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IMPLICATIONS OF NON-LINEAR STRENGTH CRITERIA IN THE STABILITY ASSESSMENT OF ROCKFILL DAMS

IMPLICATIONS DES CRITERES DE LA RESISTANCE NON-LINEAIRE DANS L'EVALUATION DE LA STABILITE DES BARRAGES REMBLAYES

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SYNOPSIS: A series of large scale triaxial tests were conducted on greywacke rockfill at low confining stresses, and the implications on the slope stability of rockfill dams are elucidated. Revised failure criteria for rockfill are proposed in non-dimensional form, where lower and upper bounds of strength envelopes are established for a wide array of granular materials, including greywacke. The influence of the confining stress on the shear strength of rockfill is studied in depth. The effect of the pronounced non-linearity of the failure envelope at low normal stress levels on the stability of a 95m high rockfill dam is discussed. Consideration of two parallel rockfill gradations for specimens compacted to similar porosities enabled the effect of particle size on the rockfill behaviour to be evaluated. The findings confirm that the particle size effect is of secondary importance, in relation to the effect of confining stress on the shear strength.

INTRODUCTION

Greywacke rockfill has been used in the construction of several dams in Southeast Asia, including the recently completed Chiew Larn Dam in Thailand. Considering the limited test results of greywacke rockfill in the past, the proper understanding of the shear strength properties of these materials shall enable better design of large rockfill dams. Greywacke found in Southern Thailand is a sedimentary rock containing quartz as the dominant mineral. Generally, this greywacke rock is found without internal partings, and has a mean uniaxial compressive strength of 135 MPa. As significant particle size differences occur between the field and small triaxial specimens, deviations from actual behaviour are inevitable in conventional testing. Therefore, a large scale triaxial equipment (0.3 m diameter specimens) was utilized in this study to minimize size dependent errors.

The objective of the current study was to quantify the strength and deformation behaviour of large scale greywacke rockfill specimens at relatively low confining pressures in relation to the stability of rockfill dams. In many of the early laboratory test programs, strong emphasis was placed on testing at confining pressures as high as 4500 kPa (Marachi et al., 1972; Marsal, 1967). As the slope failure of a rockfill dam is generally associated with relatively low normal stresses, the shear strength of rockfill must be evaluated under nominal confinement. The maximum normal stresses on a critical failure surface of large dams approaching 100m in height are unlikely to exceed 1 MPa (Lee, 1986). Testing at low confining pressures is of particular importance in evaluating the stability of rockfill slopes by the slip circle or sliding wedge procedures, as the potential slip surfaces generally occur at shallow depths with low normal stresses. Leps (1988) has also discussed the importance of rockfill testing at much lower confining pressures, in order to obtain data of greater practical value.

TESTING METHODOLOGY AND STRESS-STRAIN CHARACTERISTICS

Isotropically consolidated, drained triaxial compression tests were performed varying the effective confining pressure from 100 to 600 kPa at intervals of 100kPa. This pressure range is adequate to simulate the confining pressures existing within the 95m high Chiew Larn Dam.

Considering the similitude requirements between the field and laboratory samples, the gradation curves of the laboratory rockfill were made parallel to that of the prototype (Chiew Larn dam). In addition, the porosity of the compacted laboratory specimens was made to match the porosity of the actual rockfill in the field (about 30%). The initial water content of the rockfill was about 5%, and after compaction the mean dry density of the specimens was determined to be around 18.5 kN/m^3. During the construction of the Chiew Larn dam, each compacted rockfill layer varied in thickness from about 0.6-1.0m in the field, producing a dry density greater than 18.0 kN/m^3 achieved by heavy vibratory rollers (10 tonnes), with a compacted porosity similar to that obtained in the laboratory. Typical gradations used in the laboratory are illustrated in Fig. 1, in comparison with that of Chiew Larn dam. The effect of size ratio (maximum particle dimension/ triaxial cell diameter) on the behaviour of rockfill specimens in triaxial testing has been discussed in depth by several previous investigators (Nitchiporovitch, 1969; Marachi, 1969). In this study, size ratios of 8-12 were adopted for the laboratory gradations, which are higher than the rockfill tests conducted previously.

Shear Strength and Failure Criteria

The Mohr-Coulomb strength envelope for a typical greywacke sample (gradation A) is shown in Fig. 2. The shape of the strength envelope for gradation B is similar. At low confining stresses marked non-linearity is observed, and as expected, the zero cohesion reflects the granular nature of rockfill. A non-linear failure criterion for typical rockfill was proposed by de Mello (1977) as;

$$\tau_f = a\sigma_n^b \tag{1}$$

In the above formulation, the constants a and b are considered as characteristic parameters obtained by curve fitting. Nevertheless, the physical significance of these constants is ambiguous, because the value of a not only depends on the system of units used, but also its dimensions vary according to the value of b. In this respect, it seems that the introduction of a non-dimensional failure criterion is probably more realistic, as a wide array of different materials can then be directly compared within the framework of similitude (Indraratna, 1990).

Rockfill can be regarded as intensely fractured rock, and further breakage of individual fragments during shearing is a function of particle angularity and the point load index related to the uniaxial compressive strength σ_c of the parent

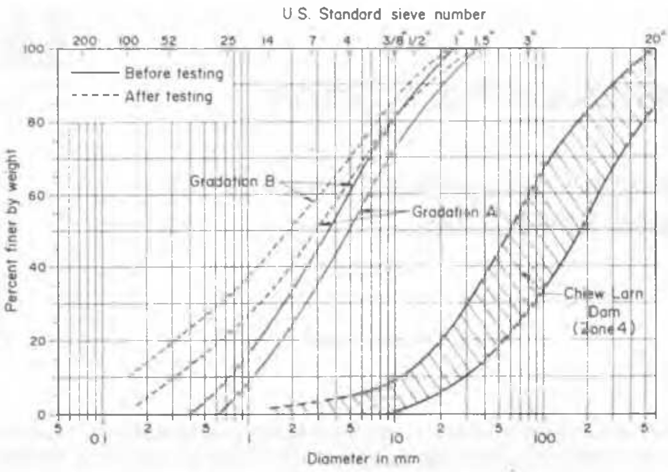

Fig. 1 Particle Size Distribution Curves of Greywacke Rockfill

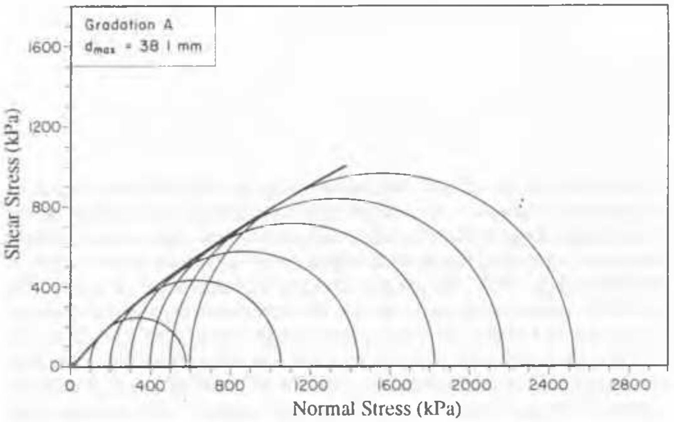

Fig. 2 Mohr-Coulomb Failure Envelopes of a Typical Greywacke Rockfill

rock. In this study, the authors have incorporated the uniaxial compressive strength in defining the failure envelope of rockfill. In reality, the value of σ_c can be estimated by conducting basic index tests even on an irregular lump. It is proposed that the failure envelope in a non-dimensional form can be expressed by the following equation, as a modification of de Mello (1977) criterion:

$$\frac{\tau_f}{\sigma_c} = a\left(\frac{\sigma_n}{\sigma_c}\right)^b \tag{2}$$

The above constants a and b are dimensionless, hence independent of the system of units used for the stresses. For linear Mohr-Coulomb materials, the constant b approaches unity, and the magnitude of a is given by the ratio of shear strength to the normal stress. In effect, the parameter a encompasses the equivalent friction angle and can be regarded as an intrinsic shear strength index. The magnitude of b dictates the non-linearity of the failure envelope particularly at low confining stresses, and thereby representing the deformation response of the rockfill, including to some extent the effects of dilation and particle sizes.

Figure 3 illustrates the normalized shear strength versus normal stress relationships plotted on log scales. Together with the current test results, an extensive assimilation of experimental data is presented here for a wide range of effective normal stresses from 100 kPa up to 8 MPa. Irrespective of the crushing strength of rock, the particle sizes, angularity, initial porosity and the initial water content, the above experimental data fall within a narrow band

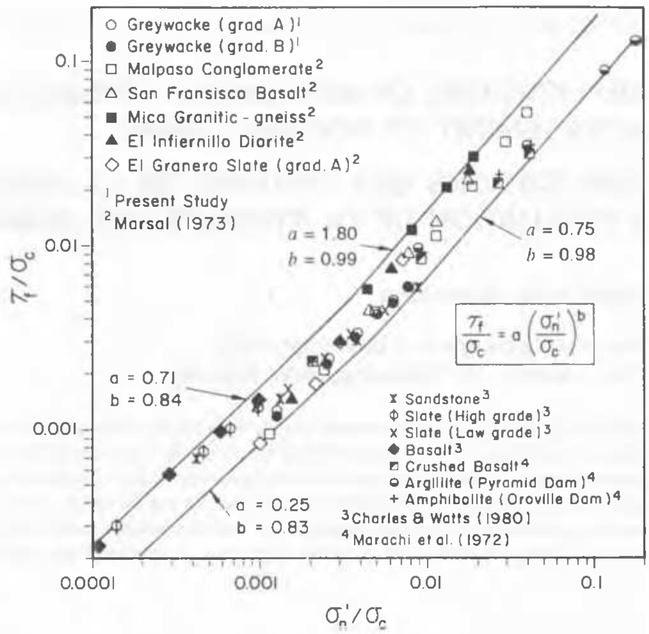

Fig. 3 Normalized Shear Strength Versus Normal Stress Response for Various Rockfill

defined by the following boundaries, applicable to the normal stress ranges given below.

For σ_n' : 100 kPa to 1 MPa (or σ'_n/σ_c: 0.0001 to 0.005)

Lower bound : $a = 0.25$, $b = 0.83$
Upper bound : $a = 0.71$, $b = 0.84$

For σ_n' : 1 MPa to 7 MPa (or σ'_n/σ_c: 0.005 to 0.10)

Lower bound : $a = 0.75$, $b = 0.98$
Upper bound : $a = 1.80$, $b = 0.99$

Alternatively, the failure of rockfill specimens can be represented by the major and minor principal stresses at failure (σ_{1f}) and (σ_{3f}). Incorporating the uniaxial compressive strength (σ_c), the following normalized expression is proposed to represent failure:

$$\frac{\sigma_{1f}'}{\sigma_c} = \alpha\left(\frac{\sigma_{3f}'}{\sigma_c}\right)^\beta \tag{3}$$

Irrespective of the differences in a wide array of granulated materials, their peak (failure) response can be represented by a narrow band width defined by the following upper and lower bounds of α and β.

For σ_n' : 100 kPa to 1 MPa (or σ'_n/σ_c: 0.0001 to 0.005)

Lower bound : $\alpha = 0.40$, $\beta = 0.62$
Upper bound : $\alpha = 0.78$, $\beta = 0.65$

For σ_n' : 1 MPa to 7 MPa (or σ'_n/σ_c: 0.005 to 0.10)

Lower bound : $\alpha = 2.71$, $\beta = 0.96$
Upper bound : $\alpha = 3.58$, $\beta = 0.90$

As a preliminary design tool in stability analysis, if the uniaxial compressive strength of the parent rock is known or determined by basic rock testing, then the failure envelope of the corresponding rockfill can be estimated from one of the proposed failure criteria. At high normal stresses ($\sigma_3' > 1$ MPa), the values of b and β approach unity, as the failure envelope becomes linear. Under these circumstances, the magnitude of a and α represent $\tan\phi'$ and the effective principal stress ratio, respectively, for a given confining stress. However, as the overall upper and lower bounds deduced for the array of rockfills (Fig. 3) can be significantly different, the corresponding coefficients should only be considered during the preliminary design phase. In particular, the expression for the high stress ranges can be regarded as equivalent to a friction angle with a lower bound of about 35° and an upper bound of about 60°, with little error.

Influence of the Confining Pressure on Friction Angle

Well documented studies have indicated that the friction angle of granular materials decreases with increasing cell pressure in drained triaxial tests (Bishop, 1966). Barton and Kjarnsli (1981) proposed an algebraic model to predict the variation of friction angle with confining pressure. The variation of the drained friction angle (ϕ') of greywacke is plotted against the effective confining pressure (σ_3') in Fig. 4. As the confining pressure is increased from 100 to 600 kPa, the drained friction angle for both rockfill gradations decreases from 45° to 32° and 43° to 33°, respectively. Although the friction angle of gradation A drops at a faster rate than that of gradation B, as the confining pressure is increased to 600 kPa, ϕ' for both gradations attains the same value irrespective of the particle sizes. In this region, the predictions based on the Barton and Kjarnsli (1981) approach deviate significantly, perhaps due to the effect of particle crushing. The effect of particle sizes on ϕ' remains to be a more complex phenomenon in contrast to the more definite influence of confining stress.

APPLICATIONS IN PRACTICE

The friction angle corresponding to the failure envelope is the most important parameter required in design for the slope stability analysis of rockfill dams. It takes its maximum value at the least normal stress, whereas at extremely high stress levels it may even approach values close to 30°. The non-linear strength envelope quantifies this phenomenon adequately. In relation to the angle of friction of common rockfill materials, current rockfill dams are constructed at much flatter slopes, in spite of the capabilities of modern vibratory rollers in compacting rockfill to achieve field porosities lower than those obtained in the laboratory. One reason for this of course is concerned with the existence of the central clay core, which influences design leading to reduced upstream slope angles. If the appropriate friction angles are not carefully selected according to

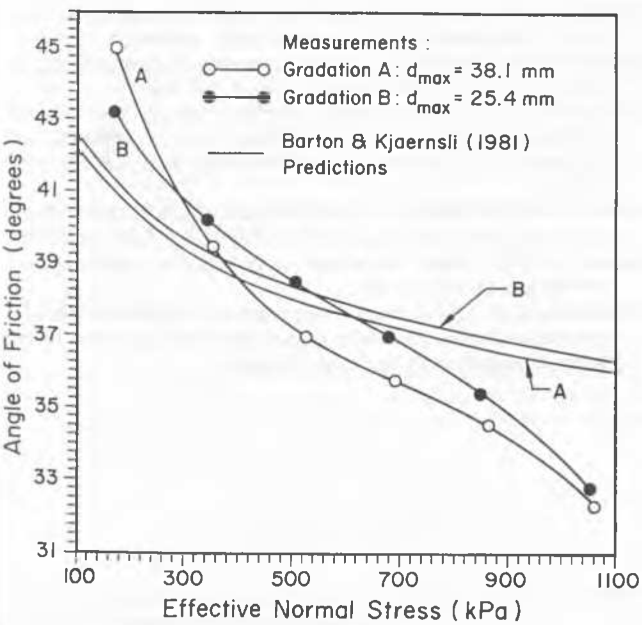

Fig. 4 Variation of Drained Angle of Friction of Greywacke Rockfill with Effective Confining Stress

the effective normal stress (σ_n'), then the prediction of critical slip surfaces or sliding wedges becomes questionable.

For large rockfill dams, design values of ϕ' ranging from 35° to 45° have been suggested for various rockfill by many investigators (Brown, 1988; Leps, 1988; Penman & Charles, 1976). The following section describes the effect of descending ϕ' with depth on the factor of safety evaluation of the Chiew Larn Dam in Thailand, based on a static, equilibrium analysis. A detailed dynamic stability analysis (including seismic inertia forces) of this rockfill dam based on the finite element method is discussed elsewhere by Wijewardena (1991).

The Chiew Larn dam is a symmetrical dam with a central clay core protected by a sandy filter and an upstream coffer dam. Angular greywacke rockfill (same material tested in the laboratory) was used in the construction of this 95m high rockfill dam. The transition zones and shells were raised with good quality greywacke, whereas only the toe berm comprised of slightly weathered rockfill. Vibrating smooth drum rollers were used to compact the upstream and downstream shells in 1.0 m layers, which resulted in compacted porosities estimated in the vicinity of 30%. Although the specifications stipulated not more than 5% passing through No. 4 (5mm) sieve, up to 30% of rockfill particles less than 25mm was found to be satisfactory in the actual construction. This seems to be in accordance with Leps (1988) who proposed a minimum specification of 40% passing 1 inch (25 mm) sieve, which does not require processing of rockfill to remove fines.

As discussed earlier, the angle of friction of greywacke rockfill decreases from about 45° to about 33°, as the normal stress is increased from 200 to 1000 kPa. This implies that for a given slip surface the mobilized friction angle must be varied according to the normal stress acting along the failure plane. In order to demonstrate the influence of the varying friction angle, a stability analysis of the Chiew Larn dam was conducted. The analysis was carried out for various non-circular failure surfaces for both upstream and downstream slopes (Fig. 5), using a computer code based on Spencer (1973) method. For each slice, the normal stress at the mid-point of the base was determined, and the corresponding friction angle was assumed from the relationship given in Fig. 4 for the saturated rockfill of gradation A (larger particles). For the central core, a cohesion intercept of 15 kPa and a constant friction angle of 31° were used, respectively. The static safety factors computed for each slip surface shown in Fig. 5 are summarized in Table 1. The corresponding safety factors computed by conventional analysis for a constant, average friction angle of 40° are also tabulated for comparison.

Table 1 Safety Factors of Slip Surfaces

Slip Surface	Factor of Safety for Varying ϕ'	Factor of Safety for $\phi' = 40°$
DS1	1.86	1.57
DS2	1.80	1.74
DS3	1.77	1.65
DS4	1.75	1.59
DS5	1.64	1.74
US1	2.09	1.75
US2	2.06	1.85
US3	2.01	1.76
US4	1.91	1.88
US5	1.89	1.92

Note: DS = downstream; US = upstream

These safety factors ensure the stability of the Chiew Larn dam, on the basis of the numerous slip surfaces considered (F > 1.5). As expected, the safety factors against sliding for shallow surfaces (US1, US3, DS1) are dramatically increased, if the correspondingly higher friction angles (i.e. $\phi' > 40°$) are used. Therefore, an adequate factor of safety could still have been maintained even if the dam had been raised with steeper slopes. For large rockfill dams, Lee et al. (1983)

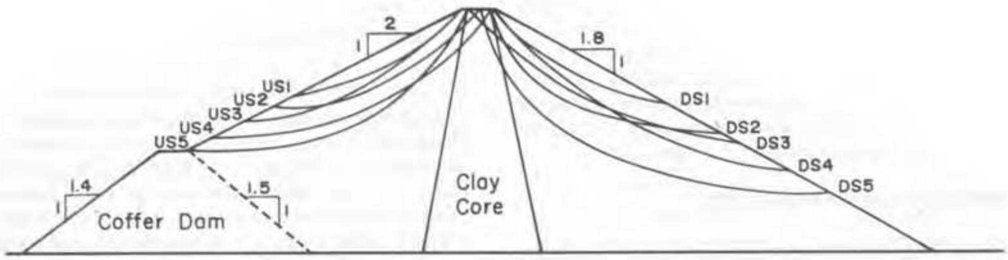

Fig. 5 Non-circular Failure Surfaces Considered in the Stability Analysis of Chiew Larn Dam

suggested that the factor of safety (F) may be estimated by assuming translational slides in an infinite slope, in which case F should decrease with depth, given the decline of ϕ' with σ_n'. For the Chiew Larn dam, the trend of safety factors computed using varying ϕ' with depth is in accordance. If a mean friction angle such as 40° is assumed throughout the depth, then the shallow slip surfaces always produce a very conservative factor of safety. On the contrary, deep seated failure surfaces (DS5, US5) over-estimate the value of F, in comparison with the varying ϕ' analysis.

The above calculations indicate that the upstream slope is more stable than the downstream slope. One reason is that the downstream slope is slightly steeper (1:1.8) than the upstream slope (1:2). The lower inclination of the upstream slope is justified, in view of the necessity for rapid drawdown. In reality, the downstream slope is often more stable than the upstream slope. Although the actual role of the initial water content on the friction angle of greywacke rockfill was not investigated in this study, a relatively dry downstream rockfill can be expected to have a greater angle of shearing resistance than the wetter upstream rockfill.

CONCLUSIONS

Greywacke rockfill tested in this study has a similar engineering behaviour to that of many other typical rockfill. Although the particle size and angularity of the rockfill influence the stress-strain behaviour including dilation, their role seems to be of secondary importance in contrast to the effect of confining stress on the shear strength. The angle of shearing resistance and the associated failure envelope of rockfill are directly related to the magnitude of confining stress. At low confining stresses (< 500 kPa), the non-linearity of the failure envelope is pronounced. At much higher confining stress levels (> 1.5 MPa), the assumption of linear Mohr-Coulomb criterion is quite acceptable. Considering the current test results of greywacke rockfill together with previous experimental findings, two modified failure criteria for rockfill have been proposed in non-dimensional form incorporating the unconfined compressive strength of the rock type. The characteristic coefficients of these criteria are independent of the system of units. The upper and lower bounds of these coefficients proposed by the authors provide the engineer with preliminary design guidelines, in the absence of detailed laboratory testing of a given rockfill. In this approach, knowing the unconfined compressive strength of the parent rock, the shear strength envelope of the quarried rockfill can be estimated.

The effect of confining pressure on angle of internal friction is most important in the stability analysis of rockfill slopes. A conventional analysis which employs a constant friction angle (average) provides an over-conservative factor of safety for shallow slip surfaces. If the actual variation of ϕ' with σ_n' is incorporated in design, most rockfill embankments can be raised with steeper slopes, still maintaining an adequate factor of safety greater than 1.5. The use of a constant mean ϕ' for deep seated slips can over-estimate the factor of safety, but the corresponding disparity may not be substantial.

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