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Aspects concerning the shear strength of rockfill material in rockfill dam engineering

The stress-dependent shear strength behaviour of rockfill is frequently estimated by simple shear strength parameter. A linear shear relation between vertical and shear stresses is almost always applied. However, the actual shear strength behaviour of rockfill differs from this theoretical line. A curve is formed which indicates that strong shear strength at low stresses gradually decreases with increasing stress.
The shear strength behaviour is influenced by many aspects, such as grain size distribution, compressive strength of the original rock, saturation, stress conditions, grain shape, etc. Reliable large-scale shear tests are not usually available and a realistic non-linear shear strength function has to be estimated using analytical approaches. Only rarely are case studies available for rockfill dams under dynamic loads induced by earthquakes. However, it is assumed that typical rockfill materials will resist dynamic loads.

Das spannungsabhängende Scherfestigkeitsverhalten von Steinschüttmaterialien wird in der Praxis häufig anhand einfacher Scherfestigkeitsparameter abgeschätzt. Hierbei wird fast immer eine lineare Beziehung zwischen Vertikal- und Scherspannung verwendet. Das tatsächliche Scherfestigkeitsverhalten von Steinschüttungen weicht aber von dieser theoretischen Gerade ab. Es bildet sich eine gekrümmte Linie aus, was besagt, dass hohe Scherfestigkeiten bei geringen Spannungen graduell mit wachsenden Spannungen abnehmen.
Das Scherfestigkeitsverhalten wird hierbei von sehr vielen Aspekten, wie z. B. Kornverteilung, Druckfestigkeit des Ausgangsgesteins, Sättigung, Spannungszustand, Kornform, etc. beeinflusst. In der Praxis mangelt es nicht selten an verlässlichen Ergebnissen von Großscherversuchen und man muss mit analytischen Ansätzen eine realistische nicht-lineare Scherfestigkeitsfunktion abschätzen. Für dynamische Belastungen von Steinschüttdämmen infolge von Erdbeben gibt es nur wenige Referenzfälle, jedoch ist aus Erfahrung davon auszugehen, dass typische Steinschüttmaterialien sich gegen dynamische Lasten als widerstandsfähig erweisen.

1 Introduction

A limit equilibrium analysis (LEA) in the form of a slip circle is calculated in the course of a standard evaluation of the slope stability of large dams and other methods of safety assessment when considering project-specific load cases and safety margins/factors as recommended in Fell et al. [1] in general, or as dictated in national guidelines and codes [2]–[7] and many more. In many cases, country-specific procedures and requirements need to be followed to reflect country-specific experience gathered over decades of dam operation and well-established and verified design guidelines.

The prevailing acceptance and approval parameter for the classic slope stability analysis according to Krey-Bishop or Janbu is the safety factor or the degree of utilization. Strength reduction is a state-of-the-art method, but is mostly applied only in cases when the shear strength parameter and the shear surface are unknown. The global safety concept still prevails internationally, although the partial safety factor concept has considerable advantages thanks to the possibility of directly decreasing or increasing the strength or the loads where uncertainties occur. The partial safety concept has become binding, or at least recommended, in Europe, hand in hand with the European harmonization of the standards and guidelines.

However, the global safety factor concept is based on more than 100 years of experience concerning dams and is therefore a sound evaluation criterion. Nevertheless, in practice, the importance of determining the design shear strength parameter of the materials used is frequently underestimated, and this gives a misleading impression of the actual degree of safety, e.g. of the slope stability of a rockfill dam. In most of the cases when the database for determining the shear strength of the crucial dam fill material is considered to be vague or insufficient, this uncertainty is resolved by selecting a conservative design approach and conservative shear strength parameters. The result is a safe design, but one that still faces an unspecified degree of safety.

For large rockfill dams, the non-linear shear strength of rockfill materials is an essential aspect in the evaluation of the slope stability of most common rockfill-type dams, such as clay-core rockfill dams (CCRD), concrete-faced rockfill dams (CFRD) or concrete-faced sand-gravel fill dams (CFSGD). Asphalt sealing or other methods, such as geomembranes, are less frequently used on large dams due to their limited application range and/or limited experience with them. Thanks to the advantages of geomembrane-lined rockfill dams with exposed linings, the number of these dams will increase in the future [8], [9].

In order to obtain a reliable safety factor, a realistic non-linear shear strength curve should be available for the design of these (large) rockfill dams. Applying other shear strength functions or models such as the well-known

Mohr-Coulomb failure criterion when considering "no load shearing strength" or an "apparent cohesion" [10] may lead to misleading failure surfaces and misleading safety factors. Given the knowledge available, neither of these approaches is satisfactory in terms of modern engineering techniques and methods.

The authors highly recommend large-scale triaxial tests for wet and dry conditions for large rockfill dam projects. The shear strength curves obtained enable the design of a more reliable and economic dam. During certain early project phases, and for some low-budget and fast-track projects, realistic and reliable rockfill shear strength data are not available and need to be estimated based on empirical methods and case study values. Several authors provide a sound database using the results of large- or full-scale laboratory and field tests [10]–[12].

The paper includes the results of large triaxial shear strength tests of the 140 m high Arkun concrete-faced sand-gravel fill dam (CFSGD), which was completed in 2014. After starting construction, a series of large-scale triaxial tests was performed at the Institute for Soil and Rock Mechanics in Karlsruhe (KIT), Germany, in order to be able to optimize the design of the downstream rockfill slope.

2 Basics of rockfill

2.1 Definition of rockfill material

It is difficult to give a clear definition for rockfill material since the rockfill materials and their properties are selected and processed for a specific site and project. In practical terms, materials with coarse particles reaching block (rock) size and a limited percentage of fines are considered to be rockfill. Some authors have provided a rough definition in the form of sieve curve ranges as shown in Fig. 1.

Rockfill is usually quarried and special importance has to be attached to the blasting method and pattern in order to obtain the desired grain sizes and distribution. Weak, weathered rocks can be excavated by jack hammer or ordinary excavators. The layer depth gives a practical limit to the maximum block size. Frequently, the layer depth equals the maximum block size, although geotechnical specifications dictate a limit of approximately half the layer depth. This practice is originated in roadway construction guidelines and does not consider the special conditions of (large) rockfill dams.

Rockfill materials should be "free-draining" in the sense that the material shows an extreme high conductivity/permeability k, which is generally at least $> 10^{-3}$ m/s for fine rockfill and $>> 10^{-1}$ m/s for coarser material. To achieve a high permeability, the content of fines is usually limited to max. 7–10% (diameter d = 0.0074 mm). Frequently, the amount of sand is also limited to < 20% (see Fig. 1). However, the design of rockfill dams should comply with modern seepage design requirements guaranteeing safe control of seepage conditions and preventing unfiltered exit of the seepage along the downstream slope.

Contrasting with the aforementioned "clean" rockfill materials, "dirty" rockfill may lack the "free-draining" characteristic. Sandstones, siltstones, shales and schists are likely to exhibit a considerable amount of fines after compaction albeit providing reliable strength and deformation values. This paper mainly addresses "clean" rockfill, since "dirty" rockfill may exhibit a different behaviour when the rockfill matrix is dominated by the fine material.

Natural deposits of sand-gravel fills are also utilized as dam fill materials and are used more or less in the same way as rockfill materials as river deposits frequently represent a more economic material source than quarried rockfill. The number of dams utilizing sand-gravel deposits is increasing worldwide, building on the successful experiences of forerunners such as Kremasta Dam (height H = 160 m) in Greece. The largest sand-gravel fill dam (CFSGD) worldwide is the Aguamilpa Dam (H = 187 m) in Mexico [13], which was completed in 1993. However, this paper will not distinguish between rockfill and sand-grav-

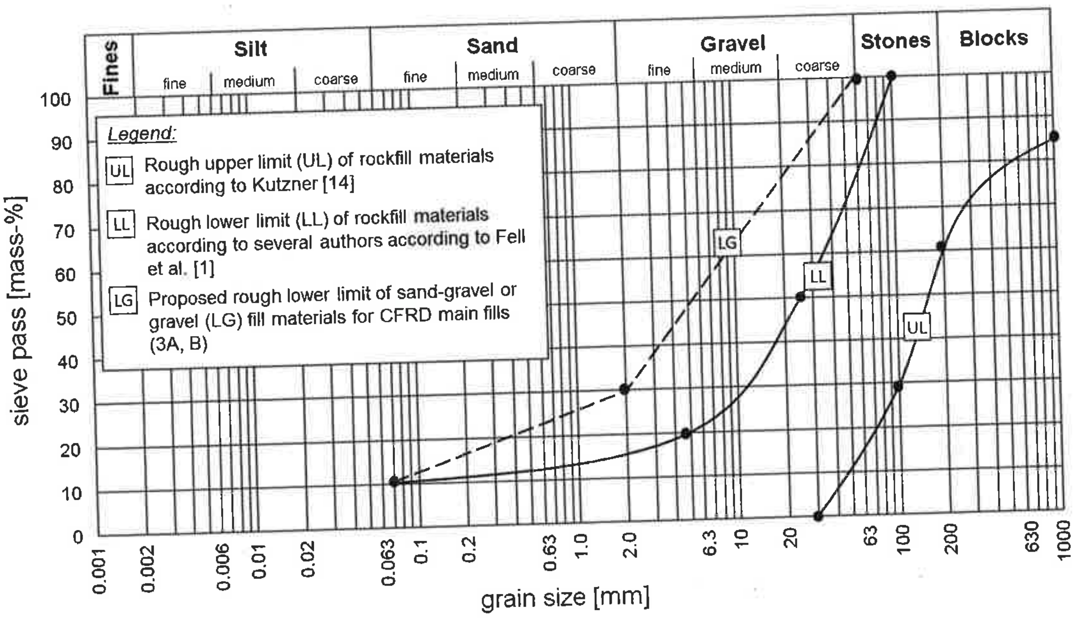

Fig. 1. Rockfill sieve curve according to [1] and [14]
Bild 1. Sieblinie von Steinschüttmaterialien nach [1] und [14]

el fill except when the particular differences of both materials are discussed.

In this context, it should be noted that sand-gravel fills are not usually considered to be free-draining. This needs to be considered in the seepage control design of a fill dam. To ensure a strong and permeable material, a lower boundary for sand-gravel fills is proposed in Fig. 1.

Other properties of the rockfill material, such as compressibility, have to be evaluated within the complete dam design, performance and the particular technical specification of the filling procedure, e.g. number of passes, watering during construction, compaction equipment and speed and construction layer thickness. The shear strength behaviour is discussed in detail below, whereas the other characteristics and aspects are discussed only in brief.

2.2 Constitutive models for shear strength curves

Three types of model – empirical, elastic and plastic – are generally distinguished in the literature. For static problems, applicable software programs based on finite element modelling usually implement different plastic models.

An overview of the various models is given in [15]. For rockfill materials, the well-known Mohr-Coulomb (MC) failure criterion, which defines a linear failure curve, is frequently used:

$$\tau = c + \sigma \cdot \tan(\varphi) \tag{1}$$

where:
- τ shear strength or shear stress
- c cohesion
- σ normal stress
- φ friction angle

Although coarse-grained materials do not usually exhibit any cohesion, dam engineers frequently "correct" the MC curve to improve the match with the non-linear actual shear stress behaviour of rockfill (see section 3) by providing a higher shear strength at lower stress levels.

In order to be able to consider the stress-dependent decrease in the shear strength of rockfill, an exponential formula as shown below [12] can be used:

$$\tau = A \cdot (\sigma)^b \tag{2}$$

where A and b are free parameters.

The parameters A and b do not have a physical meaning but are dependent on the stress unit considered (kN/m^2 here). Higher values also result in stronger shear stress curves. A range of 0.67 to 0.81 is used for the b parameter [16]. Typical curves for selected values of parameters A and b are shown in Fig. 2.

Eq. (2) enables easy modelling of non-linear stress-shear curves based on only a few sample results at various stresses and is therefore frequently used in dam engineering and also in this paper (see Figs. 6 and 9). The adaptation of both parameters A and b provides high flexibility for modelling the shear stress curves correctly.

Alternatively, the effective friction angle can be calculated decreasing with the increase in stress. Using

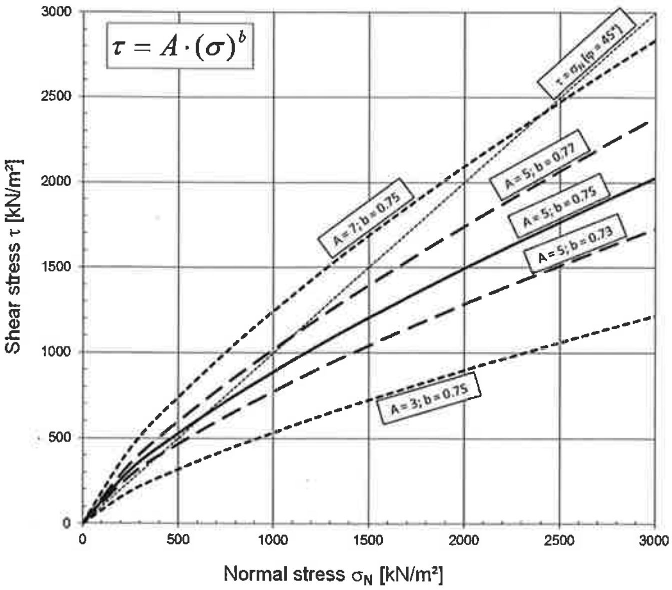

Fig. 2. Shear stress curves according to the power law given by Eq. (2) for different values of A and b [17]
Bild 2. Scherspannungskurven gemäß dem Potenzgesetz nach Gleichung 2 für verschiedene Werte von A und b [17]

Eq. (1), the friction angles given in Eq. (3) can be converted to shear stresses:

$$\varphi = \varphi_0 - \Delta\varphi \cdot \log(\sigma) \tag{3}$$

where:
- φ effective friction angle
- φ_0 friction angle at zero or low normal stresses
- $\Delta\varphi$ differential friction angle that decreases the effective friction angle corresponding to the increasing normal stresses σ

In Eq. (4), the secant angle of the shear resistance φ' is determined according to [18]:

$$\varphi' = \varphi'_B + \frac{\Delta\varphi'}{1 + \frac{\sigma_n'}{p_N}} \tag{4}$$

where:
- φ'_B basic angle of friction that reflects friction close to the critical state
- $\Delta\varphi'$ maximum angle difference
- σ'_n effective normal stress on failure plane
- p_N median angle pressure that reflects the strength or resistance of grains to crushing

Eq. (4) was developed to model non-cemented soils with zero cohesion showing a non-linear failure envelope.

The same is valid for the following expression used, e.g. in [19], which uses a material constant that defines the inclination of a linear function to express the decrease in the effective friction angle φ' with increasing normal stresses σ_n:

$$\varphi' = \varphi_{max} - a \cdot \log\left(\frac{\sigma_n}{\sigma_{n0}}\right) \tag{5}$$

where φ_{max} corresponds to the maximum value of the friction angle at zero or low stresses, which is also defined as φ_0 in Eq. (3), and σ_{n0} is a reference stress parameter defined as 0.3×9.81 kN/m³.

Barton [20] also developed an analytical method to determine the shear strength τ of crushed compacted or dumped rockfills:

$$\tau = \sigma_N \cdot \tan(R \cdot \log \frac{S}{\sigma_n} + \varphi_r) \tag{6}$$

where:

σ_n normal stress on failure plane
R dimensionless equivalent roughness parameter
S empirical stress parameter that takes into account the particle size of and the uniaxial compressive strength (UCS) of the rockfill
φ_r residual friction angle

Barton [20] developed his formula for the determination of peak shear strength values.

The literature provides a huge number of rockfill material case studies that supply engineering parameters for the equations mentioned [11], [12].

2.3 Laboratory and field testing methods for rockfill strength

2.3.1 General

The size of the rockfill and sand-gravel fill materials makes it difficult to perform extensive or even standard test programmes as, for example, are standardized for road works, where soils with a maximum grain size equivalent to that of gravel are generally used.

Special test equipment is required for testing coarse-grained soils. The equipment is quite expensive to develop, construct and maintain. As discussed later in this article, site-specific rigs are often constructed and operated for the period of rockfill placement only.

Many large-scale triaxial tests were performed in South America in the course of the construction of large rockfill dams over recent decades. A large-scale triaxial test rig constructed in the 1960s is still in use at the Institute for Soil and Rock Mechanics (IBF), Karlsruhe Institute of Technology (KIT) [21]. It is the only one of its kind in Western Europe.

It is especially difficult to obtain the shear strength at low or zero stress and at extremely high stresses. To obtain the angle of repose φ_{RP} [°] at the zero stress state, special equipment is required. This is not standard laboratory test equipment for any institution, university or laboratory. Corresponding tests were performed by utilizing site equipment [20].

Large-scale dynamic testing is restricted with regard to the specimen size and the value of the results. In [22], a cell with a diameter of 250 mm was used for dynamic tests.

Ref. [4] contains this statement: "Strengths [of] rockfill and other coarse material are commonly based on a review of [the] literature since these materials are difficult to test in the field or laboratory." Hence, for many rockfill dam projects, the emphasis is not on proper testing, but on dealing with the design uncertainties through a conservative design.

2.3.2 Large-scale triaxial cells

Testing in a laboratory has the advantage that the test can be performed under defined, stable and controlled conditions. The disadvantage is that the dimensions of the test equipment dictate the maximum grain size of the test material. Further, the material needs to be transported to the laboratory, which frequently takes days or weeks. During transportation, the material may be subjected to dynamic loads, a certain amount of breakage and a change in the moisture content. This alters the materials compared with the application conditions on site.

In the laboratory, consolidated drained (CD) tests are generally performed. They usually reflect the common site and load conditions of rockfill materials in terms of free drainage, which may not be the case for very weak rockfill extracted from shales or clay and siltstone rocks.

Generally, the maximum grain size should be ¼ of the diameter of the test rig or cell. Otherwise, the boundary influence is considered to be unacceptable. Charles & Watta [16] cite different sources that also consider a factor of six between the test material size and the actual material, which would make the test specimen smaller. Either the maximum grain size of the material is reduced considerably, or the triaxial test cell is large enough for this purpose. In practice, a maximum grain size of 100–200 mm is generally used for large-scale triaxial tests.

Fig. 3 shows selected sieve curves for test fill materials, all of which were scaled down to a maximum grain size of 100–200 mm. Corresponding to the rockfill dam and its specific design, the maximum grain size should usually be smaller than the layer depth, which is nowadays practically 1.0–1.5 m – let's say, ⅔ of the layer depth so the maximum grain size of the rockfill is theoretically in the range 0.4–1.0 m. The maximum grain/block size is also frequently selected as equal to the layer depth for economic reasons, and thanks to the limited occurrence of single large blocks does not influence the compaction works or the general characteristics of the rockfill. Protruding single large blocks are removed manually before compaction.

For many rockfill dams, the stress–strain curve in shear for the rockfill materials used are not determined by large-scale triaxial testing. Frequently, normal-size triaxial tests are used for granular materials. A minimum sample diameter of 10 cm is recommended for this [23], thus allowing a maximum grain size of 2.5 cm, which represents medium to coarse gravel material.

Testing downsized material does not allow accurate determination of the interlocking effect of larger particles at low stresses and the general effect of larger particles on the shear strength with regard to particle breakage and interlocking.

Marsal [11] lists several large-scale test rigs suitable for rockfill testing which were used during the El Infiernillo Dam construction and were later handed over to a laboratory in Mexico. Whether the equipment is still in use is not known to the authors. The triaxial cells described had a diameter of 1.13 m and a height of 1.8 m. Abbas [24] used a cell with a diameter of 0.38 m and a height of 0.81 m. In [25], test specimens with a diameter of 0.381 m and height of 0.813 m were tested in New Dehli. At the Insti-

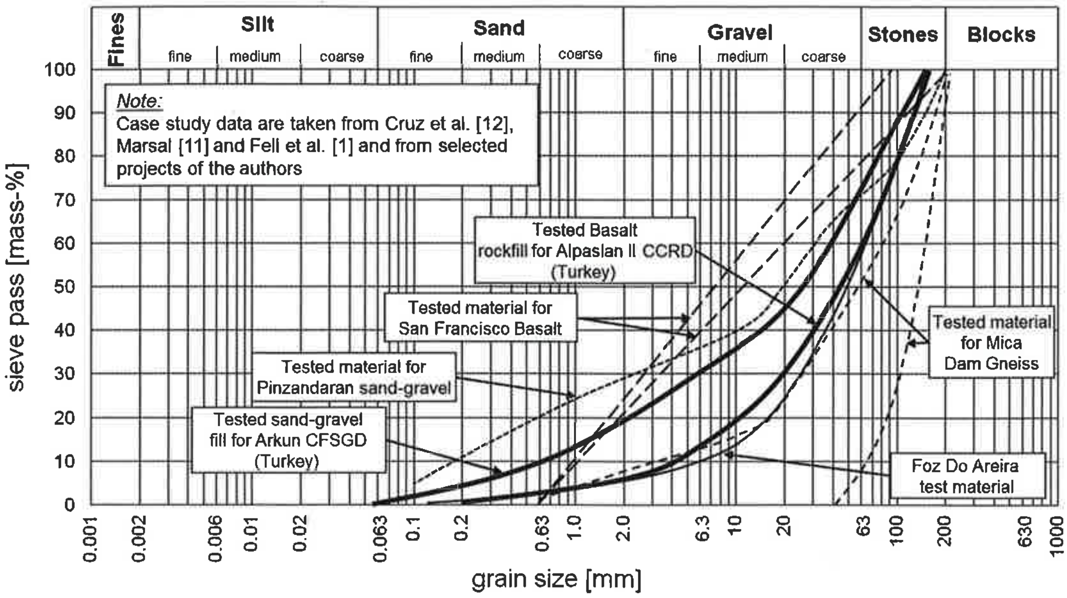

Fig. 3. Sieve curves of large-scale triaxial test materials for the rockfill and sand-gravel fills of selected projects
Bild 3. Sieblinien von Steinschütt- und Sand-Kies-Schüttmaterialien für großmaßstäbliche Triaxialversuche von ausgewählten Projekten

tute for Soil and Rock Mechanics at Karlsruhe Institute of Technology (KIT), the test cell has a diameter of 0.80 m and a height of 0.80 m, which limits the maximum grain size to 150–200 mm [26]. In [16], a large triaxial cell used at the University of California is suitable for a sample size of 0.9 m dia. × 2.3 m high. A cell with a diameter of 30 cm was used for tests in Japan [19].

The authors do not have an overview of Chinese research and test activities since the number of papers and projects outnumber those of the rest of the world [27].

The results of triaxial tests are subjected to several boundary conditions such as loading rate, cap friction, compaction and placement of the material in the laboratory. For large-scale rockfill testing, the compaction achieved in the laboratory is usually less than that obtained on site due to the lower compaction energy. Similar aspects are valid for large-scale oedometers, shear boxes, etc.

2.3.3 Large-scale field tests

A site test apparatus was used in [20]. In the main, large steel boxes or containers are sheared using only gravity and some lifting equipment. From this it is possible to determine both the angle of repose φ_{RP} and low stress friction angle φ_0, which is usually defined as the friction angle at low stresses which is applicable for ordinary testing procedures.

The angle of repose can also be estimated from slopes of trial embankments by simple measurements. For this purpose, the compaction works should also include the slope area similar to that in the future main fill.

Yamaguchi et al. [28] employed a simple construction in order to determine the angle of repose. The apparatus itself was just a container large enough not to show an effect on the compaction of the rockfill once it is buried beneath the rockfill itself. After excavation, the container is lifted. As soon as the surface particles move, the critical stage is reached and the angle is measured and defined as critical.

Another way to test the shear strength of rockfill material on site is described in [29]. A series of large steel frames is buried and later excavated down to the assumed shear surface. After placing a corresponding weight on the frame, it is pulled via a chain attached to a jack. The pulling force and the deformation are monitored and shear point data is obtained. The shear frames were rectangular with a side length of 1.225 m and a depth of 0.20 m. The layer thickness of the test fill was 0.40 m.

3 Shear strength of rockfill and sand-gravel fill

3.1 General

The shear stress behaviour of rockfill is controlled by many factors and effects that cannot be covered by laboratory or field tests. The ageing of rockfill is an important time-dependent aspect that is difficult to predict [30], [31] and not usually taken into account explicitly in the design of a dam. Long-term studies are rare and interpretations of ongoing long-term processes are difficult due to their slow evolution [32]. Nevertheless, a guideline was prepared in the USA which determines the ageing of materials to be a central issue that also dictates the definition of design shear strength parameters.

In fact, the shear stresses do not develop in proportion to the actual stress level. It is more of a curve as illustrated in Fig. 4. The stress path, environmental impacts and site construction conditions play an important role that is frequently "only" the subject of engineering judgment in terms of defining the design rockfill strength and its ageing throughout the service period of a dam. The time-dependent behaviour of the shear strength is not discussed in detail in this paper. However, it should always be considered when defining the design shear parameters that fix the theoretical deformation behaviour of the rockfill dam.

This paper concentrates on the shear stress behaviour until peak strength conditions are reached. Post-fail-

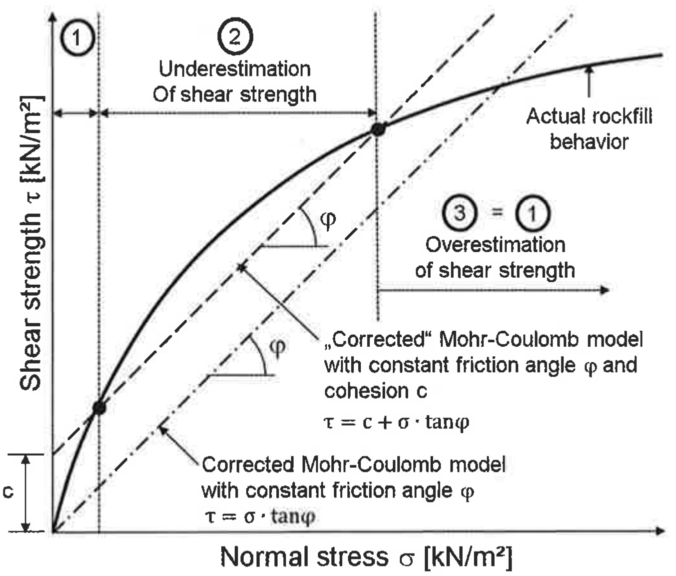

Fig. 4. Shear stress behaviour of coarse-grained soils compared with Mohr-Coulomb straight line
Bild 4. Scherspannungsverhalten von grobkörnigen Böden verglichen mit der Mohr-Coulomb-Gerade

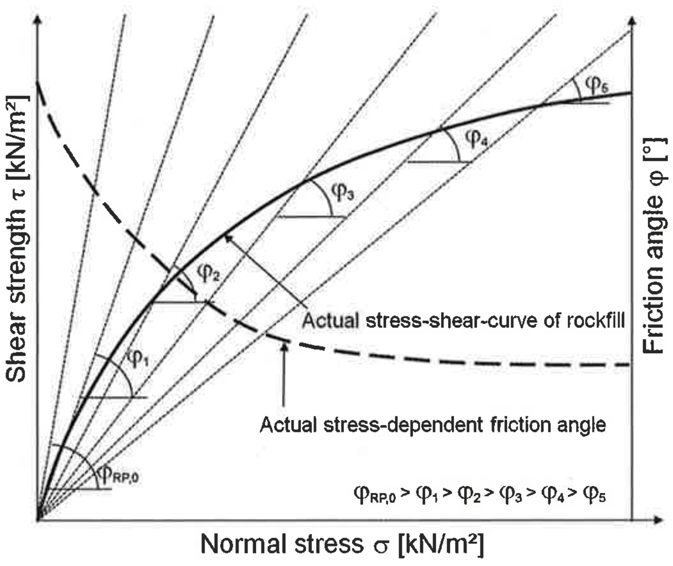

Fig. 5. Behaviour of effective friction angle of rockfill material with increasing stress
Bild 5. Verhalten des effektiven Reibungswinkels von Steinschüttmaterialien mit steigender Normalspannung

ure or residual strength conditions are not discussed, although sections 3.4 and 3.5 discuss design aspects that also refer to residual strength issues as well as the post-failure behaviour of rockfill materials.

The stress-dependent behaviour of the shear strength can be separated into three stress phases. The stress level is defined qualitatively and is totally different for different rockfill types (Fig. 3).

1. Low stresses: High friction due to interlocking of the particles; the angle of repose of rockfill and sand-gravel reaches values exceeding $\varphi_{RP} \geq 70°$. The large particles and blocks govern the shear strength. The interlocking effect is lower for sand-gravel fill.
2. Medium stresses: Friction decreases primarily due to large particles and blocks breaking and edges bursting.
3. High stresses: Breakage is at maximum. The shear strain reaches a peak. The curve transfers to softening or hardening, corresponding to several other aspects such as compaction, void ratio, stress conditions, etc.

The low stress phase also covers the zero stress state, which, as mentioned previously, is special in terms of testing and evaluation.

The "effective" friction angle decreases with increasing normal stress and decreasing interlocking effect and increasing breakage. With increasing shear stress, the effective friction angle decreases as shown in Fig. 5.

The angle of repose is the friction angle at zero stress. It cannot be determined by laboratory testing, but particular site tests can shed light on this parameter. Usually, the angle of repose and the friction angle at low stress levels are set as equal, $\varphi_{RP} \approx \varphi_0$. The near-surface shear strength is usually only important when considering near-surface failure and ravelling. Neither is critical for the global stability of slopes. If not considered correctly within a slope stability analysis, it is necessary to discuss whether the sliding surface is the critical one.

3.2 Classified non-linear rockfill shear strength curves

Considering the shear stress models given in section 2, the authors try to provide rough and cautious classifications for weak to strong rockfill shear strength curves for validating, checking or benchmarking the inputs of limit equilibrium slope stability analyses.

Classifications of the shear stress curve and the input parameters A [–] and b [–] for Eq. (2) are proposed in Figs. 6 and 7. The database is taken from [12], which refers to data obtained from different researchers and engineers such as [11] and many more. For Fig. 6, four rockfill strength classes – strong, medium, weak and minimum (very weak) – are introduced. The curve classified as "minimum" is quite close to the lower border proposed by [11], whereas the "strong" curve does not reach the theoretical upper border defined by [20]. The authors could not find data that confirms this upper border.

The curves given in Fig. 6 do reflect the relative wide range of actual rockfill material behaviour. The transition from one to the other is fluent and a careful and cautious engineering judgement approach is needed in order to estimate a non-linear shear stress behaviour from only very limited data.

The classification provided can also be introduced for parameters A and b (Fig. 7) if Eq. (2) is considered. Parameter b can be selected depending on parameter A. However, the limited database results in a quite large variation as indicated by the dashed lines. Clear separation of the defined classes is not possible and is physically unreasonable. Fig. 7 only shows three of the defined rockfill strength classes. The lower limit given in Fig. 6 corresponds to the parameters A = 0.90 and b = 0.87.

For Eq. (3), the friction angle at low stresses φ_0 versus the decreasing incremental friction angle $\Delta\varphi$ is given considering the classification described. Once φ_0 is selected, maybe as the angle of repose φ_{RP}, the stress-dependent shear curve can be estimated after $\Delta\varphi$ is determined from

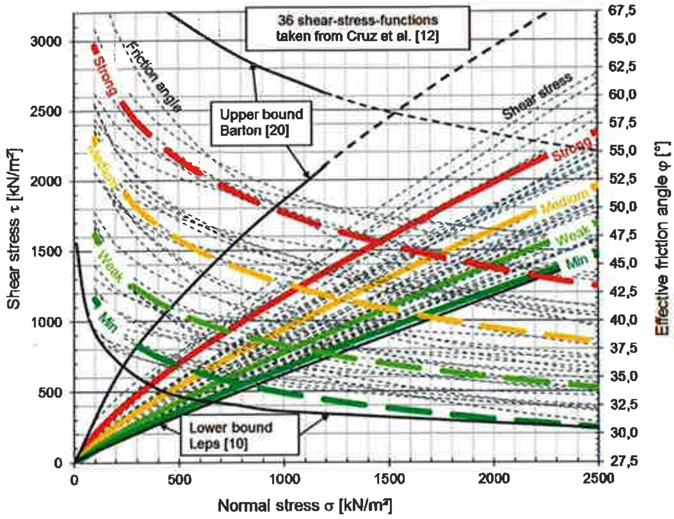

Fig. 6. Classification for shear stress behaviour on the basis of selected experimental data [12] etc.
Bild 6. Klassifikation des Scherspannungsverhaltens auf Basis von ausgewählten Versuchsdaten [12] etc.

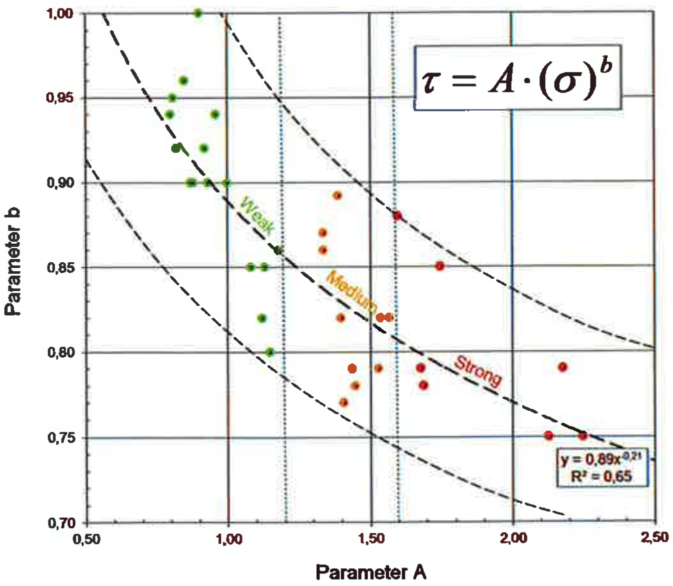

Fig. 7. Classification of the dimensionless parameters A and b for Eq. (2) (parameters A and b are valid for unit kg/cm^2 only)
Bild 7. Klassifikation der dimensionslosen Parameter A und b für Gleichung 2 (Die Parameter A und b sind nur für die Einheit kg/cm^2 gültig.)

Fig. 8 for the different classes. Again, the transition from one class to the other is fluent. For the material classified as "strong", the case study data range is again limited (Fig. 8). In this context, the classification is well defined taking into acccount the term (φ_0 - $\Delta\varphi$) mentioned in Fig. 8.

In Figs. 6, 7 and 8, single alluvial gravel fills, generally classified as weak, were sometimes considered as medium rockfill in terms of their shear strength.

3.3 Sand-gravel fill data

Data for sand-gravel fill material are rare. In most of those rare cases, Mohr-Coulomb effective shear strength parameters (φ', c') are available from data in the literature. The shear stress curves are seldom determined, since historically, this material was rarely used as dam fill material for large dams, and testing activities were limited. For example, for Aguamilpa, the world's largest CFSGD, the stress-dependent shear curve was not determined, but MC peak and residual shear strength parameters were determined [13]. Further works were performed, e.g. [25], [33].

Fig. 9 shows selected sand-gravel fill shear strength curves. including the Arkun CFSGD laboratory tests performed at KIT in Karlsruhe, Germany.

Typical "clean" rockfill and sand-gravel fill exhibit different characteristics, as already mentioned in this paper. Despite those differences, sand-gravel fills may perform as strong as medium rockfill materials regarding their shear stress performance as classified in section 3.2 of this paper.

3.4 Design strength of rockfill in dams for static load cases

There is no document in the sense of a "how-to manual" [4] for determining the design shear strength of rockfill. Finally, the design shear strength always needs to be the result of a comprehensive discussion considering the available materials, the available data, the design philosophy, the dam design itself, the critical load cases and the long-term durability of the materials, all evaluated by

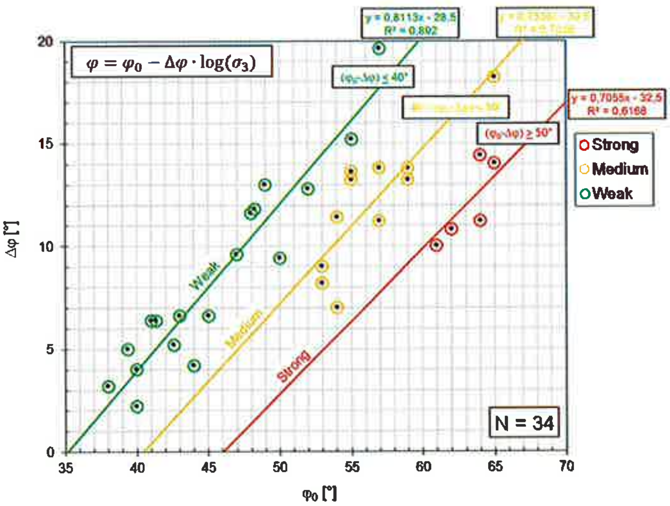

Fig. 8. Classification of φ_0 and $\Delta\varphi$ for rockfill materials depending on the rockfill strength classification strong, medium and weak with reference to Eq. (3)
Bild 8. Klassifikation von φ_0 and $\Delta\varphi$ für Steinschüttmaterialien unter Beachtung der Steinschüttfestigkeitsklassifikation hart, mittel und weich mit Bezug auf Gleichung (3)

well-educated and experienced experts. Certain aspects of the design process cannot be defined stochastically or deterministically and still need to be decided on the basis of empirical data and by applying engineering judgment.

Applying unclassified average rockfill shear strength values, e.g. for the case that the quarry area has not been identified, may be misleading in terms of safety and requires verification during the later project phases. Some authors, e.g. [10], [34], provide "average" shear strength curves for several rock types by considering numerous test results. Judging from this, the average friction angle for

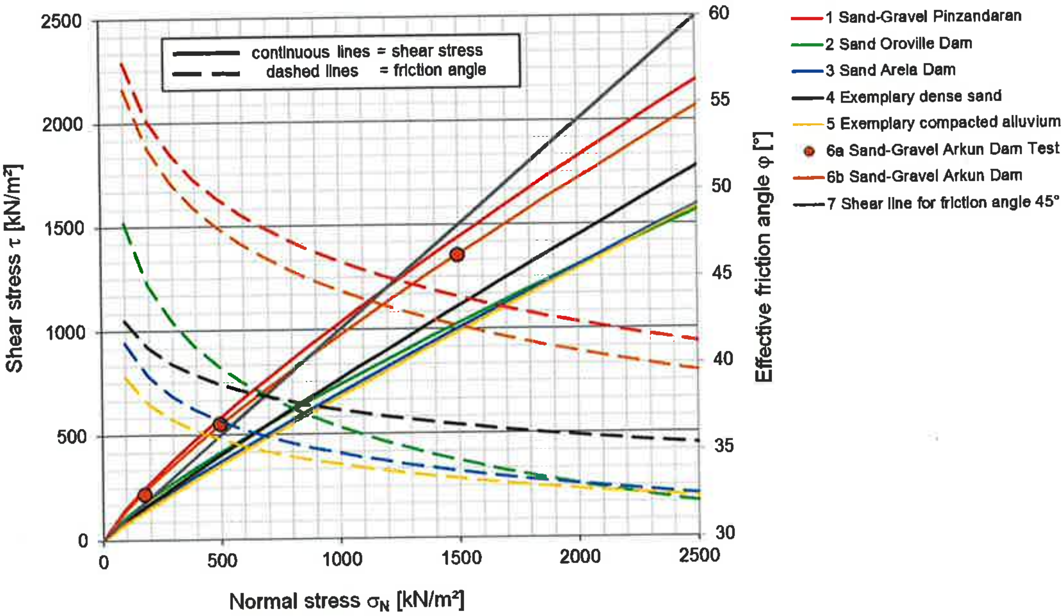

Fig. 9. Normal vs. shear stress curves and the corresponding effective friction angle curves for selected sand and sand-gravel fill materials
Bild 9. Normal- und Scherspannungskurven und die entsprechenden Kurven des effektiven Reibungswinkels für ausgewählte Sand- und Sand-Kies-Schüttmaterialien

rockfill at a stress of 1 MPa may not be less than φ = 40–45° for at least medium-strong rock types.

Alternatively, Figs. 6, 7 and 8 can also be considered for defining a non-linear shear strength curve without having detailed information about the future dam fill material, but by knowing the type of origin rock, its general compressive strength and the degree of weathering. Sand-gravel fills can be treated as weak to medium rockfill in terms of their shear strength behaviour.

Okamoto [19] admits that a design friction angle φ > 40° might be not misleading for typical rockfill dams and materials assuming a critical sliding surface at a depth of 20 m and loose compaction conditions.

Generally, linear shear strength curves resulting from Eq. (5) are applied to loose to very dense conditions. For pseudo-static earthquake slope stability analyses, a friction angle φ = 40–45° should also be applicable for Japanese conditions assuming that the residual strength also shows similar values. Shale and slate have to be considered in a different way.

Lazanyi [35] refers to tests performed on decomposed granite which were taken as the basis for defining linear MC design curves taking into account cohesion and friction as well. The overestimation of the shear strength at small stresses is acceptable if near-surface failure is excluded. At high stresses, the parameters should be selected to underestimate the actual shear strength in order to be conservative and safe.

For small dams, the slip surface is located within stress zones 1 and 2 according to Fig. 4. Hence, simple MC parameters result in a conservative analysis and design. Nevertheless, if the corresponding testing is done and the results are available, direct shear strength curves should be used for design. For large dams where the stress level is in zone 3 according to Fig. 4, the consideration of a shear strength curve is recommended since it will lead to the most economic and safe design in the course of a static slope stability analysis.

Test results and actual or predicted stress and seepage conditions within the rockfill dam need to match. For this purpose, selected tests in the form of large-scale CD (consolidated drained), CU (consolidated undrained) or UU (unconsolidated undrained) tests are required. To obtain a realistic shear strength curve for free-draining rockfill, CD tests are usually sufficient. As mentioned before, saturation conditions critically influence the shear strength. Saturated rockfill may be considerably weaker than dry rockfill. This is important for rockfill dams that allow seepage through the dam body or parts of it, such as in clay core or asphaltic core rockfill dams. A risk analysis is also recommended to shed light on the possibility that the rockfill material is saturated and its shear strength lower. This analysis goes hand in hand with the evaluation of the failure of the main sealing or drain body. German codes consider this kind of failure to be as mandatory.

As mentioned before, ref. [4] describes the "state of practice" that applies in the United States for testing materials and defining the shear strength of embankment materials. That white paper was reviewed by the major embankment dam-related engineering associations of the US such as USACE, USBR, FERC, etc. USSD (2007) [4] focuses on "earthfill materials: soils with a grain size less than 20 mm". For fine-grained soils, a test series can easily be performed so that the design curve can be selected according to the "two-thirds" rule. Two-thirds of the shear strength results show a stronger characteristic than the selected design value and envelope. Within this context, it is important to note that the number of tests required is always a topic of discussion in practice. The authors believe that savings in testing and design work will result in costs that are many times higher during the construction phase. Therefore, the number of tests should be determined by the geotechnical and dam engineer and not by anybody else.

For free-draining rockfill, the steady-state static loading conditions should be critical. For these static conditions, USACE proposes the envelope according to the results of CD tests that consider the Mohr-Coulomb criterion, without mentioning the particular shear strength characteristics of rockfill. Practically, in the USA, cohesion is used for "impervious or semi-pervious fine-grained soils" only, so that a correction of the MC failure curve with regard to cohesion is not envisaged. This practice should generally not be applied to rockfill materials used for large dams unless the specific strength characteristics of rockfill are to be ignored.

As already mentioned above, a special topic is the application of partial safety factors as introduced by the Eurocode (EC) in Europe. The use of partial safety factors that decrease the strength "punishes" strong soils more than weak soils. This shows an effect on the actual slip surface and the safety value obtained. Although implemented and obligatory, the EC is still controversial in relation to the effect of this procedure on stability analysis and design. However, it is understood that the "new" regulations will not abandon "old", established design standards and philosophies. A comparison of both safety philosophies is given in [36] for different German dam case studies.

3.5 Dynamic shear strength and earthquake aspects

Different kinds of analysis can be performed for slope stability analyses under dynamic loads. Frequently, a simplified limit equilibrium slope analysis is performed for CFRD and CFSGD having to withstand a horizontal design acceleration < 0.2–0.3 g. This simplified method was developed in [37] for dams that withstood earthquakes of magnitude M = 7.0 or slightly stronger. For large dams that are prone to larger earthquake magnitudes (M > 7.5), a full dynamic analysis may supply more reliable results compared with the analytical approach of [37]. Generally, verification and validation of such sophisticated earthquake analyses are difficult due to the lack of reference data and the number of assumptions that have to be made when defining the input parameters.

Since rockfill materials are generally "free-draining", the occurrence of excess pore water pressure during dynamic loading can be excluded within the rockfill zones. Dry, well-compacted, strong rockfill shows a particularly high resistance to dynamic loading. Shallow planar surface failures, wedge or deep-seated circular failures are critical, but are rarely documented by case studies [38]. For sand-gravel fill dams, the pore water pressure conditions may change during dynamic loading and therefore they have to be evaluated specifically. Coarse sand-gravel fills usually show a "free-draining" characteristic thanks to a high permeability of about $k \geq 10^{-4}$ m/s.

For the pseudo-static slope stability analysis, a horizontal acceleration value $k_h = a_h/g$ is applied as an additional static load within a slope stability analysis (a_h = horizontal acceleration, g = gravitational acceleration imposed on whole slope and exerting a destabilizing force). In earthquake-induced shaking, it is clear that large slopes/dams have a lower probability that the direction of all the simultaneous dynamic movements are directed out of the slope or relieve the slip shear surface from its stabilizing gravity load. This is the reason why this factor is higher for small slopes than it is for large ones.

If the static global safety factor η (classic analysis) of the slope is > 1.0, even though a pseudo-static factor k_h is applied, large deformation is unlikely to occur. Nevertheless, defining the k_h acceleration value is always a problem. US practice assumes that the value should be $k_h = 0.2$ as a maximum for large embankment dams, as is also adopted in Turkish practice.

Day [39] summarizes the works of several authors starting with Terzaghi, trying to provide reliable values for k_h. As was said before, the problem is that k_h used for a simple sliding analysis can only be estimated reliably by a dynamic analysis. Once a dynamic model is established, a full dynamic analysis can be performed without the need to follow the analytical method of [37], [40].

Without suggesting a reduction in the engineering efforts required to define the suitable dynamic analysis and the appropriate rockfill material behaviour, the experience of large rockfill dams under earthquake loads is positive, particularly for CFRDs. "Rockfill dams generally perform well, particularly if well compacted" [41]. Single failures of large dams due to earthquakes were mainly the result of liquefaction or poor foundations and/or lack of construction quality. Rockfill is considered to be highly resistant to dynamic forces, although particle breakage may occur, which changes the gradation towards smaller particles and may also decrease the shear strength.

The degradation of basalt gravel material under cyclic triaxial loading is investigated for railways in [42]. The degradation did not govern the shear strength. Large-scale cyclic triaxial tests on coarse filter material to determine the damping and shear strain behaviour were performed in [22]. The permanent displacements were determined by performing a non-linear dynamic analysis combined with the Newmark method of applying a residual strength parameter derived from static triaxial tests. The dynamic behaviour of the Göschernalp Dam (H = 155 m) was investigated and resulted in horizontal deformations of max. 1.14 m, which is also < 1 % of the dam height.

However, the database for earthquake-induced deformations and stresses is considered to be still quite poor. FEM modelling of large rockfill dams under strong earthquakes is still more of an academic field with regard to the reliability of the results trying to provide data for permanent displacements or stresses.

The authors would like to point out that the input parameters for such analyses frequently include uncertainties that cannot be covered by sensitivity analyses. As mentioned previously, validation data is missing. Thus, a conservative approach for the selection of the loads and material behaviour might lead to theoretically unacceptable behaviour that contradicts the excellent performance of dams worldwide that have experienced strong earthquake motion.

In this context, [38] contains this statement: "Experience [of] the actual performance of these dams under earthquake loading conditions is still very scarce. There is no case history of a modern CFRD that was shaken by a very strong earthquake." Ref. [43] also reports that during two subsequent earthquakes in north-west Turkey in 1999, with magnitudes M = 7.4 and 7.2, some 53 fill dams were located in the area close to the epicentre, none of which suffered from critical failures or cracks. Two of the dams were 61 and 108.5 m high. These references originate before the Wenchuan earthquake occurred in 2008, when the 156 m Zipingpu CFRD was struck by an earthquake with a magnitude of 7.9 with an epicentre 240 km away. The reservoir water level was low and the maximum crest settlement was approx. 0.5 % of the dam height. Several more case studies of dams that were subjected to a strong earthquake with M > 7.0 close to the epicentre are given in [41]. These case studies showed a crest settlement < 1 % of the dam height and a generally reliable behaviour.

The strength of granular materials is frequently defined by the dynamic shear strength in the form of shear strain versus shear modulus and damping factor diagrams applying a linear equivalent analysis. A rough simplification assumes a linear elastic material behaviour, which is not able to contribute to the determination of permanent displacements [44]. Direct non-linear material models defining the shear strain in relation to the stress require considerable resources in terms of model preparation and materials testing. It should also not be forgotten that site-specific acceleration time histories are required which might show a considerable discrepancy with respect to actual future earthquake events. Frequently, time histories from well-recorded earthquakes are applied after adopting the duration and the pga (peak ground acceleration).

In practice, residual strength parameters derived from laboratory tests are used to reflect dynamic shear strength values for pseudo-static slope analyses. The cohesion c_R value is usually set to zero. If the residual friction angle φ_R of the large-grained material is not known, a reduction by a factor of 10–20 % may be applied to the peak strength friction angle. It can also be used to decrease the shear strength of the shear curves considering that dynamic loading will have an effect on the static and dynamic shear strengths, although knowing that the application of a reduction factor may still lead to a misleading result.

In conclusion, the permanent deformations originating from the alteration, degradation and loss of shear strength of the rockfill during earthquake are the critical parameters. According to [41], the settlement of large dams under strong earthquake motion will be limited to some 0.5–1.0 m. Generally, for large-magnitude earthquakes, too ($M_L > 8$), dams should not show a permanent displacement of more than 1 % of the dam height for embankment dams if they are not located near the earthquake centre (distance > 500 km, [45]). For modern rockfill dams, this value should be smaller.

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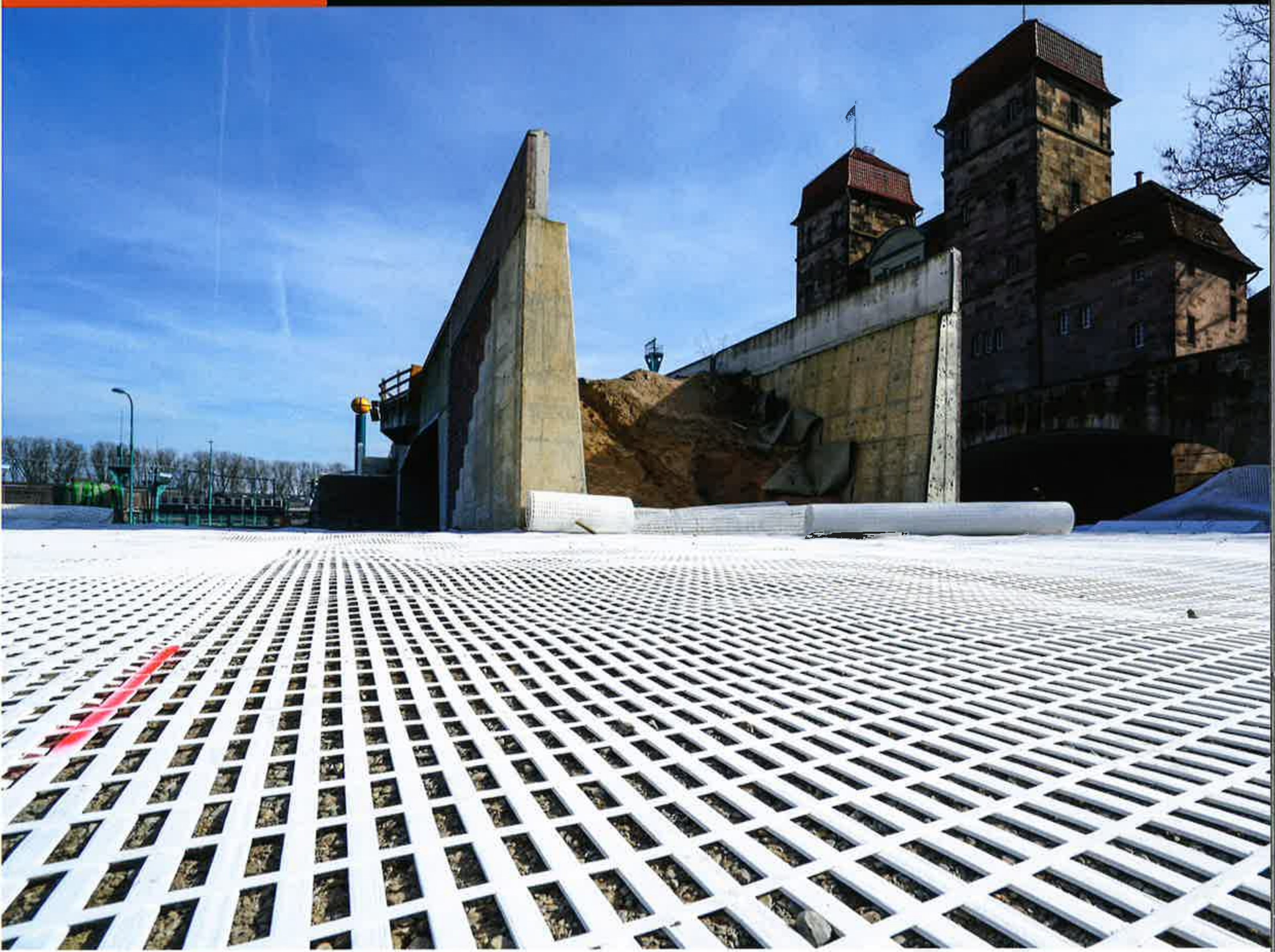

- Standsicherheit in der tiefen Gleitfuge
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- Zugversuch an Dichtungsbahnen
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