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Determination of rockfill shear parameters for dam stability analysis of an embankment dam

Détermination des paramètres de cisaillement de l'enrochement pour une analyse de stabilité d'un barrage en remblais

M. Smesnik
Pöyry Austria GmbH., Vienna, Austria

S. Krstic, M. Verdianz
Pöyry Austria GmbH., Vienna, Austria

ABSTRACT: Embankment dams constructed with rockfill material are commonly used as dam type worldwide. Improved knowledge of rockfill material properties, particularly shear resistance for stability assessment, is of an utmost importance, especially for higher dams with stepper slopes in regions with higher seismicity. Due to the heterogeneity and variety of gradation, particle shapes and mineral constituencies, the characterization of rockfill is a challenging task. It is mainly the particle size, that distinguishes the rockfill material from earthfill. Due to the often predominant large average particle size, the design of the test grading curve can be challenging. Various approaches for converting in-situ grading curves to test grading curves are commonly accepted, however, they always include uncertainties. In-situ tests (e.g. a large-scale in-situ direct shear test) are useful to determine material parameters (such as, stress-dependent friction angle in case of low stress levels, etc.) in the field and may provide useful additional knowledge for the interpretation of laboratory tests. This paper deals with the determination of material characteristics, that can be used for constitutive laws in numerical modelling, based on combined laboratory and in-situ testing, as well as on academic literature.

RÉSUMÉ: Les barrages en remblais construits à base de matériau d'enrochement sont un type courant dans le monde entier. Une meilleure connaissance des propriétés du matériau d'enrochement, en particulier sa résistance au cisaillement pour l'évaluation de la stabilité, est de la plus grande importance en particulier pour les barrages de plus grande hauteur et à pente plus raide dans des régions plus sismiques. Du fait de l'hétérogénéité et de la variété de la répartition en taille, des formes des agrégats et des constituants minéraux, la caractérisation de l'enrochement est difficile. C'est en particulier la dimension des agrégats qui distingue le matériau d'enrochement d'un remplissage de terre. Du fait de la prédominance courante de grosses dimensions moyennes d'agrégats, la conception de la courbe de répartition pour essai peut être difficile. Différentes approches de conversion des courbes de répartition sur site en courbes de répartition d'essai sont couramment acceptées mais incluent toujours des incertitudes. Les essais sur site (c'est-à-dire essai direct de cisaillement sur site à grande échelle) sont utiles pour connaître les paramètres du matériau (angle de frottement sous contrainte dans le cas de faible niveau de contrainte, etc.) sur le terrain et peuvent fournir des connaissances supplémentaires utiles pour l'interprétation des essais en laboratoire. Ce document traite de la détermination des caractéristiques de matériau, pouvant être utilisées pour définir des lois de constitution en modélisation numérique, à partir d'une combinaison d'essais en laboratoire et sur site, ainsi que de la littérature disponible.

Keywords: shear strength; large-scale in-situ direct shear test; large-scale triaxial test; dam stability

1 INTRODUCTION

For the construction of large civil structures, like embankment dams, coarse grained materials (e.g., rockfill) are of importance. Traditionally, material parameters of rockfill materials have rather seldom been tested, because of the size of the required apparatus and the corresponding costs. Improved knowledge of rockfill material properties, particularly, shear resistance for stability assessment, is of utmost importance, especially for higher dams with stepper slopes in regions with higher seismicity. Pioneering experimental work in large-scale testing (e.g., large-scale triaxial tests) of a wide range of different materials was developed in the 1960s by Marshall as well as by Marachi and colleagues. Nowadays, and as it has also been mentioned by Indraratna, Wijewardena and Balasubramaniam (1993), geotechnical laboratories commonly do not have large scale tests in their scope. In order to gain additional information about material behavior, it is beneficial to use existing data from comparable projects or literature. Often it is not possible to use the existing data, because either the information is not published (i.e., no open access) or the data simply do not exist. When using the material parameters estimated from literature, it is important to understand and be aware of the test procedure, as well as of the evaluation process of test results. This paper deals with the determination of shear parameters of basalt rockfill material for dam stability calculations, based on laboratory and in-situ material testing as well as on literature. For the evaluation of the parameters a large-scale in-situ direct shear test (DST) has been performed during the dam fill works of a 130 m high ACED. The test results and the gained knowledge about the material behavior have been compared with data from prior carried out large-scale triaxial tests and literature.

The first and major step in a dam design is the definition of material characteristics, that serve as an input for the analytical and numerical calculations with various software tools. As commonly known, the main difficulty is that the material characteristics and behavior are stress depended. The construction of high dams and related various stresses demand knowledge of the material behavior in low as well as in high stress levels. For the definition of the shear resistance in a basic dam design phase it is common to use the data assembled and provided in literature (triaxial test data for various rockfill types) by Leps (1970). Figure 13 shows the interpreted effective friction angles as function of the effective normal stress. According to literature the shear strength of rockfill material can be estimated for basic design stages by using grading curves in combination with UCS values (section 2.3). However, it is not sufficient to use only literature data, especially in more detailed design stages, where material investigations are mandatory. A more appropriate way is to perform large-scale tests in the field and in laboratory. Moreover, large-scale triaxial tests in the laboratory should be every time the first choice, due to their ability to test the material in various loading conditions. Due to different stress conditions along a circular slip plane, as shown in Figure 1, the combination of large-scale in-situ DST and large-scale triaxial tests is effective for the determination of material behavior.

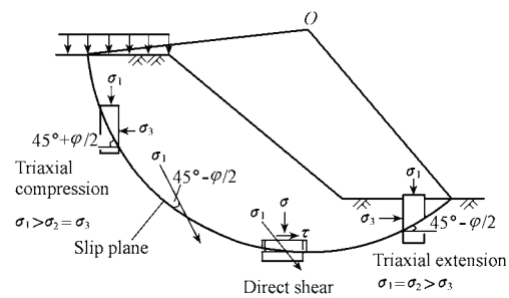

Figure 1. Average stress conditions along circular slip plane (Liu, 2009)

1.1 Large-scale in-situ direct shear tests

The principle of most performed large-scale in-situ direct shear tests are comparable to direct shear tests in the laboratory. Generally, the aim is to determine the Mohr-Coulomb strength envelope of a material. The challenge of performing in-situ DST on site is that the confining stress is mostly not known and difficult to determine.

1.2 Large-scale triaxial tests

In comparison to the large-scale in-situ DST, large-scale triaxial tests have to be performed in laboratory. For the evaluation of the material characteristics, a minimum number of tests is required, that is linked with a corresponding amount of material and can be, therefore, time and cost intensive. Taking in consideration, that these tests are not in the main focus of geotechnical laboratories, it is not easy to find test cells with a sufficient diameter for most rockfill materials. It is mostly required to design a test grading curve derived from the rockfill grading curve for testing. According to Indraratna et al. (1993), the limitation of the cell diameter and the related limitation of the maximum particle size in the test resulted in an incomplete understanding of the true behavior of rockfill (e.g., dilation and particle crushing phenomena), particularly, in relation to shear strength mobilization. The effect of size ratio on the behavior of rockfill specimens in triaxial testing has been discussed in depth by several investigators. The triaxial behavior of rockfill is influenced by several factors, including the confining stress, saturation, porosity, and particle size distribution. The uncertainties resulting from converting rockfill grading curve to test grading curves are also present in in-situ DST. Commonly larger in-situ shearing frame dimensions in comparison to triaxial cells reduce converting effects. For a better understanding of rockfill, used in dams, it is common to run CD (consolidated drained triaxial tests) large-scale triaxial tests.

2 EVALUATION OF MATERIAL CHARACTERISTICS OF BASALTIC ROCKFILL MATERIAL (A CASE STUDY)

In order to gain sufficient data for the evaluation of the dam stability of the 130 m high ACRD, trial embankments and large-scale in-situ DST have been carried out. As the next step, the test results have been compared with prior performed large-scale triaxial tests on comparable rockfill material (UCS, grading curve, etc.). Kutzner (1996), as well as Asadzadeh and Soroush (2009) provide techniques for the design of test grading curves, related to the original grading curve. Generally, realistic test results can be obtained only if the gradations at least show:

1. Similar grain size distribution curves
2. Comparable field and test compaction
3. Similar angularity of particles (conforming shapes).

2.1 Large-scale in-situ direct shear test

The test has been carried out on five different stress levels (65, 115, 160, 210 and 260 kPa) to determine the shear resistance of rockfill material. In total, five tests (each with several various vertical loads) have been carried out. The vertical load has been applied with reinforced concrete blocks (each block with a weight of 8.4 tons). The vertical stress levels in the shear plane are a superposition of the weight of the concrete blocks and dead weight of the rockfill material inside of the shear frame. The steel shearing frame (130 cm x 130 cm x 65 cm) has been pulled with a horizontal force applied through a hydraulic jack. The shear force has been recorded with a load cell while a constant vertical (normal) force has been applied. During all tests the horizontal force, as well as the horizontal and vertical displacements, have been recorded. The tests have been performed on the actual placed dam fill. The rockfill test grading

curve with a d_{max} = 250 mm for the filling of the shearing frame and for the subgrade (Figure 3) has been an adoption of the in-situ grading curve to the used testing equipment following common practice (Asadzadeh & Soroush, 2009; Indraratna et al., 1993; Kutzner, 1996). The subgrade, as well as the rockfill material in the shearing frame have been compacted with a 16 tons roller by 6 passes and slushing water amount of 100 l/m³, according to the outcomes of performed trial embankments.

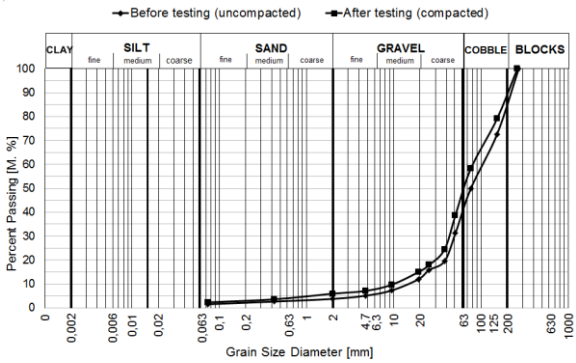

Figure 2. Test grading curves for rockfill material (large-scale in-situ DST)

Figure 2 shows the test grading curve of the compacted rockfill material after the test in comparison to the test grading curve of the uncompacted rockfill material. Only a slight shift of the grading curve can be seen, that might be a hint for indication of a sufficient compaction of the material (no/ less particle breakage, etc.).

Figure 3. Subgrade for in-situ DST (dashed box)

For the reduction of the friction between the steel frame and the subgrade, that can possibly influence the test results, a geotextile has been applied (Figure 4).

Figure 4. In-situ large-scale DST (1 concrete block ≡ 8.4 tons)

2.1.1 Test results

Figure 5 shows the variation of shear strength with effective normal stress for all performed tests. It has been investigated, if a test on the same location (multilevel incremental test) has an influence on the ratio between the shear strength and effective normal stress. The trend lines, depicted in Figure 5, indicate that a retest slightly affects the test results. Therefore, all test results (multilevel incremental tests) have been used for further evaluations. The non-linear failure criteria, calculated according to Indraratna et al. (1993) with the parameters σ_c= 200 MPa, a = 0.30 and b = 0.82, are additionally shown in Figure 5.

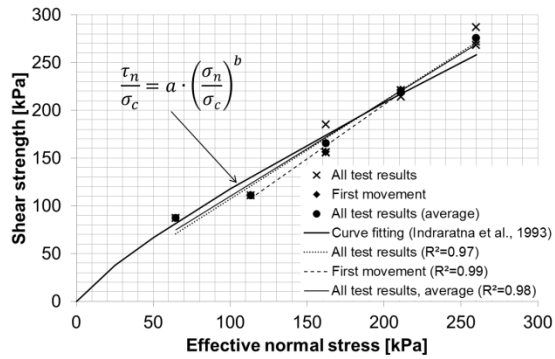

Figure 5. Variation of shear strength with normal stress

Figure 6 depicts the calculated friction angles, that have been determined with the following formula:

$$\tau = \sigma_n' \cdot tan\,\varphi' + c \qquad (1)$$

Where τ (kPa) is the shear strength, σ_n' (kPa) is the effective normal stress, φ' (°) is the friction angle and c the cohesion (kPa). The friction angles for the average, as well as for the results of test 5, depicted in Figure 6, have been calculated following equation (1) with an assumption of zero cohesion. On the contrary, the friction angles for the lowest line in Figure 6 have been determined as tangents on the non-linear failure curve shown in Figure 5. If the drawn tangents for each stress level do not intersect the origin of τ – σ diagram, cohesion must be assumed for the determination of the shear strength besides a friction angle. Figure 6 shows friction angles according to both determination methods.

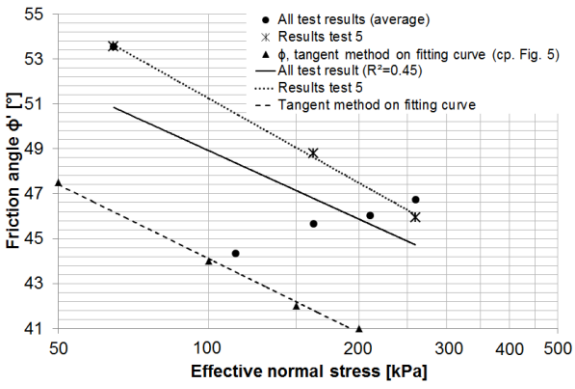

Figure 6. Friction angle vs. effective normal stress

2.2 Large-scale triaxial tests

Two series of three tests each have been carried out, one series on unsaturated specimens and the other on saturated specimens. The effective confining pressure (σ_3') has been set to 0.1/0.3/0.8 MPa for both series. The test specimens have had a diameter D = 800 mm and a height H = 800 mm. The material has been placed in layers of about 150 mm in the sample former. Each layer has been compacted with a vibrator until no further settlement of the specimens surface could be observed. After the first half of the specimens have been inserted, it has been compacted additionally by a static load using the hydraulic jack of the triaxial apparatus. Then the upper half of the specimen has been inserted and compacted in the same manner. During the shear phase the lateral pressure has been kept constant and the axial load has been applied with a constant deformation rate of 0.05 %/min. When the axial deformation had reached more than 10 %, the deformation rate has been changed to 0.005 %/min in order to obtain information about the influence of different deformation velocities on the mechanical behavior of the rockfill material. (Karlsruhe Institute of Technology, 2013)

2.2.1 Test results

In order to identify possible particle breakage, the grading curve before and after the performed tests has been compared. Figure 7 and Figure 8 display the difference between the grading curves before and after testing (0.1/0.3/0.8 MPa), due to particle breaking during shearing for the unsaturated and saturated specimens. The test grading curves of the unsaturated test specimens (Figure 7) have been shifted towards fines with increasing confining pressure. For all grading curves of the saturated specimens (Figure 8) only a slight shifting towards fines has been detected.

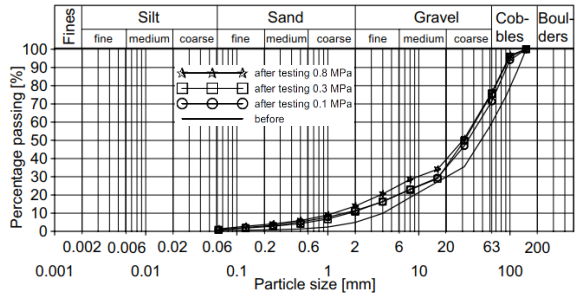

Figure 7. Grading curves before and after triaxial testing, unsaturated specimens (Karlsruhe Institute of Technology, 2013)

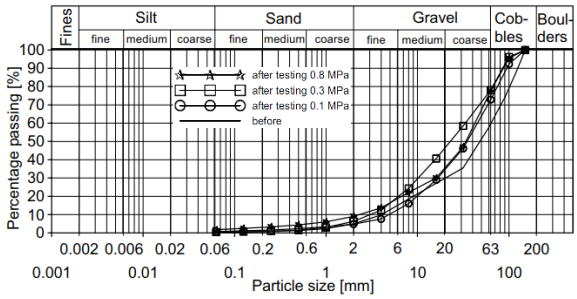

Figure 8. Grading curves before and after triaxial testing, saturated specimens (Karlsruhe Institute of Technology, 2013)

The Mohr-Coulomb failure envelopes have been determined for both test series. Resulting by limited tests, the friction angles have been determined by drawing a straight line from the origin to the tangent point of each Mohr-circle (Table 1, evaluation approach 1). This approach can lead to an overestimation of the friction angle.

Table 1. Friction angle φ' (triaxial testing), evaluation approach 1 (Karlsruhe Institute of Technology, 2013)

Confining pressure [MPa]	**Initial dry density [g/cm³]**	**φ' [°]**	**Principal stress ratio (σ'_1/σ'_3)**
0.1_unsat	1.98	56	11
0.3_unsat	1.97	51	8.2
0.8_unsat	1.99	46	6.5
0.1_sat	1.99	48	7
0.3_sat	1.99	51	8
0.8_sat	1.97	41	4.8

In addition to approach 1 Figure 9 presents the estimated friction angles, while using a straight line between the tangent points of two Mohr-circles (evaluation approach 2). This approach results in friction angels between 43° (σ_3' = 0.8 and 0.3 MPa) and 48° (σ_3' = 0.3 and 0.1 MPa).

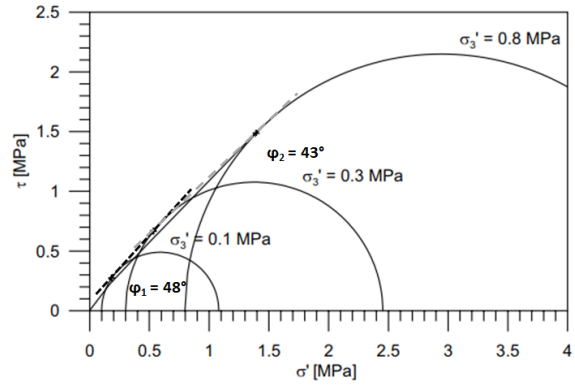

Figure 9. Large-scale triaxial test (unsaturated), evaluation approach 2 (Karlsruhe Institute of Technology, 2013), modified by Smesnik

The third evaluation approach is presented in Figure 10 by using the equation of Indraratna et al. (1993) in combination with the parameters σ_c= 200 MPa, a = 0.39 and b = 0.79. The dotted line, generated by curve fitting, shows the τ – σ relation. Tangents on the curve, that reflect a friction angle for a stress level are used for comparison with Figure 13.

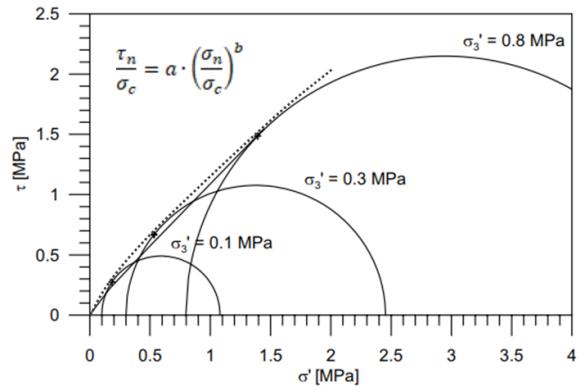

Figure 10. Large-scale triaxial test (unsaturated), evaluation approach 3 (Karlsruhe Institute of Technology, 2013), modified by Krstic

Having used different determination methods similar effects on calculated friction angles (shear strength) have been observed as in section 2.1.1.

Marsal and Marachi (1969) as well Charles and Watts (1980) found that the principal stress ratio for drained tests has been considerably increased at low confining stresses (cell diameter 1.0 m and 0.23 m) (Indraratna et al., 1993). Present test results have been added in Figure 11.

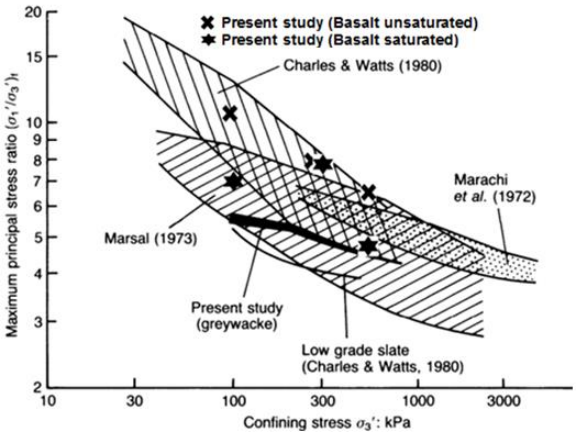

Figure 11. Variation of effective principal stress ratio at failure with effective confining stress for various rockfills (Indraratna et al., 1993), modified by Smesnik

Figure 12 shows the relation between shear strength and effective normal stress (low stress range) for various rockfill types. For comparison purposes, the curve generated with evaluation approach 3 from Figure 10, has also been presented in Figure 12.

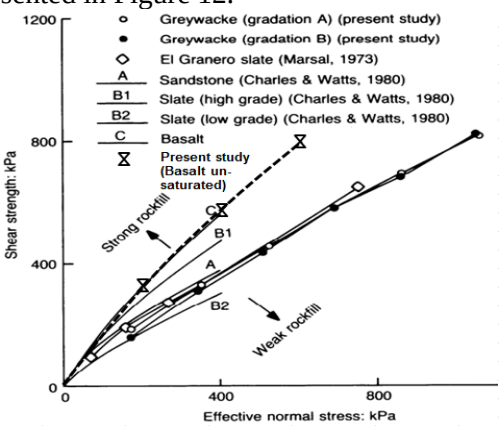

Figure 12. Variation of shear strength with normal stress for various rockfill types from (Indraratna et al., 1993), modified by Krstic

2.3 Determination of stress dependent friction angle based on literature

The reduction of the friction angel φ' with increasing confining pressure is commonly known and can be, probably, associated with the significant increase of crushing of angular particles by increasing confining pressure. It is also widely accepted that the effect of particle size on φ' remains as a more complex phenomenon, than the marked influence of confining stress (Indraratna et al., 1993). Formula (2) developed by (Barton & Kjaernsli, 1981) provides an empirical approach for the estimation of the friction angle.

$$\varphi' = R \cdot log\left(\frac{S}{\sigma_n'}\right) + \varphi_r' \qquad (2)$$

Where R (-) is the equivalent roughness of rockfill, related to initial porosity of rockfill and genesis, angularity and surface roughness of particles; S (-) is the equivalent strength of rockfill particles expressed as a function of the uniaxial compressive strength of the parent rock; σ_n' (kPa) is the effective normal stress and φ_r' (°) is the residual friction angle. Barton developed a scheme for estimating the R-value for rockfills as well as for the estimation of S, as a function of σ_c', weather for rounded gravels (e.g., alluvial fill materials) or for rough quarried rock.

3 CONCLUSION

This paper focuses on a comprehensive discussion about the selection of shear strength in different design stages. For the analysis of the slope stability in an early design stage, such as feasibility or basic design, the estimation of the rockfill shear strength (φ') of an embankment dam, introducing an equivalent roughness approach (Section 2.3), is adequate at low to medium normal (vertical) stress levels (dashed line Figure 13). However, the more accurate definition of UCS as well as of φ_r' has a crucial impact on the estimated stress dependent rockfill shear strength and should be therefore done with care.

For further and more detailed design stages it is deemed to be mandatory to perform laboratory (e.g., large-scale triaxial tests) and in-situ (e.g., large-scale in-situ direct shear tests) material tests. Considering different sources of material testing the final selection and determination of material characteristics of rockfill materials is still a challenging task. During the evaluation process some assumptions, which can have a crucial impact on the determined material characteristics, are done (section 2). It can be stated that a combination of laboratory and field testing is seen by the authors as the most accurate way to determine the material characteristics of rockfill. In order to shed the light on gaining detailed knowledge of the material behavior, large-scale triaxial tests in combination with large-scale in-situ DST on site are recommended. Conclusively, based on the outcome of the case study, the performed large-scale in-situ DST has provided reliable test results and valuable information, allowing elaborating the dam design on a more accurate design basis.

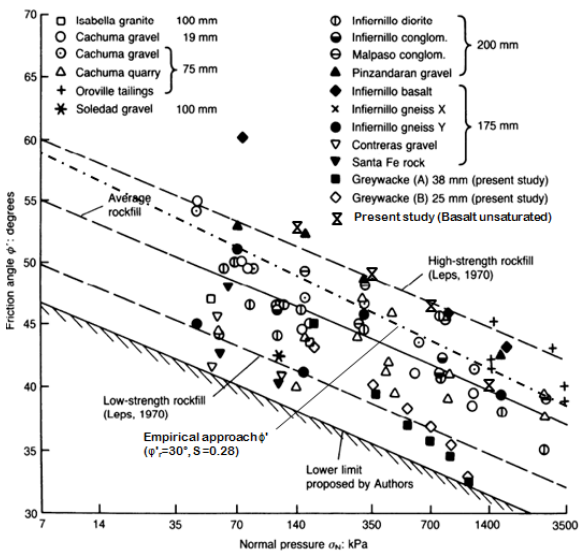

Figure 13. Assembly of peak shear strength data for rockfills (Indraratna et al., 1993), modified by Smesnik

4 ACKNOWLEDGEMENT

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