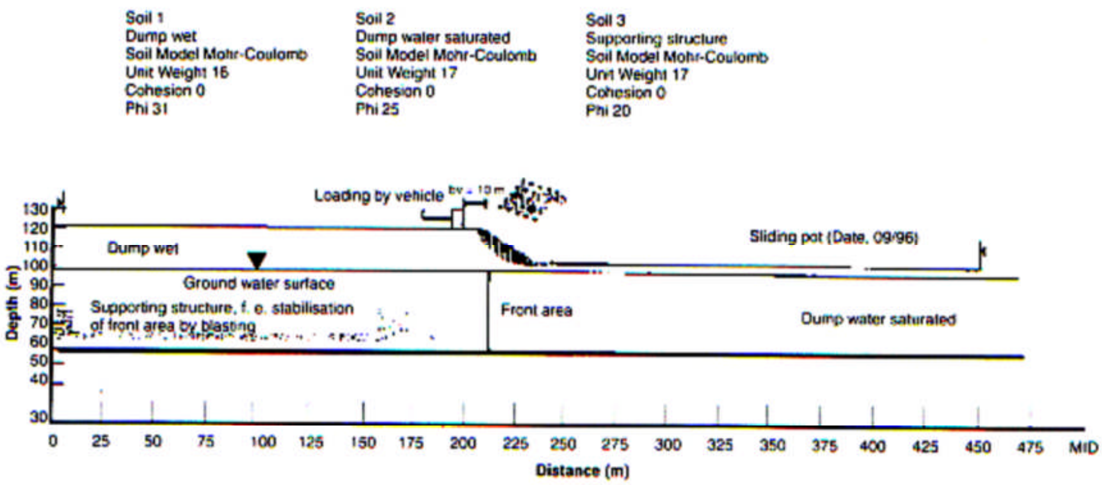

Fig. 5. Calculation model for a dump slope, residual pit Burghammer [2].

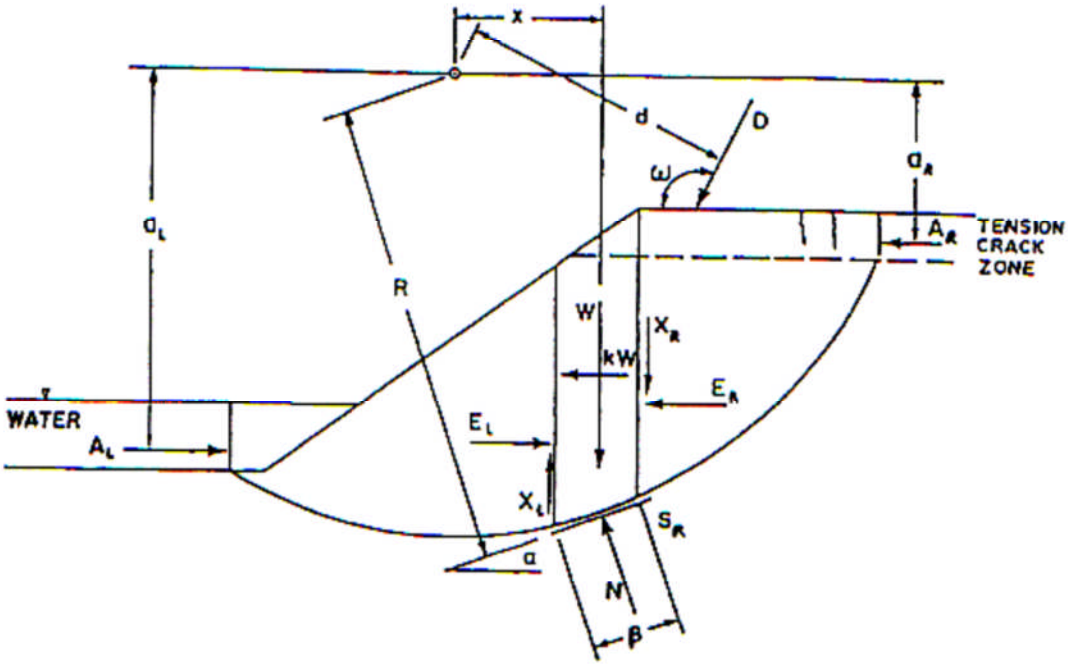

Fig. 6. Safety definition for circular-cylindrical sliding planes.

Three deep-compaction methods and its combinations have been succeeded. These methods are commonly used in civil construction engineering, but their use in rehabilitation mining is something very special because of the great depth to be compacted (up to 70 m with compaction by vibration). For this reason, extreme powerful and mighty machines have to be used.

2.3.1. Compaction by falling weight

This compaction method is very effective for large-scale building-ground improvement of the upper dump sections (up to a depth of 15 m below ground). Recently BUL Sachsen GmbH carries out extensive compaction activities by falling weight at a site which is called "Lusatian Ring", a future Formula 1 and Indycar course (Fig. 7). With this technique, the compaction procedure can be adapted to the soil composition.

The compaction depth can be assessed with [16]:

$$\frac{h_v}{l_r} = a \cdot \sqrt[3]{\left[\frac{1}{10}\frac{m_f \cdot h_f}{m_r \cdot l_r}\right]^{\frac{3}{2}}} \tag{2}$$

where

h_v	compaction depth (m)
l_r	reference length (= 1 m)
α	material parameter
m_f	mass of the falling weight (metric tons)
h_f	falling height of the falling weight (m)
m_r	reference mass (1 metric ton).

The value of the material parameter depends on the kind of the loose rock to be compacted, and it reaches values between 0.5 and 1.0. The value is often higher for dump soils. At compaction works at the dump "Meuro" (direct disposal dump, area of future Lusatian Ring) a value of 1.6 was determined from a kinetic energy of 400 tm and a proofed compaction depth of over 10 m.

Table 2
Strength parameters of loose bed-rocks in the Lusatian area [2]

Loose rock	ϕ' (°)	c' (kN/m²)	ρ (t/m³)
Pleistocene sand and gravel	35	3.0	1.75
Clay	20	10.0	1.90
Brown coal	25	200.0	1.15
Boundary layer coal–clay	16	9.0	–

Fig. 7. Deep soil compaction by falling weight on future "Lusatian Ring" (aerial photo), BUL Sachsen GmbH.

For a description of the compaction modulus it is possible to determine profiles of the compaction moduli from the primary and secondary loading with the plate-loading test on the bore-hole bottom. This also applies for the following compaction methods.

2.3.2. Compaction by blasting

This compaction method is used for a large-scale stabilization of dumps in water-saturated, non-cohesive or low-cohesive loose rocks. According to [4], the distance of the blasting holes can be calculated as following:

$$\alpha = 2 \cdot K_4 \cdot \sqrt[3]{Q} \tag{3}$$

where

- a distance of blast-holes in one set in (m)
- K_4 equation parameter
- Q charge (kg).

2.3.3. Compaction by vibration

The compaction by vibration is an effective deep compaction method for non-cohesive or low cohesive loose rocks, especially in flow-slide-prone areas and at the shore sites of former open-pit holes. BUL Sachsen GmbH recently carries out vibration compaction from a pontoon on the lake "Senftenberger See" for LMBV mbH, which is the responsible enterprise for the rehabilitation of the former lignite open pits (see also Fig. 8).

It is proposed to define the distance of the compaction points as follows [5]:

$$a = K_5 \cdot \sqrt[3]{\frac{J}{t_r^2}} \cdot \sqrt{1 + K_6 \frac{|f_0 - f_{R\ddot{u}}|}{f_{R\ddot{u}} + f_0}} \tag{4}$$

where

- a distance of the compaction points (m)
- K_5 equation parameter
- J moment of inertia of the unbalance mass (kg m²)
- t_r reference dwelling time (0.5 min)
- K_6 equation parameter
- f_0 characteristic frequency of the soil (Hz)
- $f_{R\ddot{u}}$ frequency of the vibrator (Hz).

For the determination of the equation parameters K_5 and K_6, additional tests are necessary [5].

These include the determination of the material parameters α, K_4, K_5, K_6, with which the influence of the

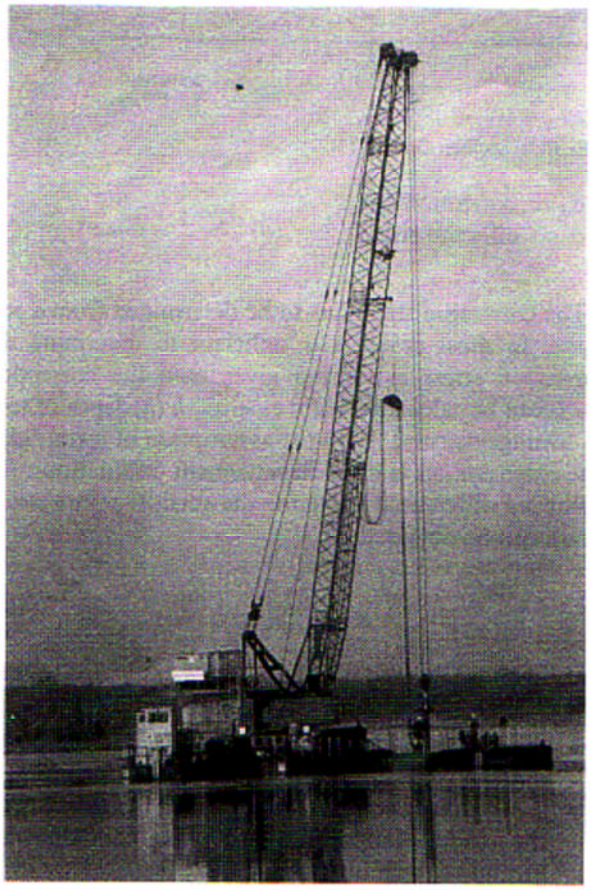

Fig. 8. Deep soil compaction by vibration, stabilization of the shore site of an isle in the lake "Senftenberger See", BUL Sachsen GmbH.

compaction can be determined or with which the bearing layers can be planned safely.

3. Soil mechanical tests

3.1. Objectives of soil mechanical tests

For the assessment of the stability of the dump slopes and for the assessment of the building ground improvement, it is necessary to determine deformation indexes (assessment of the flow-prone layer) and strength indexes (assessment of load capacity as well as the stability) with different soil mechanical tests.

Förster and Lersow [6,12,13] improved a direct procedure, the plate-loading test so that now, amongst others, profiles of deformation-parameters and shear strength parameters of the loose rock can be determined. On this basis, the settlement behavior as well as the load-capacity behavior can be described.

With the help of correlation relations, deformation parameters can also be obtained from cone-penetration tests, but a pre-condition for the use of this method is a calibration at the dump site. It is shown that the plate-loading test can be used as a calibration test for the cone-penetration test.

Deformation parameters for the loose rock in the horizontal plane can be obtained with pressiometer tests. Because of the anisotropy of the dump a transfer to the vertical plane is doubtful. However, it will be shown that pressiometer tests for anisotropic soil dumps are suitable as a complemententary method for PDV-BS.

3.2. Soil mechanical test procedures

3.2.1. Plate-loading test on the bottom of the bore hole

The description of the device can be found in [6]. Fig. 11 shows the probe. As a relatively new procedure, it will be introduced briefly. Further discussion can be found in [12,13].

3.2.1.1. Determination of stiffness modulus with PDV-BS. Fig. 9 shows the pressure-settlement function of a plate-loading test on the bottom of the bore-hole (PDV-BS). Fig. 10 shows the stresses under a loaded circular plane of a virgin half-area. The common interpretation according to the German Industrial Standard DIN 18134 can be found in [12].

Eqs. (5a) and (5b) describe the relationship between deformation modulus and stiffness modulus. Examples for the Poisson's ratios for different loose rocks are given in Table 3 [7].

$$E_S = \frac{(1-\nu)}{(1+\nu)\cdot(1-2\cdot\nu)}\cdot E \qquad (5a)$$

where

E deformation modulus
E_S stiffness modulus
ν poisson ratio.

For $\nu = 0.2\ldots0.35$

$$E_S = (1.2 \ldots 1.6) \qquad (5b)$$

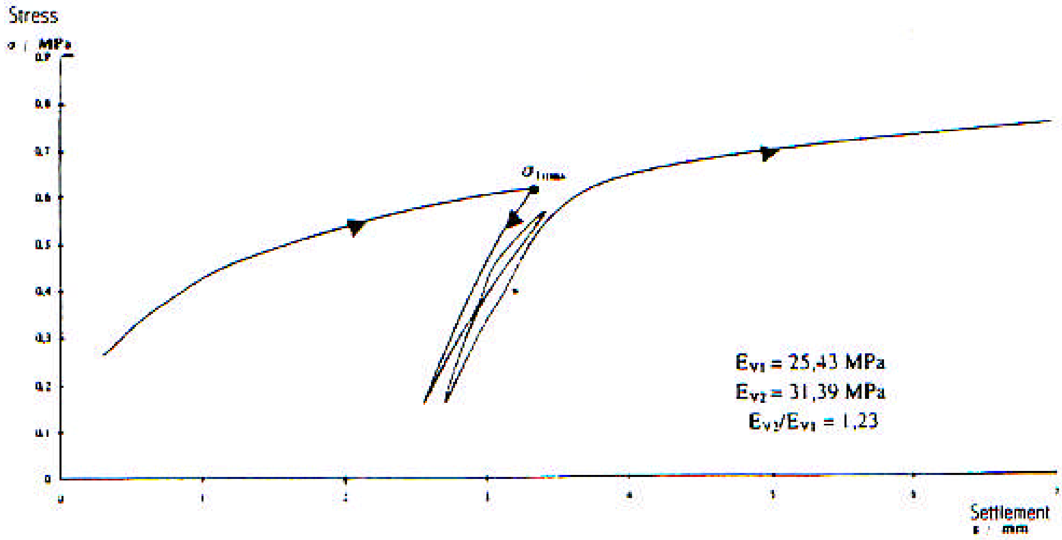

Fig. 9. Pressure–settlement curve, PDV-BS, BUL Sachsen GmbH [13].

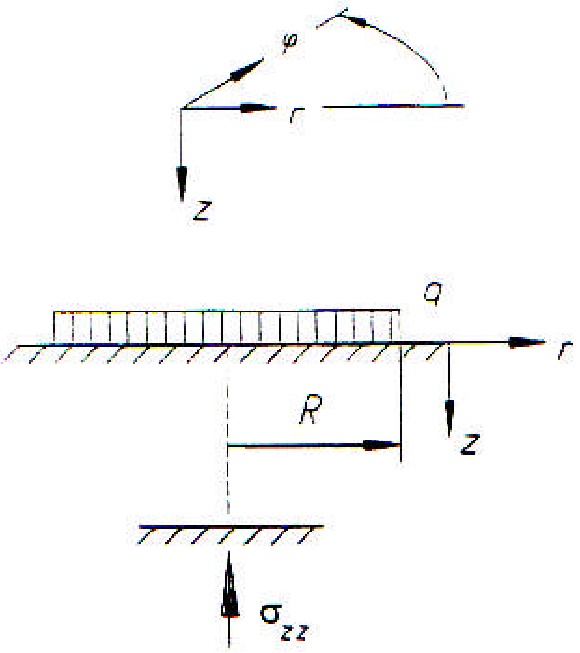

Fig. 10. Stresses under a loaded circular plane according to Poulos and Davis [10].

For the stiffness modulus, the following relation was chosen, which expresses the stress relation of the stiffness modulus [8].

$$E_S = v \cdot p_a \cdot \left(\frac{\sigma' + \sigma_0}{p_a}\right)^w \tag{5c}$$

where

p_a atmospheric pressure (100 kPa)
v coefficient of stiffness
w stiffness exponent
σ_0 correction stress
σ' effective stress.

The correction stress has to be determined from a test curve. In most cases, it is sufficient to determine the correction stress σ_0. For a given case the correction stress can be calculated with: $\sigma_0 = \gamma_m \cdot h$ (h, depth of test; γ_m average density). With the assumption of a restricted side extension, as is usual in settlement calculations, the following differential value for the vertical compression strain can be obtained [9]:

$$d\varepsilon_{zz} = \frac{d\sigma'_{zz}}{E_S(\sigma'_{zz})} \tag{6a}$$

Under the precondition, that:

- the loading with the plate is carried out under drained conditions
- a pre-stress condition is negligible compared with the stresses induced by the plate, the following formula can be obtained:

$$\varepsilon_{zz}(z) = \int_0^{\sigma'_{zz,q}} \frac{d\sigma'_{zz}}{E_S(\sigma'_{zz})} \tag{6b}$$

or

$$\varepsilon_{zz}(z) = \frac{1}{v \cdot (1-w)} \left(\left(\frac{\sigma'_{zz,q}(z) + \gamma_m \cdot h}{p_a}\right)^{1-w} - \left(\frac{\gamma_m \cdot h}{p_a}\right) \right) \tag{6c}$$

The settlement is:

Table 3
Poisson's ratio for different loose rocks [7]

Material	ν
Sand	0.29...0.35
Clay	0.36...0.42

$$\bar{s} = \int_0^{z_G} \varepsilon_{zz}(z)\mathrm{d}z \tag{7a}$$

z_G depth limit, $z_G >= 4R$; in this case $z_G = 6R$ was chosen.

The formula for the calculation under a rigid plate is used. This gives:

$$\bar{s}(q) = \frac{1}{v\cdot(1-w)}\int_0^{6R} \left(\left(\frac{\gamma_m\cdot h}{p_a} + \frac{q}{2\cdot p_a}\frac{1+3\left(\frac{z}{R}\right)^2}{\left(1+\left(\frac{z}{R}\right)^2\right)^2} \right)^{1-w} - \left(\frac{\gamma_m\cdot h}{p_a}\right)^{1-w} \right)\mathrm{d}z \tag{7b}$$

Unfortunately, a closed solution is not available, but, using the Simsons rule for the numerical integration, gives:

$$\bar{s}(q) = \frac{R}{v\cdot(1-w)} \left(\left(\frac{\gamma_m\cdot h}{p_a} + \frac{1}{2}\frac{q}{p_a}\right)^{1-w} + 4\cdot\left(\frac{\gamma_m\cdot h}{p_a} + \frac{7}{50}\frac{q}{p_a}\right)^{1-w} + \left(\frac{\gamma_m\cdot h}{p_a} + \frac{105}{2738}\frac{q}{p_a}\right)^{1-w} - 6\cdot\frac{\gamma_m\cdot h}{p_a} \right) \tag{7c}$$

The parameters v and w have to be determined this way, so that the integral error of the calculated settlement curve $\bar{s}(q)$ against the measured curve $s(q)$ becomes a minimum in the region of the loading, $q = 0 \ldots q_{max}$.

The further solution can be found in [12] and should not be discussed here. The same applies to the determination of the influences on the in-situ determined deformation modulus.

3.2.1.2. *Determination of shear strength parameters with PDV-BS.*

As shown in Fig. 9, basically three regions with different material behaviors can be distinguished during the PDV-BS. The non-linear primary loading branch is valid until a given yield criteria is fulfilled. In this first region, a deformation remains even after disloading.

With further loading, a non-linear plastic yield appears. For loading and disloading, an approximately linear elastic material behavior is assumed. In Fig. 12 the distinction in an elastic s_P^{el} and a plastic part s_P^{pl} of the settlement determined in point P is shown.

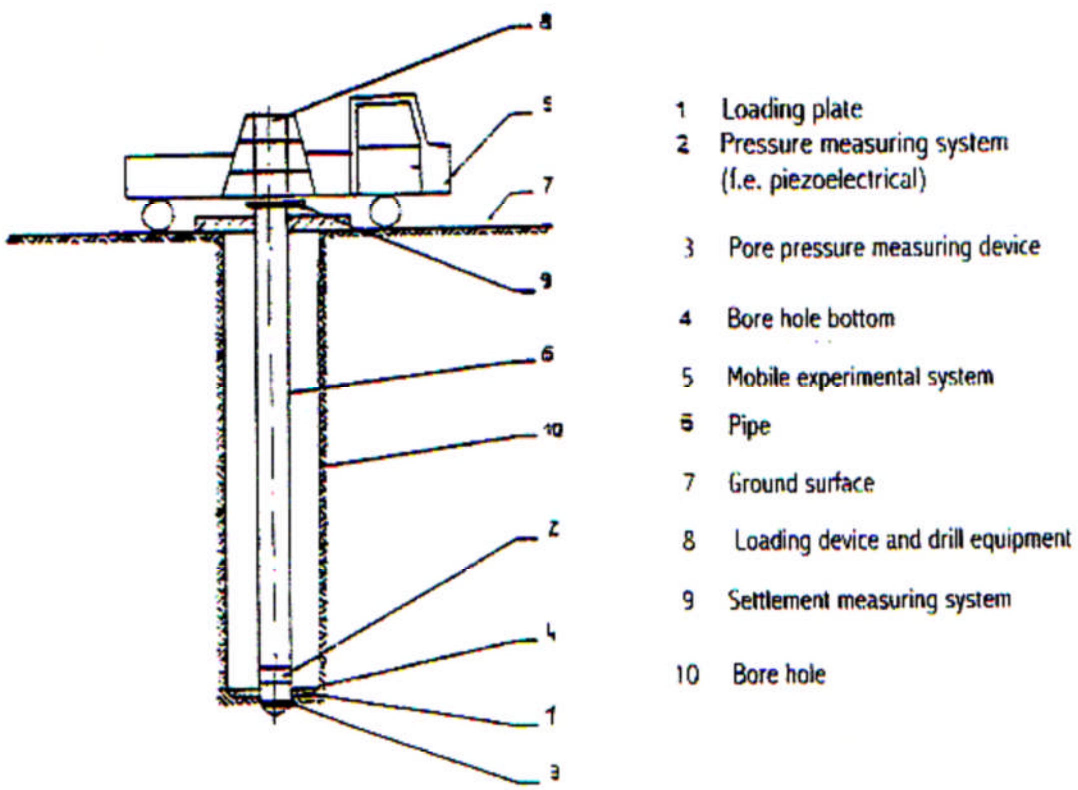

Fig. 11. General set-up of the equipment for the plate-loading test at the bottom of the bore hole [6].

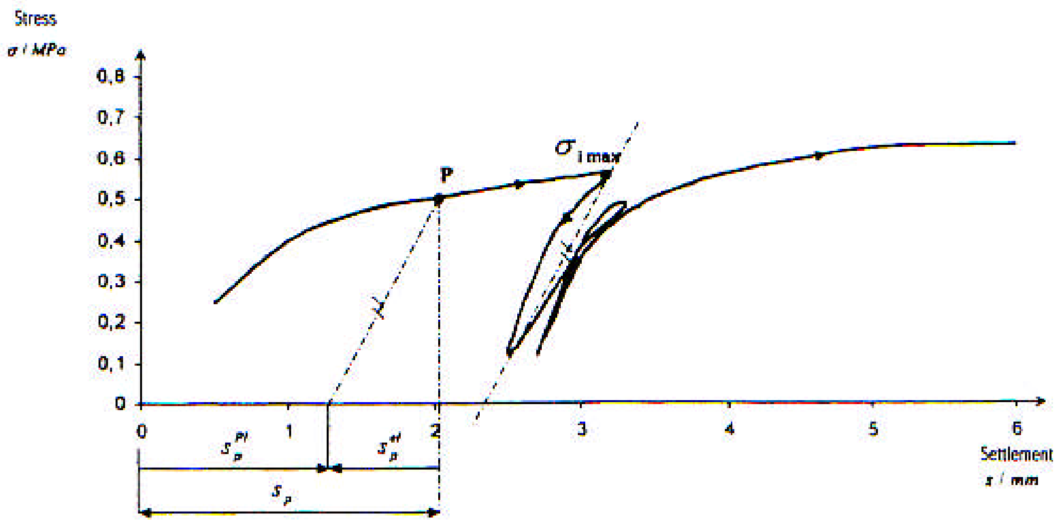

Fig. 12. Division of settlement, measured with PDV-BS at point P [13].

The linear elasticity theory gives the following settlements for an isotropic material under a regular loaded circular plane with the radius R on an elastic half-area (Fig. 10) [10]:

In the middle of the loading-plate

$$s(r=0) = 2\cdot\frac{(1-\nu^2)}{E}q\cdot R \tag{8a}$$

At the edge of the loading-plate

$$s(r=R) = 4\cdot\frac{(1-\nu^2)}{\pi\cdot E}q\cdot R \tag{8b}$$

For the plate-loading test, it is usual to determine a deformation modulus by help of the elasticity theory, according to DIN 18134. According to formula (8a) with $\nu = 0.5$, the deformation modulus is calculated:

$$E_V = 1.5\cdot R\frac{\Delta q}{\Delta s} \tag{9}$$

The following procedure is now offered: with a PDV-BS the pressure–settlement curve of loose rock is determined in the depth h. For the loading steps q_i, the settlements s_i are recorded. From the disloading branch, the disloading modulus can be determined with Eq. (9)

$$E_{\mathrm{Ent}} = 1.5\cdot R\frac{\Delta q_{\mathrm{Ent}}}{\Delta s_{\mathrm{Ent}}} \tag{9a}$$

If one assumes an imaginary disloading in point P (see Fig. 12) of the disloading branch, this will follow a straight line with the inclination of the disloading branch. This gives the elastic part of the settlement in the point P.

$$s_{\mathrm{P}}^{\mathrm{el}} = 1.5\cdot R\frac{q_{\mathrm{P}}}{E_{\mathrm{Ent}}} = q_{\mathrm{P}}\frac{\Delta s_{\mathrm{Ent}}}{\Delta q_{\mathrm{Ent}}} \tag{9b}$$

Now the plastic portion of the settlement s_p can be calculated with:

$$s_{\mathrm{P}}^{\mathrm{pl}} = s_{\mathrm{P}} - s_{\mathrm{P}}^{\mathrm{el}} \tag{9c}$$

Now the question is where the yielding starts. The determination of the limiting load q_0 can be done as follows: Fig. 13 shows the known behavior of loose rocks [11].

The PDV-BS can also prove the material behavior of loose rocks shown in Fig. 13. The inelastic portion of the settlement of the loose rock against the loading is shown in Fig. 14.

The relation between inelastic settlement and loading, which was found by a division of the determined settlement s_p in an elastic portion and an inelastic portion, gives a similar curve to the curve for the change in volume, shown in Fig. 13 [11].

At the reloading branch, no further plastification appears. Plastification first appears if the pressure–settlement curve follows the initial loading branch.

The limiting load, which initiates yielding, is that load above the positive inelastic settlement. Further discussion can be found in [13].

3.2.2. Cone penetration tests

Until now, no common theory has been introduced for cone-pressure tests which takes into account the quantitative influence of loose rock properties on the progress of the peak pressure. For this reason, one often tries to correlate measured peak pressures of the dump material or the building ground to the mechanical properties of the loose rock, but first of all this correlation is unknown. In the German Standard DIN 4094 a correlation relation for the determination of stiffness of non-cohesive and cohesive loose rocks with certain grain-size distribution is proposed, based on in-situ tests in virgin ground and on calibration measurements. A quantitative interpretation shows that this correlation gives insufficient results for cone-penetration tests in dumps. A quantitative assessment requires, therefore, the establishment of specific correlation systems for dumps.

With the help of the established peak-pressure curve, the determination of the stiffness-moduli is possible in two ways [14]: the determination of pressure-dependent stiffness moduli and the determination of depth-dependent stiffness moduli.

3.2.2.1. Derivation of stress-dependent stiffness moduli. For the derivation of stress-dependent stiffness moduli from cone-penetration tests, two correlation relations are necessary:

Relation between vertical stress and peak pressure

$$\frac{q_c}{p_r} = f\left(\frac{\sigma'_V}{p_r}\right) \quad (10)$$

where

q_c peak pressure
p_r reference stress
$f(\ )$ function
σ'_V effective vertical stress.

Relation between peak pressure stiffness modulus

$$\frac{E_S}{p_r} = g\left(\frac{q_c}{p_r}\right) \quad (11)$$

where

E_S stiffness modulus
$g(\ )$ function

Both relations can be calculated with a power series expansion.

3.2.2.2. Derivation of depth-dependent stiffness moduli. The starting point for the determination of the stiffness moduli from cone-penetration tests are the calculated stiffnesses, obtained from settlement-level results for certain depth steps, and from the results from peak pressure curves.

A common procedure would be to calculate the stiffness modulus with a power series expansion of the peak pressure.

$$\frac{E_S(z)}{p_r} = \sum_{i=0}^{n} Q_i \left[\frac{q_c(z)}{p_r + 1}\right]^i \quad (12)$$

The unknown coefficients Q_i can be obtained from test loadings, whereby the peak pressures $q_{c_i}(z)$ as well as the amounts of the settlements s_i can be measured at one point (because of the increase of the effective stresses over the depth as a result of the surface load q). The solution gives a non-linear optimization problem with unknown coefficients Q_i.

One of the simplest solutions would be the introduction of a proportionality factor β, which gives a relation between the peak pressures and the stiffness moduli.

$$E_S(z) = \beta \cdot q_c(z) \quad (13)$$

3.2.2.3. PDV-BS calibration test for cone-penetration tests. Starting from the modified correlation [according to formula (16)], it is proposed to determine β by help of a calibration test with the PDV-BS. The peak pressure is obtained from cone-penetration tests. By help of a regression analysis, $q_c(z)$ can be determined:

$$q_c(z) = a + b \cdot z \quad (14)$$

where

z depth
a, b regression coefficients.

In Table 4, the regression coefficients of cone-penetrations tests at the disposal site Gröbern (southern area of the city of Leipzig) are shown.

Taking into account the results of the PDV-BS, formula (5c), the following relation for the depth z_i is obtained:

$$v_i \cdot p_a \left[\frac{\sigma' + \sigma_0}{p_a}\right]^{w_i} = (a + b \cdot z_i) \cdot \beta_i \quad (15)$$

Now the proportionality factor can be determined

$$\beta_i = \beta_i(\sigma') \quad (16)$$

It is now possible to establish a profile of β as function of the effective stresses for the tested loose rock.

3.2.3. Pressiometer tests

With such a test, the amount of the horizontal stresses and therefore the stiffness of the horizontal plane can be determined.

Because of the anisotropy of the loose rock, this value cannot be transferred to the vertical direction but, in

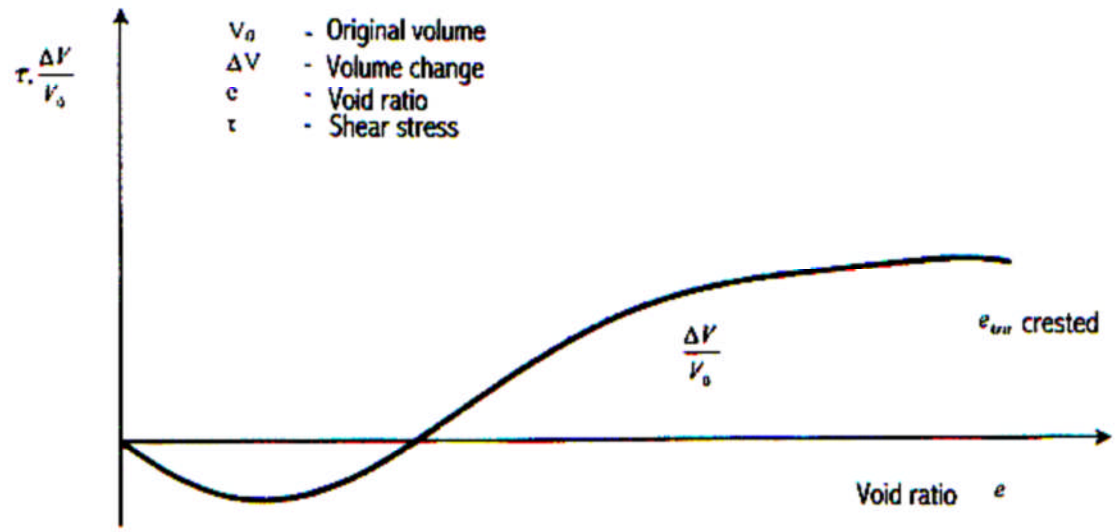

Fig. 13. Volume change at shear test in loose rock [11].

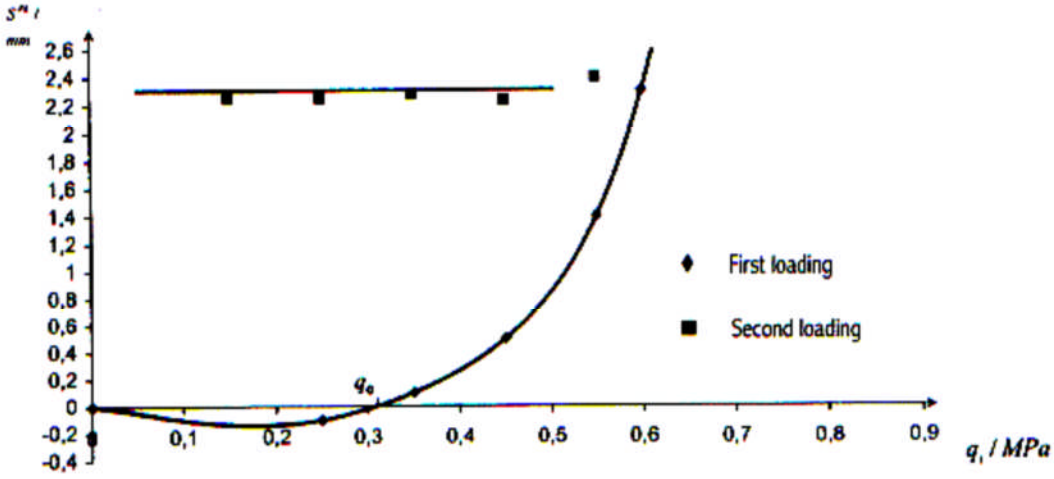

Fig. 14. Inelastic part of deformation at the plate-loading test at the bottom of the bore hole as function of the loading [13].

Table 4
Regression coefficients for cone penetration tests, disposal site Gröbern [15]

DS number	a (MN m^{-2})	b (MN m^{-3})	Correlation coefficient
2–13	−1.01129	0.265345	0.955095
24	2.17634	0.161439	0.689819
24a	−7.87406	0.590386	0.849254
2–6	−0.223683	0.244968	0.885242
2–7	−3.33383	0.125369	0.496726
2–8	−0.470096	0.268112	0.856491
2–9	3.43668	0.175812	0.949680
29a	−0.959411	0.213584	0.932856
30	−0.139756	0.232886	0.724043
31a	2.21298	0.122971	0.765281
34	−6.7668	0.366112	0.932122
35	−2.70202	0.230851	0.706560

connection with other tests, it is possible to prove the anisotropy of the loose rock.

The interpretation of the pressiometer test curves based on the assumption of an isotropic, linear elastic behavior of the loose rock.

The elasticity moduli $E_{e,p}$ and $E_{w,p}$, determined from these pressiometer tests have to be converted into stiffness moduli $E_{S,e,p}$ and $E_{S,w,p}$. A relation between pressiometer moduli and by re-calculation, determined stiffness moduli can be found in formulas (17) and (18):

$$E_{S,p} = f\left(\frac{E_{S,f}}{p_r}\right) \tag{17}$$

$$E_{S,p} = A_0 + A_1\left(\frac{E_{S,f}}{p_r}\right) \tag{18}$$

where

$E_{S,p}$ stiffness modulus from pressiometer tests
$E_{S,f}$ stiffness modulus from in-situ tests
A_0, A_1 coefficients.

3.2.3.1. Pressiometer test as supplement for PDV-BS. With the combination of pressiometer tests for the determination of the horizontal elasticity indexes and the PDV-BS tests for the determination of the vertical elasticity indexes, the degree of anisotropy can be determined. Furthermore, for a transversal-isotropic behavior (see Fig. 15), the following formula is valid:

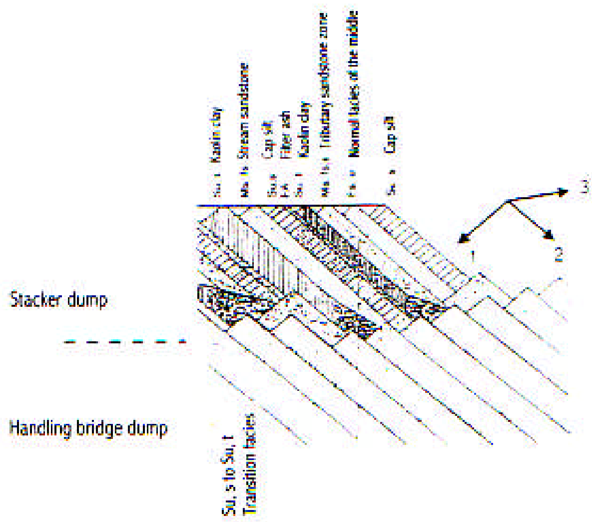

Fig. 15. Transversal isotropic behavior of loose rock of the former open-pit dump "Espenhain" (project Gröbern [15]).

$$2 \cdot \nu_2 < \frac{E_2}{E_1}(1 - \nu_1) \tag{19}$$

where

E_1 elasticity modulus in the isotropic plane
E_2 elasticity modulus in the anisotropic plane
ν_1 Poisson's ratio in the isotropic plane
ν_2 Poisson's ratio in the anisotropic plane.

Dumps are often layered. The Poisson's ratio for the anisotropic plane can be assessed with Eq. (19).

If one assumes the dump as an isotropic body (or slightly anisotropic), then the stiffness modulus determined with PDV-BS becomes an in-situ modulus, which can be used as an input parameter for the cone-penetration test. Consequently, the PDV-BS test can be used as calibration test for the pressiometer test. The coefficients A_0 and A_1 can be calculated according to Eq. (18)

4. Summary

For the stabilization of dumps with the construction of hidden dams and for building ground improvement, for instance for traffic lines over dumps, nearly all applied compaction methods have the aim to reduce the pore volume in the loose rock.

With these methods, a homogenization of the compacted loose rock will be obtained too.

The compaction methods of compaction by falling weight, compaction by vibration and compaction by blasting have been introduced, and their applications and efficiencies have been shown.

For the estimation of the effective depth of the compaction and for a safe planning of the bearing layer, respectively, the necessary material parameters have to be determined for each deep compaction method. Proposals for the determination of these parameters have been made within this paper.

In connection with the stabilization of flow-slide-prone dump slopes, as well as for the improvement of dump areas for the use as building ground, it is necessary to assess the deformation behavior and the bearing capacity. To assess the resulting building ground improvement, deformation indexes (assessment of the flow-prone layer) and strength indexes (assessment of the bearing capacity) have to be determined with soil mechanical tests.

Förster and Lersow [6,12,13] improved a direct procedure, the so-called plate-loading test. With this improved procedure, it is possible to produce profiles of deformation parameters and shear-strength parameters of the loose rock. On this basis the settlement behavior and the bearing behavior of the ground can be described.

The PDV-BS cone-penetration test and the pressiometer test are compared and the reliability of the soil mechanical indexes are assessed critically.

The PDV-BS can be used as a calibration test for cone-penetration tests as well as for the calibration of pressiometer tests. With the application of a PDV-BS and a pressiometer test in combination in a testing field, the anisotropy properties of the loose rock can be proved.

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