

Large-scale triaxial testing of greywacke rockfill

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This Paper describes the results of a series of large-scale triaxial tests conducted on greywacke rockfill, used in dam construction in Southern Thailand. The tests are conducted at low to moderate confining stresses to relate their findings to the stability of rockfill dams. Considering the current test results in conjunction with previous laboratory data, revised failure criteria for rockfill are proposed in non-dimensional form. For both low and high confining stresses, lower and upper bounds of strength envelopes have been established, based on a wide array of granular materials. The influence of the confining stress on the shear strength of rockfill is studied in depth, and the implications of a non-linear envelope at low normal stress levels on the stability of rockfill dams are discussed. Although two parallel rockfill gradations for specimens compacted to similar porosities are considered, the exact role of particle size effect on shear strength is not examined in detail, as the difference in maximum particle sizes tested in this study is not sufficiently large.

KEYWORDS: dams; deformation; failure; laboratory tests; shear strength.

Cet article décrit les résultats d'une suite d'essais au triaxial à grande échelle réalisés sur du remblai rocheux (greywacke) utilisé pour la construction de barrages dans le sud de la Thaïlande. Ces essais sont exécutés à des pressions de cellule faibles à modérées pour rapprocher les résultats de la stabilité de barrages à enrochement. En considérant les résultats existant d'essais, en conjonction avec les données de laboratoire précédentes, des critères de rupture de l'enrochement sont proposés dans une forme sans dimensions. Pour des pressions de cellule faibles et élevées, les enveloppes basses et hautes de résistance ont été établies, basées sur un vaste ensemble de matériaux granuleux. L'influence de la pression de cellule sur la résistance au cisaillement est étudiée en profondeur et les conséquences d'une enveloppe de rupture non linéaire à contraintes normales faibles sur la stabilité de barrages à enrochement sont discutées. Bien que deux gradations parallèles d'enrochement pour des échantillons compactés à des porosités semblables aient été considérées, le rôle exact de l'effet de la taille des grains sur la résistance au cisaillement n'est pas examiné en détails car la différence entre les tailles maxis des grains testés pour cette étude n'était pas suffisante.

INTRODUCTION

Greywacke rockfill has been used in the construction of several dams in Thailand, including the recently completed Chiew Larn Dam. The Electricity Generating Authority of Thailand has proposed the construction of similar dams in several parts of Thailand in the future. Detailed testing of greywacke rockfill has not been carried out in the past: proper understanding of the shear strength properties of this material should enable better design of large rockfill dams in the future. Greywacke found in Southern Thailand is dark grey to black in colour, and can be classified as a tough sedimentary rock, formed under unstable geological environments. It is generally found without internal stratification or parting. A study of thin sections indicated that quartz is its domi-

nant mineral, and feldspar and mica are the other main constituents. The mean uniaxial compressive strength determined using eight NX core samples was found to be of the order of 135 MPa for saturated specimens.

In the past, emphasis has been placed on triaxial tests conducted at high pressures, where confining stress levels as high as 2.5–4.5 MPa have been applied (Marsal, 1973; Marachi, Chan & Seed, 1972). In most realistic situations, the shear strength of rockfill must be related to much lower confining stresses. Charles & Watts (1980) have reported that the maximum possible normal stress on a critical failure surface of a 50 m high rockfill dam is unlikely to exceed 400 kPa. Even in the case of the highest rockfill dams in South-East Asia, the maximum normal stresses are unlikely to exceed 1 MPa (Lee, 1986). Therefore, the scope of the current investigation has been to study the strength and deformation behaviour of large-scale greywacke rockfill specimens at low to medium confining pressures, and to relate these

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findings to the stability of rockfill dams. As a slope failure of a rockfill dam is generally associated with relatively low normal stresses, the shear strength of rockfill should be evaluated under nominal confinement. At elevated confining stresses, other mechanisms such as intense particle crushing can complicate the laboratory observations.

Of course, even at low applied confining pressures particle breakage may occur, because the stresses at the contact points between individual grains can still exceed their crushing strength. Nevertheless, as the rockfill can dilate under small confining pressures, the intensity of particle crushing would be considerably less than that at elevated stress levels. With a view to extrapolating the laboratory findings to field predictions, a low to moderate confining stress range of 100–600 kPa was used in this study.

One objective of this study was to modify the existing failure criteria for rockfill and jointed rock based on previous studies by de Mello (1977) and Hoek & Brown (1980) respectively. In the proposed modifications, the uniaxial compressive strength is incorporated in non-dimensional failure criteria. In this approach, not only are the coefficients that describe the rockfill failure envelopes independent of the system of units used; also, the uniaxial compressive strength takes account of the type of rockfill in question with respect to crushing strength and hardness. The influence of normal stress on the angle of shearing resistance is investigated in detail for the relatively low confining stress range considered here. The applicability of the equivalent roughness method proposed by Barton & Kjaernsli (1981) is examined for the range of normal stresses used in the tests.

Limitations of laboratory testing in simulating actual rockfill sizes and shapes have resulted in an incomplete understanding of the true behaviour of prototype rockfill, particularly in relation to shear strength mobilization. The disparity between the particle sizes in the field and in conventional triaxial specimens leads to misleading deformation response in the laboratory, due to size-dependent dilation and particle crushing phenomena. In this study, the utilization of large-scale triaxial equipment (0.3 m dia. specimens) has enabled the testing of somewhat larger rockfill, leading to fewer size-dependent errors. The difference in maximum particle size of the two gradations used was not sufficiently large to avoid the exact role of particle sizes in the rockfill behaviour to be determined. The influence of varying the initial moisture content and the compacted density, the role of particle crushing and the effect of angularity on the shear strength of rockfill have not been examined in detail.

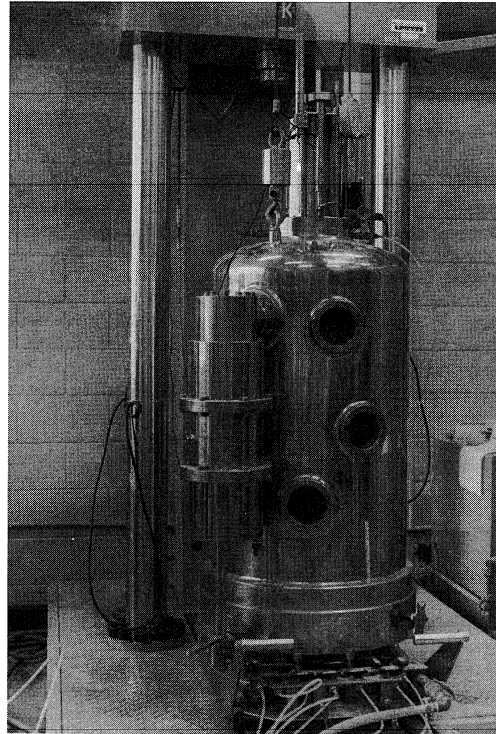


Fig. 1. Front view of large scale triaxial testing apparatus

TESTING PROGRAMME AND MATERIAL

A large-scale triaxial testing apparatus (Fig. 1) which can accommodate samples of 300 mm diameter and 600 mm high was used for testing greywacke rockfill. The apparatus consists of six main parts: the triaxial chamber, the vertical loading unit, the air pressure and water control unit, the oil reservoir and pump, the servo-control unit and the digital data acquisition system. A 3 mm thick rubber membrane was used to confine the cylindrical specimens. Isotropically consolidated drained triaxial compression tests were conducted, varying the effective confining pressure from 100 to 600 kPa in intervals of 100 kPa. This pressure range is adequate to simulate the confining pressures existing within the 95 m high Chiew Larn Dam, as the maximum K_0 value determined for this rockfill does not exceed 0.38 (Lee, 1986). The importance of testing rockfill at low confining stresses has also been emphasized by Leps (1988).

The triaxial behaviour of rockfill is influenced by several factors, including the confining stress, porosity and particle size distribution. In order to determine the role of particle sizes on the stress–

strain behaviour of rockfill, specimens of two rockfill gradations with maximum particle sizes of 38 mm and 25 mm were tested. Taking account of the principles of similitude (Fumagalli, 1969; Lowe, 1964), it may be postulated that realistic results can be obtained only if the gradations at least show

- (a) similar grain size distribution curves
- (b) comparable field and laboratory compaction
- (c) similar angularity of particles (conforming shapes)
- (d) similar sample size ratios.

With regard to condition (a), the grain size curve of the laboratory rockfill must be parallel to that of the prototype; to satisfy condition (b) the porosity of the laboratory specimens must be close to that of the compacted rockfill in the field. Condition (c) is difficult to fulfil exactly, but the laboratory particles must be similar in shape to the actual rockfill, although smaller in size. Fig. 2 compares the parallel gradations A and B used in the testing programme with the grain size distribution of rockfill used in the Chiew Larn Dam. The difference between the gradation curves before and after testing (at $\sigma_3' = 600$ kPa) indicates the extent of particle breakage during shearing. A reduction in d_{50} of at least 30% is observed for both gradations A and B at the maximum confining pressure of 600 kPa. At small cell pressures (less than 200 kPa), the degree of grain crushing is insignificant.

Table 1 summarizes the characteristics of the

laboratory rockfill, including the uniformity coefficient C_u and the coefficient of curvature C_c , before testing. The sample size ratio is defined by the diameter of the triaxial specimen (300 mm) divided by the mean diameter of the maximum particle size. The effect of size ratio on the behaviour of rockfill specimens in triaxial testing has been discussed in depth by several investigators. Fagnoul & Bonnechere (1969) and Nitchiporovitch (1969) proposed a minimum sample size ratio of 5. Marachi (1969) concluded that a size ratio of at least 6 must be employed in order to minimize size effects for rockfill specimens with less than 30% of particles in the maximum sieve size range. In this study, size ratios of 8 and 12 were adopted for rockfill gradations A and B respectively.

The test specimens were compacted within the protective membrane in several layers, each 50–60 mm thick, using a hand vibrator. For both gradations, an initial compacted porosity of the order of 30% was achieved. The initial water content of the rockfill was $\sim 5\%$; after compaction the mean dry density of the specimens was determined to be ~ 18.5 kN/m³. During the construction of the Chiew Larn Dam, each compacted rockfill layer varied in thickness from ~ 0.6 to 1.0 m in the field, producing a dry density of greater than 18.0 kN/m³, achieved by heavy vibratory rollers (10 t), with a compacted porosity similar to that obtained in the laboratory.

Terzaghi (1960) noted that dam settlements after impounding can be significant for soft rock-

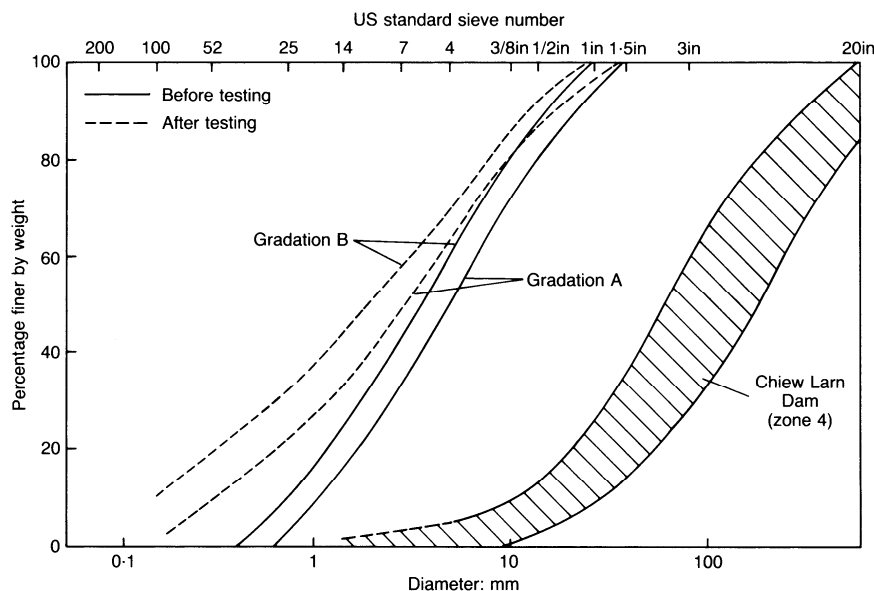


Fig. 2. Particle size distribution curves of greywacke rockfill

Table 1. Grain size characteristics of greywacke rockfill: C_u and C_c are determined from the grain size distribution curve

Gradation	d_{max} : mm	d_{50} : mm	C_u	C_c	Size ratio
A	38.1	4.9	6.0	0.9	8
B	25.4	3.6	6.0	0.9	12

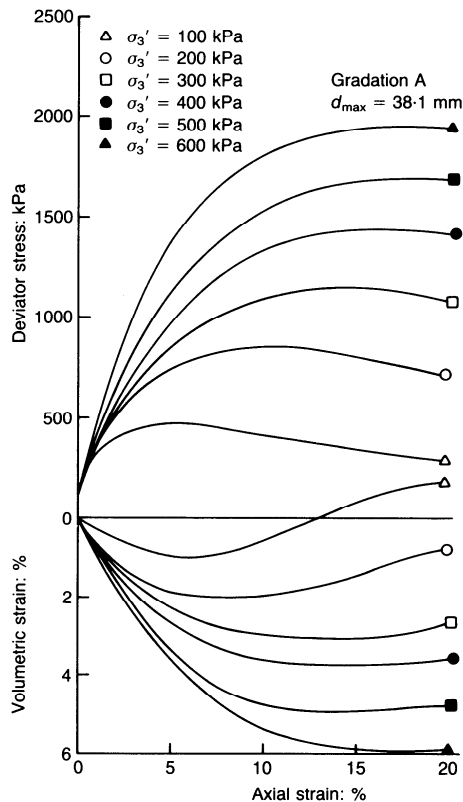
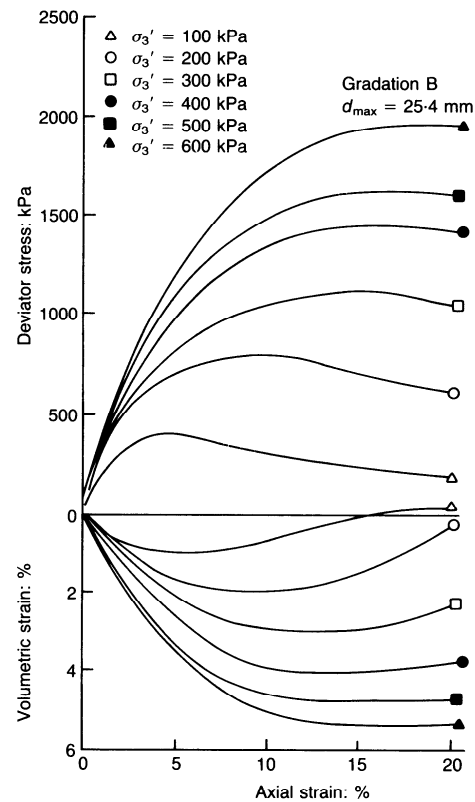
fill (e.g. schist), whereas for harder granites such settlements are generally small. He related this phenomenon to the reduction in strength upon saturation. In the field, wetting occurs after impounding, and the resulting settlements depend on the initial water content of the rockfill. It may be anticipated that the greater the placement water content is, the smaller will be the settlements after impounding. In the construction of the Chiew Larn Dam, although greywacke rockfill was placed relatively dry with a mean moisture content $<10\%$, significant settlements have not been reported after impounding. In the laboratory, before loading, the initially dry rockfill specimens were saturated by applying a back pressure of 100 kPa. No significant settlements were recorded during this saturation phase. This implies that a relatively small water content (5–

10%) during placing of greywacke rockfill is sufficient to minimize settlements due to wetting. Nevertheless, for soft rockfill materials such as reef limestones, the settlements during saturation can be significant (Brown, 1988).

EXPERIMENTAL RESULTS AND DISCUSSION

Stress-strain behaviour

The results of the isotropically consolidated drained triaxial compression tests are shown in Figs 3 and 4 for gradations A and B. As expected, the peak deviator stress increases with increasing applied confining stress. The gradual decrease in deviator stress after peak indicates the ductile strain-softening response of rockfill. Similar behaviour has been reported for heavily fractured rock (Ito, 1983; Indraratna & Kaiser, 1990). For

**Fig. 3. Stress-strain behaviour of greywacke rockfill (gradation A)****Fig. 4. Stress-strain behaviour of greywacke rockfill (gradation B)**

the rockfill samples subjected to very small confining pressures (≤ 200 kPa) dilation is pronounced, whereas at higher confining pressures ($\sigma_3' > 300$ kPa), dilation is suppressed even at axial strains exceeding 15%. At low confining stresses, gradation A shows more dilation than gradation B; this can be attributed to its larger particle sizes. However, at higher σ_3' , only a slight difference in the volumetric strain response is observed between the two gradations, suggesting that the particle size effect may be of secondary importance as compared with the influence of confining pressure on the volumetric strains. However, it is important to note that quantification of the role of particle size would require further testing, with a much greater difference in grain size.

The stress-strain response of gradations A and B shows that the initial tangent stiffness increases with increasing confining pressure. The actual magnitudes of the initial deformation modulus as a function of the confining stress are shown in Fig. 5 (each data point represents the mean of three tests). The difference between the initial moduli of any two corresponding samples at the same confining pressure is relatively small, and diminishes with increasing σ_3' . Furthermore, comparison of Figs 3 and 4 shows that the peak deviator stresses of the corresponding samples are almost the same. These results suggest that for parallel gradations with adequate sample size ratios, the influence of particle size may diminish if the initial porosities of the test specimens are similar. If the compacted field porosity is significantly different from that of the test specimens, the actual deformation response cannot be simulated, even if parallel gradations are used in the

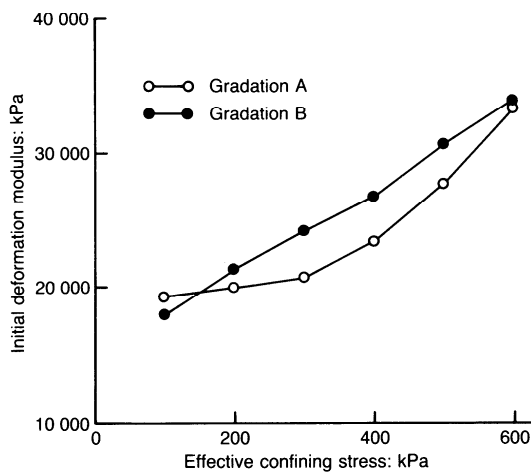


Fig. 5. Variation of initial deformation modulus with effective confining stress

laboratory. Therefore, the relative densities of the compacted laboratory specimens must reflect the compaction requirements in the field.

The volumetric strains measured at the peak deviator stress not only increase almost linearly with the confining pressure, but are also independent of the particle gradation (Fig. 6). Lee (1986) has made a similar observation, even for a greater particle size difference. Data reported by Marachi *et al.* (1972) for other types of rockfill indicate that the volumetric strains at failure increase with increasing σ_3' at a gradually decreasing rate, approaching a constant value at high σ_3' (2.5 MPa). While the results obtained in this study for much lower confining stresses (100–600 kPa) are not strictly in conflict with Marachi *et al.* (1972), they cannot be extrapolated to predict behaviour at very high stress levels. The axial and radial strains at $(\sigma_1' - \sigma_3')_{\max}$ increase at a diminishing rate as σ_3' is increased. It is interesting to note that the radial strains at $(\sigma_1' - \sigma_3')_{\max}$ approach a constant value (6%) at confining pressures beyond 300 kPa. On the basis of the triaxial strain response discussed above, it may be concluded that the confining stress σ_3' is the dominant factor controlling the deformation behaviour of rockfill, while the particle size effect seems to be of secondary importance, unless the difference between the maximum particle sizes is considerable.

Shear strength and failure criteria

Well-documented studies have indicated that the friction angle of sands decreases with increasing cell pressure in drained triaxial tests (Vesic & Clough, 1968; Bishop, 1966; Ponce & Bell, 1971). A comprehensive series of large-scale triaxial tests (1 m dia.) conducted on many types of rockfill was described by Marsal (1967) and Marachi (1969). They found that the principal stress ratio for drained tests $(\sigma_1'/\sigma_3')_f$ was considerably increased at low confining stresses. Charles & Watts (1980) verified this phenomenon with smaller rockfill specimens (0.23 m dia.) in tests conducted on sandstone, basalt and slate. The experimental data are summarized in Fig. 7, which also shows results obtained for the greywacke rockfill samples (0.3 m dia.) for comparison.

Although these specimens are weaker than a number of other materials reported in the literature, greywacke is accepted in Thailand, as a satisfactory rockfill. In contrast, the soft, low-grade slate was considered an unsuitable rockfill by Charles & Watts (1980). The relatively low principal stress ratio of the greywacke specimens may be attributed to their lower initial density and the relatively low uniaxial compressive

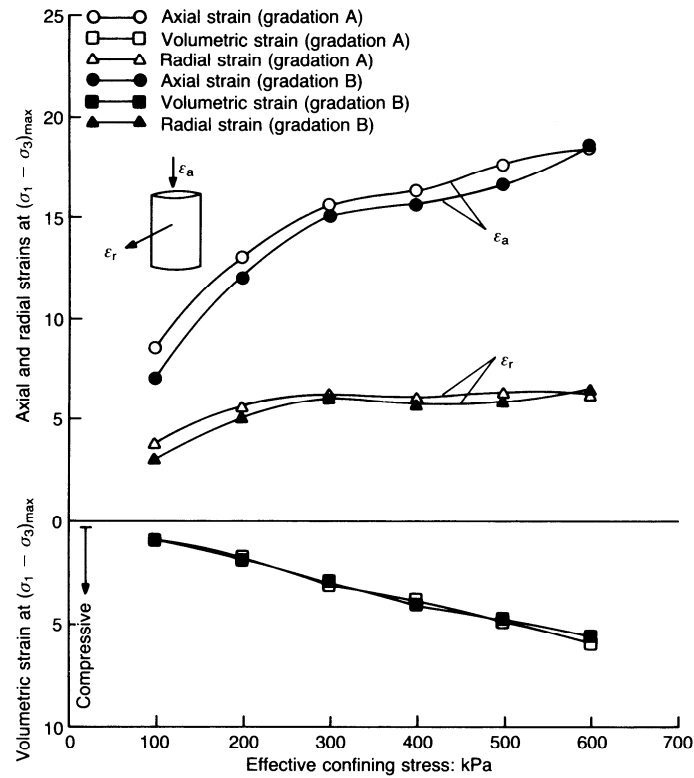


Fig. 6. Influence of effective confining stress on sample strains at peak deviator stress

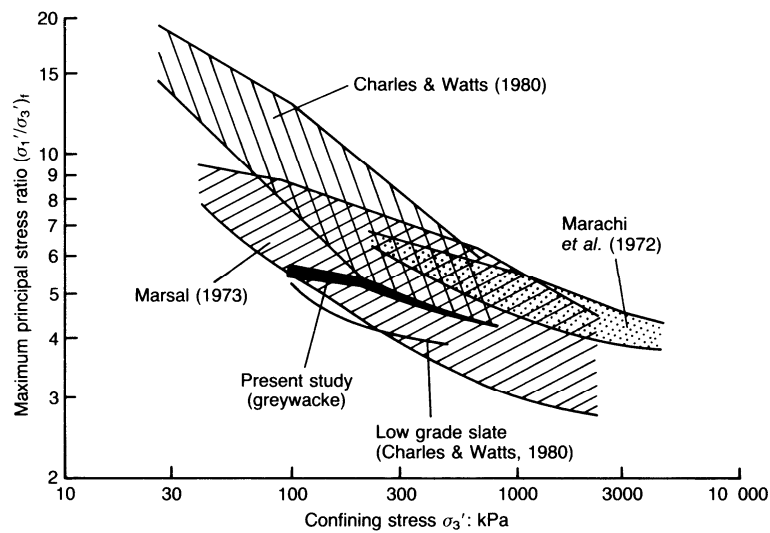


Fig. 7. Variation of effective principal stress ratio at failure with effective confining stress for various rockfills

strength of greywacke as compared with highly compacted, harder rock fragments. The degree of compaction and hence the initial porosity can have a major effect on the shear strength. At a confining stress of 200 kPa, an increase in porosity of 1% can reduce the angle of friction by at least 0.5° (Marachi, 1969).

The strength envelopes for the greywacke gradations A and B considered in this study are shown in Fig. 8(a) and (b). Both envelopes corresponding to the low confining stress region clearly reveal non-linearity, and pass through the origin, indicating zero cohesion as expected of a granular material. De Mello (1977) proposed a non-linear failure criterion for typical rockfill

$$\tau_n = a\sigma_n^b \quad (1)$$

where the constants a and b are considered to be characteristic parameters obtained by curve fitting. The physical significance of these constants is not clear, because the value of a not only depends on the system of units used, but 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 useful, as a wide array of materials can then be compared directly within the framework of similitude (Indraratna, 1990).

The significance of the uniaxial compressive strength σ_c as a normalizing parameter was recognized by Hoek & Brown (1980) for rock samples tested in triaxial compression, and allowed evaluation of the characteristic constants appropriate for both intact and fractured rocks. Rockfill can be regarded as intensely fractured rock, and further breakage of individual fragments during shearing is a function of particle angularity, confining pressure and the point load index related to σ_c of the parent rock.

Marsal (1973) introduced the particle breakage index B_g to characterize rockfill behaviour by comparison of the grain size distribution curves before and after testing. However, the magnitude of B_g is sensitive to the applied confining stress, irrespective of rock type. Consequently the value of B_g cannot be interpreted as a material property. In this study, the uniaxial compressive strength has been incorporated in defining the failure envelope of rockfill. In reality, σ_c can be estimated more reliably than B_g for any rockfill by conducting basic index tests on even an irregular lump. It is proposed that the failure envelope in a non-dimensional form can be expressed as a modification of the de Mello (1977) criterion by

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

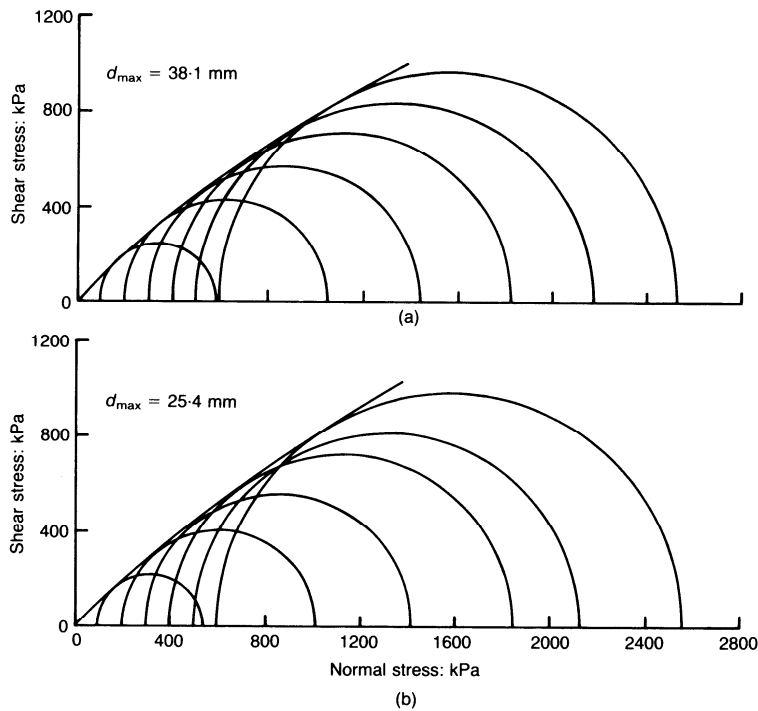


Fig. 8. Mohr-Coulomb failure envelopes of greywacke rockfill for: (a) gradation A; (b) gradation B

The constants a and b are dimensionless, hence independent of the system of units used for the stresses. For linear Mohr–Coulomb materials, 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 represents the deformation response of the rock-

fill, including to some extent the effect of dilation and particle sizes.

Figure 9 shows the variation of shear strength with the normal stress for several types of rockfill tested at different confining stress levels. For comparison, the present results for greywacke rockfill are plotted in Fig. 9(a) together with those discussed by Charles & Watts (1980) for other types of rockfill tested at low normal stresses. The failure envelopes for the two greywacke gradations (A and B) coincide except at the lowermost

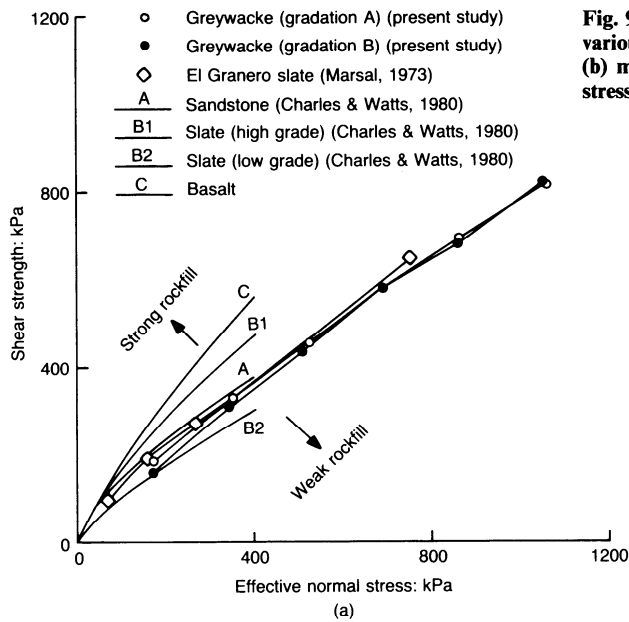
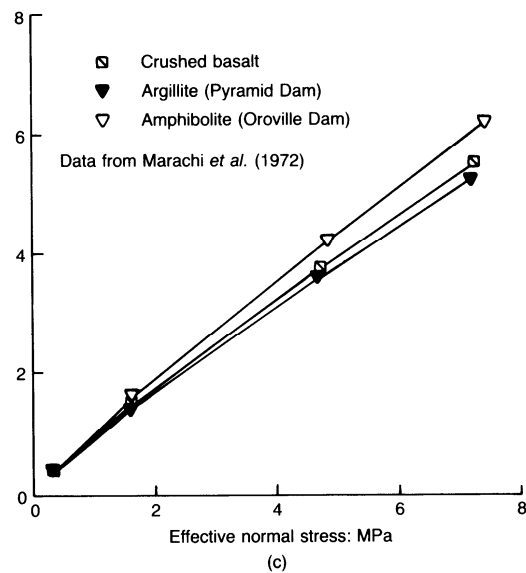
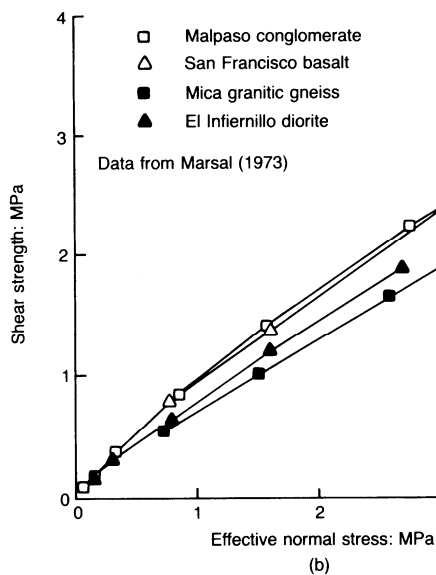


Fig. 9. Variation of shear strength with normal stress for various rockfill types: (a) low to medium stress ranges; (b) medium to high stress ranges; (c) high to very high stress ranges



normal stress values, i.e. the effect of particle sizes is minimized at higher normal stresses, where dilation is inhibited. It is significant that all the rockfill samples shown in Fig. 9(a) have comparable particle sizes and initial porosities. In this stress range, the non-linearity of the failure envelopes is pronounced, and a clear distinction between the strong (hard) and weak (soft) rockfill can be observed. Fig. 9(b) and (c) illustrates the behaviour of rockfill at higher stress levels, using data from Marsal (1973) and Marachi *et al.* (1972) respectively. Note that at elevated effective normal stresses, the failure envelopes approach linearity, hence the conventional Mohr–Coulomb analysis is sufficient to represent the failure of rockfill.

Figure 10 plots the normalized shear strength against normal stress relationships on log scales. Together with the present test results, experimental data are presented for a wide range of effective normal stresses from 100 kPa–8 MPa. The corresponding data points for each rockfill specimen can be joined by a straight line (omitted for

clarity) with a regression coefficient $r^2 > 0.95$. The associated constants a and b (equation (2)) are given in Table 2 for all these rockfill materials. For this wide range of rockfill, the values of a and b are less than 0.6 and 0.9 respectively, except for Malpaso conglomerate. For very low confining stresses (< 100 kPa), b tends to be less than 0.8; at high confining stress levels (> 1.5 MPa) it approaches 0.9. Irrespective of the compressive strength of rock, particle sizes, angularity, initial porosity and initial water content, the above experimental data fall within a narrow band defined by the boundaries given in Table 3.

Alternatively, the failure of rockfill specimens may be represented by the major and minor principal stresses at failure σ_{1f}' and σ_{3f}' : this procedure is often adopted in rock mechanics. Incorporating the uniaxial compressive strength σ_c , the following normalized expression is suggested to represent failure

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

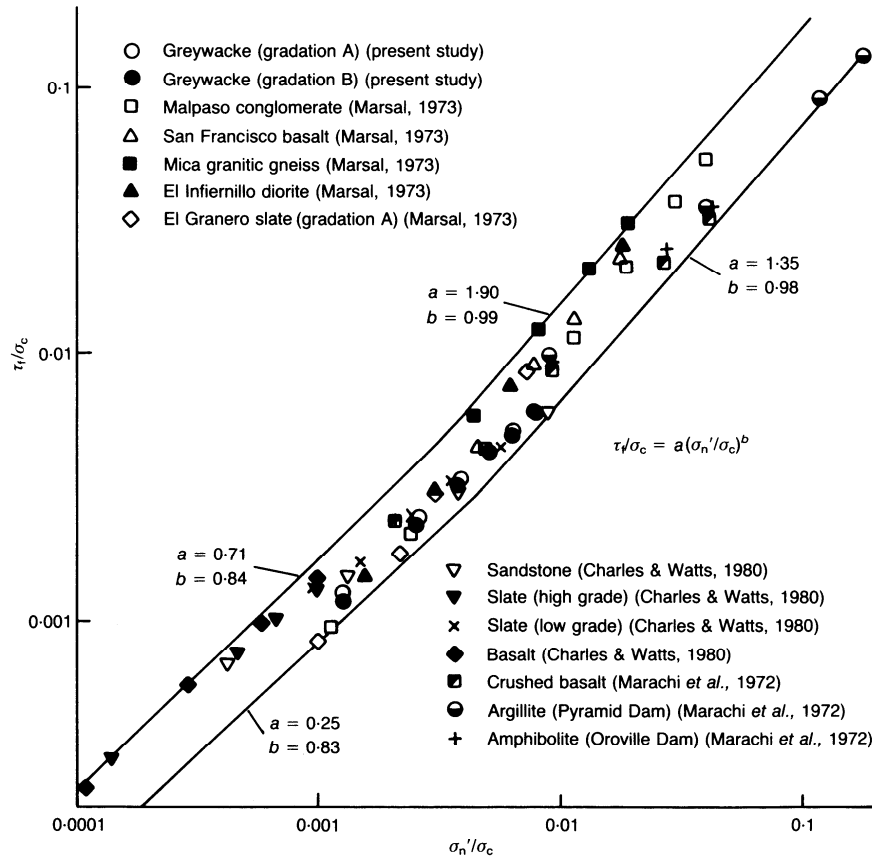


Fig. 10. Normalized shear strength–normal stress relationship for various rockfills

Table 2. Coefficients of proposed failure criteria for various types of rockfill

Type of rockfill	σ_c : MPa	Initial porosity: %	d_{max} : mm	d_{50} : mm	C_u (d_{60}/d_{10})	Sample size ratio	Range of σ'_3 : MPa	Coefficient			
								a	b	α	β
Greywacke (grad. A)*	136	30-32	38	4.9	6	8	0.10-0.60	0.39	0.86	1.61	0.82
Greywacke (grad. B)*	136	30-32	25	3.6	6	12	0.10-0.60	0.47	0.89	2.14	0.87
Malpaso conglomerate†	75	26-30	200	22	63	5.7	0.04-0.60	0.46	0.86	1.75	0.76
San Francisco basalt†	175	25-29	80	7	11	5.7	0.60-2.50	0.76	0.93	3.68	0.90
Mica granitic-gneiss†	125	24-28	200	51	14	5.7	0.47-2.47	0.42	0.84	1.88	0.81
El Infernillo diorite†	105	33-38	200	68	5	5.7	0.45-2.47	0.42	0.89	1.97	0.88
El Granero slate†	90	30-35	200	105	10	5.7	0.09-0.50	0.37	0.84	2.09	0.83
Sandstone†	120	20	38	5	—	6	0.50-1.69	0.47	0.90	2.19	0.87
Slate (high grade)†	312	25	38	5	51	6	0.04-0.46	0.35	0.77	1.06	0.72
Slate (low grade)†	58	32	38	4	—	6	0.03-0.70	0.14	0.68	0.34	0.57
Basalt†	360	25	38	13	13	6	0.10-0.52	0.29	0.76	0.94	0.72
Crushed basalt§	175	—	50	13	7	6	0.10-0.52	0.24	0.73	0.53	0.66
Argillite (Pyramid Dam)§	40	—	50	13	7	6	0.03-0.70	0.33	0.77	1.34	0.72
Amphibolite (Oroville Dam)§	175	—	50	13	7	6	0.20-4.50	0.50	0.87	2.14	0.84
							0.20-4.50	0.58	0.88	2.62	0.85
							0.20-4.50	0.61	0.90	2.78	0.86

* Present study.

† Marsal (1973).

‡ Charles & Watts (1980).

§ Marachi *et al.* (1972).

Table 3. Boundaries of experimental data applicable to normal stress ranges

	$\sigma_n' = 0.1-1 \text{ MPa}$		$\sigma_n' = 1-7 \text{ MPa}$	
	a	b	a	b
Lower bound	0.25	0.83	1.35	0.98
Upper bound	0.71	0.84	1.90	0.99

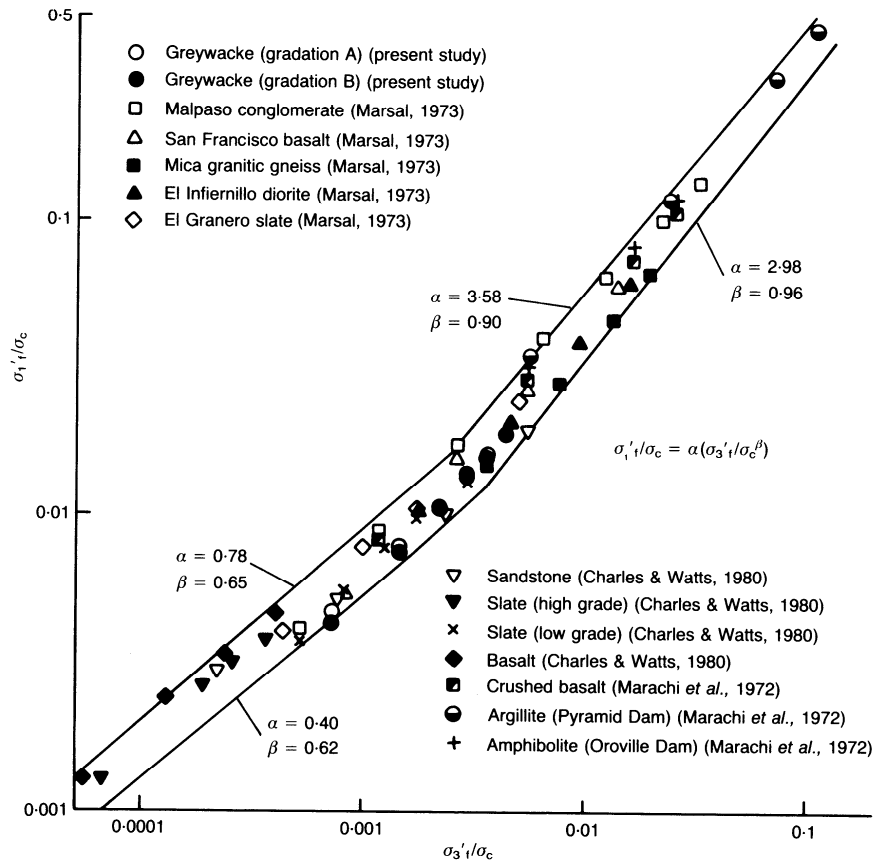
Table 4. Upper and lower boundaries of peak (failure) response

	$\sigma_n' = 0.1-1 \text{ MPa}$		$\sigma_n' = 1-7 \text{ MPa}$	
	α	β	α	β
Lower bound	0.40	0.62	2.98	0.96
Upper bound	0.78	0.65	3.58	0.90

This approach can be regarded as an extension of the empirical method discussed by Hoek & Brown (1980), who proposed a square-root relationship between the principal stresses for the failure of both intact and jointed rocks. Marsal (1973) and Indraratna (1990) have also discussed the relevance of representing failure criteria in terms of principal stresses at failure, with particular reference to triaxial testing. Equation (3) emphasizes that in the absence of any confining pressure, the strength of the rockfill sample is

negligible. The relationships between the major and minor principal stresses at failure for various rockfill materials normalized by their uniaxial compressive strength are plotted in Fig. 11. Notwithstanding the differences in this wide array of granulated materials, their peak (failure) response can be represented by a narrow band width defined by the upper and lower bounds of α and β given in Table 4.

The coefficients α and β determined for individual rockfills are given in Table 2. For most rock-


Fig. 11. Normalized relationships of major and minor principal stresses at failure for various types of rockfill

fills, although the values of b and β are quite similar, the value of α is generally greater than unity. For very low confining stresses (< 100 kPa) the value of β is less than 0.75; at high stress levels (> 1.5 MPa) it is often greater than 0.85. 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 magnitudes of a and α represent $\tan \phi'$ and the effective principal stress ratio respectively for a given confining stress.

Influence of confining stress on friction angle

The variation of the drained friction angle ϕ' of greywacke is plotted against the effective normal stress in Fig. 12, where each point represents the average of three independent tests. As the confining pressure and hence the normal stress is increased, the drained friction angle for rockfill gradations A and B decreases from 45° to 32° and from 43° to 33° respectively. Although the friction angle of gradation A drops faster than that of gradation B, as the normal stress is increased to 1 MPa, ϕ' for both gradations attains the same value irrespective of the particle sizes. The surprisingly large reduction in the angle of shearing resistance at high confining pressures is probably associated with the significant increase in crushing of angular particles. At low stress levels, parti-

cle crushing is small for most rockfill (Leps, 1988). While the data obtained by Marachi *et al.* (1972) indicated a slight reduction of ϕ' with increasing particle sizes, a contradicting trend was reported by Tombs (1969) for maximum grain sizes varying from 10 to 75 mm. The effect of particle sizes on ϕ' remains a more complex phenomenon than the marked influence of confining stress. The role of the grain size distribution of the angle of friction cannot be verified purely on the basis of the current test results. Barton & Kjaernsli (1981) proposed that the drained friction angle ϕ' of rockfill can be evaluated from the empirical expression

$$\phi' = R \log \left(\frac{s}{\sigma_n'} \right) + \phi_b' \quad (4)$$

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 fraction of the uniaxial compressive strength of the parent rock; σ_n' is the effective normal stress and ϕ_b' is the basic friction angle of smooth, planar unweathered rock surfaces.

Figure 12 also compares the test results of greywacke rockfill with the values predicted by the equivalent roughness method. In determining the angle of friction using this empirical method, the equivalent roughness R was taken as 6.5 for quarried (sharp to angular) greywacke rockfill of compacted porosity 30%. The mean uniaxial compressive strength and the basic friction angle for greywacke from southern Thailand have been given by Cheng (1986) as 136 MPa and 25° . The experimental results indicate good agreement with the predicted values at low normal stresses. At high normal stress levels, the laboratory friction angle decreases faster than is predicted by the equivalent roughness method. It is important to realize that the structural component of the frictional resistance is not intended to model the particle size effect or the potential breakage of angular grains. It may be argued that due to the dependence of the parameter s on the uniaxial compressive strength σ_c , the term $R \log (s/\sigma_n')$ would not decay as rapidly as the trend shown by the current test results. On the basis of these observations, it may be concluded that the Barton & Kjaernsli (1981) approach should not be extended to extrapolate the frictional behaviour of rockfill at high effective normal stress levels, particularly beyond 1 MPa. Nevertheless, the equivalent roughness method is adequate to estimate ϕ' at low normal pressures for the tested greywacke rockfill.

Figure 13 compares the findings of the present study with the summary of triaxial test data for

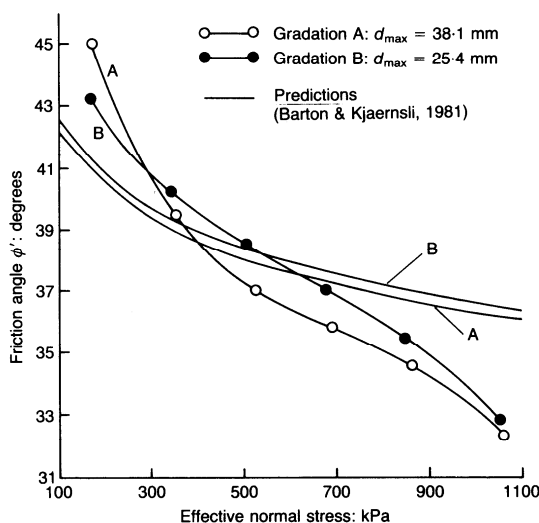


Fig. 12. Comparison of current test results with predictions from the equivalent roughness method

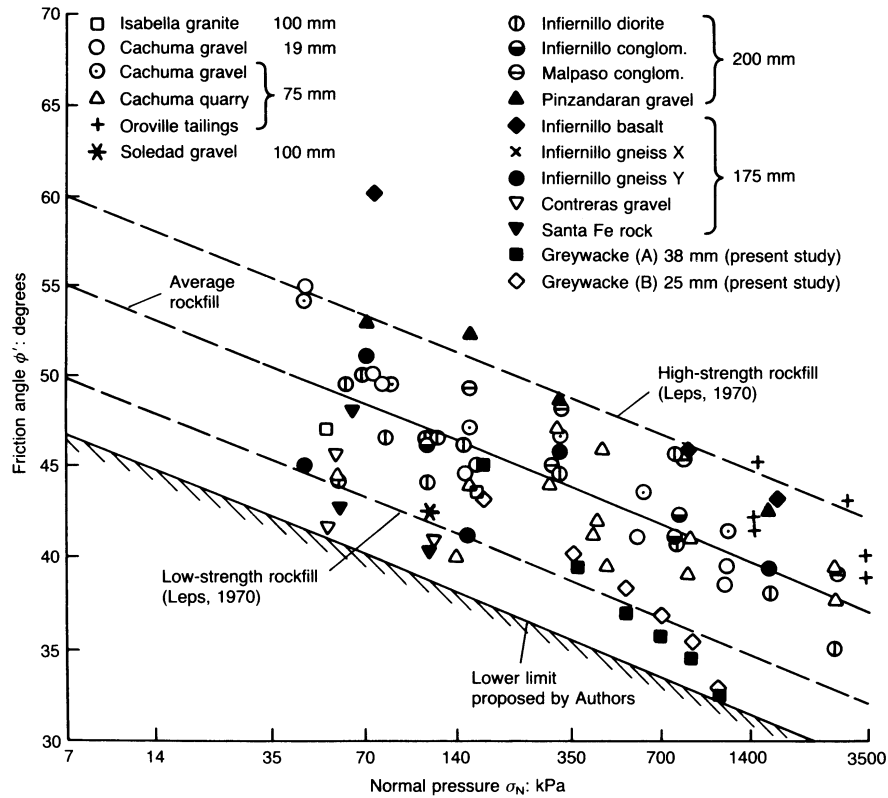


Fig. 13. Comparison of current test data with previous studies

other rockfill presented by Leps (1970). The data for greywacke rockfill fall between this lower bound and the line of 'average rockfill', except where the effective normal stresses exceed 700 kPa. Considering the test results of Contreras gravel and Santa Fe rock together with the current data on greywacke rockfill, it seems that the lower boundary proposed by Leps (1970) for low density rockfill is slightly overestimated. A more conservative lower bound (3° less) as shown by the hatched line in Fig. 13 is suggested by the Authors. Note also that no experimental data are available for confining pressures below 40 kPa, indicating the difficulty of testing rockfill samples at such low confining stresses due to the lack of cohesion. In the present study, a minimum confining pressure of ~ 50 kPa was required in order to prevent the samples from bulging during the saturation stage.

Influence of relative density and degree of compaction

The initial porosities of the test specimens were maintained at 30–32%, in order to relate to the actual field conditions of the Chiew Larn Dam.

For both gradations A and B of this greywacke rockfill, the corresponding relative densities were between 62–65%, as determined from several test specimens. The effect of varying the initial degree of compaction on the angle of shearing resistance was not investigated in detail, because the primary objective was to evaluate the effect of confining pressure (normal stress) on the shear strength of greywacke rockfill. Nevertheless, Brown (1988) has shown that excessive compaction encourages particle breakage, although the initial friction angle may be enhanced for the same confining pressure.

Excessive compaction of wet rockfill may not only induce crushing of angular fragments, but also cause excess pore pressures if the permeability is too low. In this respect, it is always good laboratory practice to saturate the rockfill specimens before drained loading, so that the friction angles obtained in this manner are more realistic (conservative) than the higher values encountered for dry specimens. As the downstream shell of a dam would usually be unsaturated, the stability of the downstream slope may be enhanced due to the additional shear resist-

ance provided by suction, depending on the particle size distribution and the relative density.

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; at extremely high stress levels it may even approach values close to 30° . The non-linear strength envelope quantifies this phenomenon adequately. In terms of 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 the presence 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 the effective normal stress, the prediction of critical slip surfaces or sliding wedges becomes questionable.

Table 5 summarizes some important characteristics of rockfill dams in Thailand. It is quite clear that the smaller dams (<40 m high) are often designed at steeper slopes than their larger counterparts. For such small dams, the critical failure surface may be predicted reasonably well by using a constant friction angle and still maintain an acceptable factor of safety, without having to provide berms or external support at the toe. For instance, the Ubol Ratana dam has an average downstream slope of almost 40° , whereas the slopes of the highest Sri Nagarind dam are less than 30° .

Some research has been conducted on particle crushing during compression. Although particle breakage has been quantified in the laboratory by Marsal (1973) under high confining stresses, in practice crushing of particle edges may occur during compaction of highly angular fragments. For less angular rockfill, crushing may not be a

serious consideration. Nevertheless, significant crushing of naturally soft or weathered rock during field compaction may cause non-compliance with the permeability requirements (free draining), and enhance the risk of developing construction pore pressures. It has been shown by Balasubramaniam, Lee & Wijeyakulasuriya (1987) that any significant development of small pore pressures can affect the effective shear strength of rockfill. Therefore, good engineering practice should not allow any excess pore pressure development in the rockfill. In this respect, the placement moisture content, grain size distribution and degree of compaction in the field have an important role to play. Penman (1978) has proposed that the average permeability of rockfill should not be less than 0.001 cm/s, so that the amount of fines can be limited to minimize the risk of excess pore pressure development.

CONCLUSIONS

Greywacke rockfill tested in this study shows a similar engineering behaviour to many other typical rockfills. Although the particle size and angularity of the rockfill also influence the stress-strain behaviour, including dilation, the effect of confining stress on the shear strength is of primary importance. While the degree of grain crushing may be enhanced at elevated confining pressures, for competent rockfill subjected to realistic normal stresses particle breakage may not be a serious influential factor. The initial density of rockfill, however, is important as it is linked directly to the degree of compaction. The laboratory data on shear strength depend on the initial porosity and relative density of the compacted test specimens. Therefore, while excessive field compaction is not justifiable, sufficient compaction with conventional machinery must be ensured, so that the laboratory and field porosities are similar.

The angle of shearing resistance and the associated failure envelope of rockfill are directly related

Table 5. Details of some rockfill dams in Thailand (Thai National Committee on Dams, 1977)

Dam	Dam height: m	Crest length: m	Slope		Type of rockfill
			Upstream	Downstream	
Sri Nagarind	140	610	1:2.0	1:1.8	Limestone and quartzite
Chiew Larn	90	700	1:2.0	1:1.8	Greywacke
Chulabhorn	70	700	1:1.7	1:1.6	Sandstone
Pattani	85	422	1:2.0	1:1.8	Greywacke
Nam Pung	40	1120	1:2.0	1:1.7	Conglomerate
Ubol Ratana	32	800	1:1.3	1:1.3	Sandstone
Sirindhorn	42	940	1:1.8	1:1.6	Siltstone

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 the 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, and their values for a variety of rockfill types have been determined from low to very high stress ranges. 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. Using this approach, if the unconfined compressive strength of the parent rock is known, the shear strength envelope of the quarried rockfill can be estimated.

The effect of confining pressure on angle of internal friction is very important in the stability analysis of rockfill slopes. A conventional analysis that 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 the design, most rockfill embankments can be raised with steeper slopes while maintaining an adequate factor of safety greater than 1.5 (Wijewardena, 1991). The use of a constant mean ϕ' for deep-seated slips may overestimate the factor of safety, but the disparity may not be substantial.

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