Indraratna, B., Wijewardena, L. S. S. & Balasubramaniam, A. S. (1994). *Géotechnique* **44**, No. 3, 539–543

DISCUSSION

Large-scale triaxial testing of greywacke rockfill

B. INDRARATNA, L. S. S. WIJEWARDENA and A. S. BALASUBRAMANIAM (1993). *Géotechnique* **43**, No. 1, 37–51

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In the Paper it is mentioned that the moduli reported are the mean of three tests at each confining pressure; if this is so, are the data in Figs 3–6 and in Fig. 8 averages of three tests, and what was the variability of the individual data, as the results presented show great consistency?

There seems to be something amiss with the parameters shown in Table 3. I have replotted the lines represented by the values in Table 3 and found that they do not correspond with those shown in Fig. 10. I have determined values of a and b from the lines shown in Fig. 10 and compared them in Table 6 with those given in Table 3. I took the boundary between low and high stress ranges to be a normalized stress (σ_n'/σ_c) of 0·004, as shown in Fig. 10. The bounds shown in Fig. 10 have some b values greater than unity; this has the paradoxical result that the Authors' failure criterion expressed in equation (2) predicts that the failure shear strength would reduce with increasing σ_c of the rock clasts. This is a serious deficiency in the lines shown in Fig. 10.

I have not done the same exercise for Table 4 compared with Fig. 11 and wonder whether the Authors checked the consistency of these data with the bounds given.

I also had difficulty in reconciling some of the data given in Table 2 and Figs 10 and 11. For example, for Slate (high grade) the data in Table 2 ($\sigma_c = 312$ MPa, minimum $\sigma_3' = 0{\cdot}10$ MPa) would appear to indicate that the minimum data point for this material should be at $\sigma'_{3f}/\sigma_c = 0{\cdot}0003$, not the 0·000 08 shown in Fig. 11. Which is correct?

Table 6. Differences in values of parameters given in the Paper and those determined from Fig. 10

	Low stress range		High stress range	
	a	*b*	*a*	*b*
Lower bound				
Author	0·25	0·83	1·35	0·98
Mostyn	0·28	0·85	0·76	1·03
Upper bound				
Author	0·71	0·84	1·90	0·99
Mostyn	0·65	0·86	2·11	1·07

Also the data for the Mica granitic gneiss plot virtually along the upper bound in Fig. 10 and along the lower bound in Fig. 11. Are these correct? Using the data in Table 2 to plot the failure envelope approximately for this material indicates that the data are probably plotted incorrectly in Fig. 10. Thus there appear to be errors in both Figs 10 and 11.

The boundary between high and low stress for the bounds is given as an effective normal stress of 1 MPa in Tables 3 and 4; this cannot be correct. The crossover should not be in terms of σ_n' but in terms of σ_n'/σ_c for Table 3 and Fig. 10, and in terms of σ'_{3f}/σ_c for Table 4 and Fig. 11. For the values of a and b given in Table 3 the lower bound crossover occurs at $\sigma_n'/\sigma_c =$ 0·000011 and the upper bound crossover at 0.0014, not as shown in Fig. 10. The values for the parameters I used are 0.0039 and 0.0037 for the upper and lower bounds respectively, which I would suggest can be taken as 0·004, and is as shown in Fig. 10. The correct value of crossover gives a crossover of 0·02 MPa between low and high stress bounds for a very weak rock rockfill (say, $\sigma_c = 5$ MPa), and thus for all intents and purposes only the high stress range counts in most problems. Similarly a crossover of 1·2 MPa is obtained for high strength rock rockfill (say, $\sigma_c = 300$ MPa) and thus only the low stress bounds will be effective in many problems. Both bounds will have an impact for many intermediate strength rock rockfills.

The Authors' work indicates that both b and β increase with increasing normalized stress. This fact and examination of Figs 10 and 11 indicate that the upper and lower bounds for the high normalized stress range must also form lower bounds to the upper and lower bounds, respectively, for the low normalized stress range. As b is approximately equal to unity for the upper and lower bounds over the high normalized stress range, the failure envelopes shown in Figs 10 and 11 are approximately linear with an equivalent ϕ of 37° for the lower bound and of 65° for the upper bound. These bounds are so wide as to be of less help than any reasonable geotechnical engineer's guesstimate. In fact, in my experience bounds would seriously overestimate the actual strength of much rockfill, especially that composed of weak rock. The bounds given in the Paper could be dangerous if applied to rockfill comprised of weak to medium strength rock (say, σ_c less than 15 MPa) which is quite common in the coal mining industry. In my experience the actual (not lower bound) angle of friction for such material is often less than 30° if zero cohesion is assumed. The anomaly probably reflects the fact that the lowest strength rock in the Authors' data had $\sigma_c = 40$ MPa. Further these bounds (37° and 65°) do not compare well with the data the Authors show in Fig. 13, where only one point is above 56° and several are below 37°. I have both bounds would seriously overestimate the actual strength of much rockfill, especially that composed of weak rock. The bounds given in the Paper could be dangerous if applied to rockfill comprised of weak to medium strength rock (say, $\sigma_c < 15$ MPa) which is quite common in the coal-mining industry. In my experience the actual (not lower bound) angle of friction for such material is often less than 30° if zero cohesion is assumed. The anomaly probably reflects the fact that the lowest strength rock in the Authors' data had $\sigma_c = 40$ MPa. Further these bounds (37° and 65°) do not compare well with the data the Authors show in Fig. 13, where only one point is above 56° and several points are below 37°. I have assumed that the data in Fig. 13 are secant friction angles.

I prefer the use of a linear failure envelope and believe that most geotechnical engineers have a very good feeling for the parameters c and ϕ and can quickly look at data and ascertain whether or not they support particular parameters. Unfortunately this appears to be almost impossible for a, b and α, β as virtually identical failure envelopes can have very different parameters and the relationships between the parameters and actual strength are almost impossible to fathom. This would inhibit their use and may lead to mistakes in their application.

Given a typical set of data, say that for Greywacke A in Table 2 and shown in Fig. 8(a), it is possible to replace this with a good fitting linear failure envelope with $c' = 55$ kPa and $\phi = 36°$. There is a maximum difference between the linear and non-linear envelope of about 3% over the range for which there are test data (i.e. 0·15–1·1 MPa). The linear envelope would slightly overestimate the strength for $\sigma_n' < 0{\cdot}15$ MPa (where there are no data) but if the overestimate is of concern then a zero cohesion and $\phi = 50°$ could be adopted over this low stress range.

Even though the data in Fig. 8(a) show little variability I understand that three tests were completed at each confining pressure and thus the full data set may well be more variable. I believe that the (linear) Mohr–Coulomb envelopes given here are more than adequate considering that for the Greywacke

(*a*) the maximum size of the actual in-place rockfill is one to two orders of magnitude larger than that in the triaxial specimens (i.e. 1000–2000 mm compared with 38·1 mm)
(*b*) the coefficient of curvature of the particle size distributions C_c of the specimens (especially after testing) is different from that of the in-place rockfill
(*c*) there is inevitable variability both in the laboratory and in place, especially of σ_c which would not be constant within almost any geological environment.

Examination of a more curved failure envelope further illustrates some of the problems with envelopes such as those proposed by the Authors. Their data indicate that the Sandstone ($\sigma_c = 120$ MPa) of Charles & Watts (1980) has one of the most curved envelopes over the stress range for which test results were obtained. It is a simple matter to map equation (2) to the principal stress plane used for equation (3). Fig. 14(a) shows the failure envelopes over the range of σ_3' tested (30–700 kPa) for the parameters given by the Authors (i.e. a, b of 0·14, 0·68 for equation (2) and α, β of 0·34, 0·57 for equation (3)) and they are reasonably similar. Also shown in Fig. 14(a) is a linear

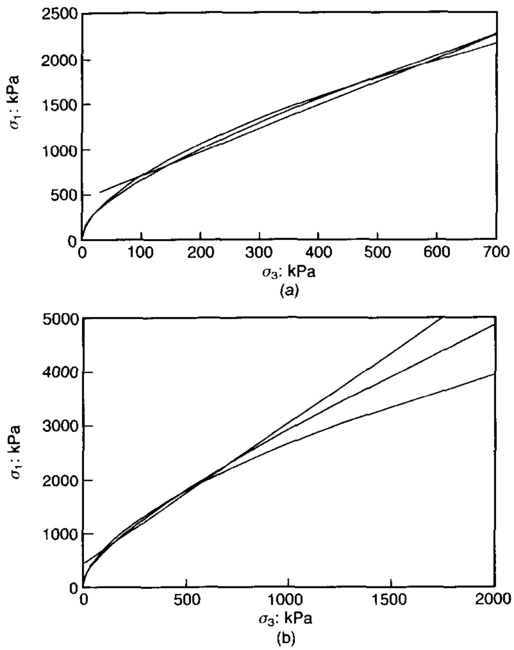


Fig. 14. σ_1 plotted against σ_3 from a, b and α, β for sandstone

failure envelope with $c = 140$ kPa and $\phi = 26°$; there is little to choose between the envelopes over the range for which there is actually test information. Charles & Watts (1980) specifically indicated that such curved failure envelopes were valid over only a limited stress range, and that a suitable linear envelope could also be used. All three criteria rapidly diverge at higher stresses (Fig. 14(b)) and at $\sigma_3' = 2$ MPa the values obtained from equations (2) and (3) differ by 25%. This amply demonstrates the contention that any failure envelope should be extrapolated only with extreme caution. Extrapolation is often reasonable with linear failure envelopes as most geotechnical engineers have some understanding of sensible parameters, but I contend this is not the case with either equation (2) or equation (3).

Using the information in Table 2 I have approximately replotted Fig. 11 by using the values for σ_c, α and β given for each rockfill to determine σ_{1f}'/σ_c for four values of σ_3', being the minimum and maximum values given in the table and the arithmetic and geometric means of these. The values of σ_{1f}'/σ_c determined for the minimum and maximum σ_3' should be very close to the maximum and minimum data for that rockfill as shown in Fig. 11 as these presumably are actual test pressures. The two means were used to provide intermediate data for analysis. The derived data are plotted in Fig. 15 in the same form as Fig. 11. The data in Table 2 were assumed correct and Figs 10 and 11 were ignored as there are discrepancies between them and also between them and Table 2.

Adopting the failure criterion given in equation (3) single upper ($\alpha = 2{\cdot}24$, $\beta = 0{\cdot}77$) and lower ($\alpha = 0{\cdot}92$, $\beta = 0{\cdot}77$) bounds can be obtained as shown in Fig. 15. Compared with the bounds given in Fig. 11 these bounds are wide at the extremes of the data, but I believe that the relative closeness of the bounds at the extremes in Fig. 11 is due to the lack of data and should not be relied on. These bounds are simpler and, I believe, preferable to those given in the Paper.

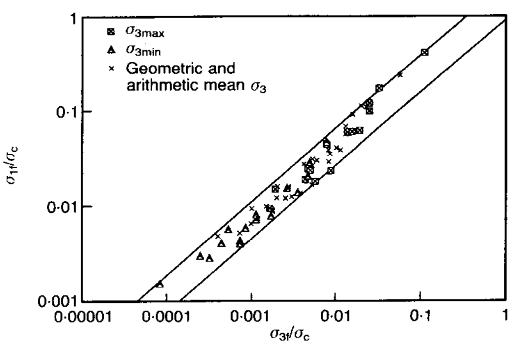
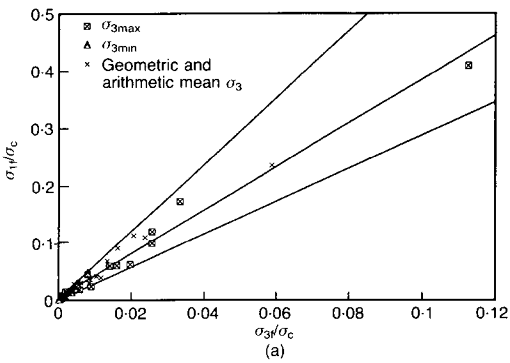


Fig. 15. Table 2 replotted

I believe that linear scales have considerable merit, and use them to present two simple views of the derived data: normalized strength data and principal stress data.

Normalized data are shown in Fig. 16; all data are shown in Fig. 16(a) and those for $\sigma_{3f}'/\sigma_c < 0{\cdot}01$ in Fig. 16(b). Adequate bounds for these data are a lower bound given by $\phi = 29°$ and an upper bound with $c' = \sigma_c/1500$ (this can be set to zero with little impact) and $\phi = 45°$. The sparse data for normalized stresses greater than 0·04 (two points) should not be used to tie the bounds closely but these data fall well within the bounds determined from lower normalized stresses. Not only are these bounds simpler than those given by the Authors but they also accord well with my experience. Further, a bilinear predictor of the mean normalized strength has been obtained by least squares regression and is shown between the two bounds in Fig. 16. The parameters are zero cohesion and $\phi = 45°$ for $\sigma_3'/\sigma_c < 0{\cdot}0026$, and $c = \sigma_c/750$ and $\phi = 36°$ for higher normalized stresses. On my data this predictor has $r^2 \approx 0{\cdot}98$. For weak rock rockfill the high normalized stress range predictor would generally be applicable and, in fact, is equivalent to a best estimate of the strength of such rockfill of simply $\phi = 36°$.



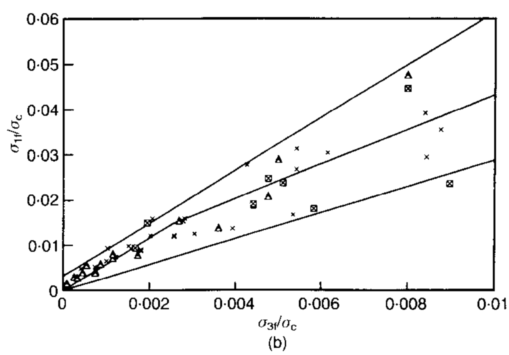


Fig. 16. Table 2 replotted on linear scale: (a) all data; (b) low stress data

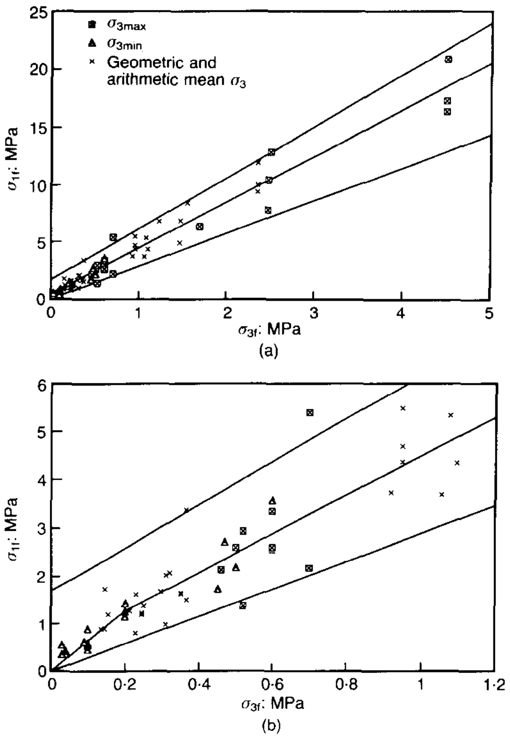


Fig. 17. Principal stress plot: (a) all data; (b) low stress data

Principal stress data and reasonable bounds are plotted in Fig. 17; all data are shown in Fig. 17(a) and those for $\sigma_{3f}' < 1{\cdot}2$ MPa in Fig. 17(b). A lower bound to all the data is simply $\phi = 29°$ and an upper bound of $c' = 400$ kPa and $\phi = 39°$. These bounds are reasonably tight and very simple. Again a bilinear predictor of the mean strength is shown between the two bounds in Fig. 17. The parameters are zero cohesion and $\phi = 45°$ for $\sigma_3' < 200$ kPa, and $c = 117$ kPa and $\phi = 37°$ for higher stresses. On my data this predictor has $r^2 \approx 0{\cdot}96$.

In summary, reasonable lower and upper bounds and mean strengths have been presented; all are simple, in accord with the data and, I believe, more useful than those presented by the Authors.

I attempted to regenerate much of the data in Fig. 11 by using the information in Table 2. There are some differences between these data points and those of the Authors, and thus in the results of statistical analysis on them.

Authors' reply

Many of the comments and the alternative analysis presented by Mr Mostyn fine tune the work in the Paper. Using the data in the Paper, an alternative analysis based on a slightly different statistical scheme is proposed. Although this analysis may throw some additional light on the subject, it does not alter the original failure representations and concepts proposed in the Paper.

In the Paper it is stated (p. 48) that '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'. Fig. 5 is also based on the average of three tests. Data points in Figs 3, 4, 6 and 8 are not averages of several tests but are of selected individual specimens. The standard deviation of the deformation modulus was less than 8%, but the standard deviation of the friction angle was less than 3% for all the data points given in Figs 5 and 12.

There is indeed a slight discrepancy between some of the values given in Table 3 and those shown in Fig. 10. The only significant error in the Paper was the value of the coefficient a for the high stress range (lower bound). The value of 1·35 given in the Paper was corrected to 0·75 in the corrigendum published in *Géotechnique* (1993) **43,** No. 3, p. 506, which is close to the value of 0·76 proposed by Mr Mostyn. The differences in the remaining values in Table 3 as calculated in the Paper and by Mr Mostyn are also small. In an engineering sense, further refining of these numbers would therefore seem to be an unnecessary task.

The values in Table 4 are consistent with Fig. 11. These upper and lower bound coefficients have been determined using a regression analysis with $r^2 > 0{\cdot}97$. Therefore, there is no discrepancy between the values given in Table 3 and Fig. 10, and Table 4 and Fig. 11. Slight deviations of the values proposed by Mr Mostyn are perhaps due to the variation of regression analysis used, as well as due to the inevitable errors which can be made in measuring off values from small printed graphs plotted on log scales.

There are no errors in the plotting of the original data points in Figs 10 and 11. Table 2 summarizes most tests that have been conducted in the past. Figs 10 and 11 contain a small number of additional data which have been taken from the stress–strain and failure plots published previously for the same rockfill types, but using smaller sample size ratios. The values of uniaxial compressive strength for various rock types given in Table 2 are the mean values of several intact specimens. More important, the range of σ_3' given in Table 2 is applicable for most samples tested in the triaxial apparatus. A few specimens were tested outside the general range of σ_3' given in Table 2, both at very low normal stresses and at very high stress levels. The data points in Figs

10 and 11 are based on the work of Marsal (1973), Marachi (1969), Charles & Watts (1980) and Marachi, Chan & Steed (1972), as well as on their own work. Details of the original stress–strain and failure responses, including the test results associated with very low and very high confining pressures, are given in these references. Furthermore, due to the congestion and overlapping of many data points at some parts of the plots, a few data points were omitted from Figs 10 and 11 for clarity. In summary, Table 2 was intended to provide a general guide to the type of tests conducted by various researchers, whereas Figs 10 and 11 were constructed using the original stress–strain curves and/or associated failure data available from published literature. It is unreasonable to assume that each data point in Figs 10 and 11 can be obtained using Table 2 only. This practice is not implied in the Paper.

We agree fully that the bounds proposed in the Paper overestimate the shear strength of very weak types of rockfill. This is indeed reflected in Fig. 13, where the low-strength rockfill envelope given by Leps (1970) has been lowered. Furthermore, Fig. 7 indicates that even low-grade slate falls significantly below the strength envelopes of many other rockfill types. Therefore, the coefficients given in the Paper should not be applied directly to predict the behaviour of weak rockfills, without accurate determination of their coefficients (a, b) and (α, β) by further triaxial tests. Mr Mostyn mentions rockfill having uniaxial strengths σ_c of 15 MPa, and very soft materials used in the coal-mining industry. Our proposals were based on significantly higher compressive strengths, in conjunction with many other types of rockfill material used in the past. As indicated in Table 2, the lowest uniaxial compressive strength considered was 40 MPa for intact standard size specimens. At much lower values of σ_c the findings highlighted in the Paper may not be directly applicable. The essence of the Paper is concerned with the type of failure envelope proposed. For a given rockfill to be used in practice, its exact coefficients (a, b) and (α, β) should be determined from large triaxial tests, whereas the coefficients in the Paper should serve as guidelines.

The data in Fig. 13 are taken from Leps (1970), except for the greywacke which we tested ourselves. We do not agree that friction angles of common rockfill materials are often less than 30°, except in the case of very weak soft rocks, which fall below the lower envelopes proposed by us as well as Leps (1970). For such weak types of rockfill, the coefficients and bounds given in the Paper may overestimate their shear strength. We strongly disagree that a linear failure envelope is better in design than a non-linear envelope as proposed in the Paper. Many large-scale laboratory tests conducted since the late 1960s have shown that rockfill failure is characterized by a decreasing ϕ' with increasing effective normal stress. By assuming a linear envelope, the rockfill is modelled similar to the behaviour of a rock joint or a bedding plane which often satisfy a constant c, ϕ condition during shear. In our opinion, rockfill should be modelled more as a heavily fractured rock medium where realistic non-linear failure envelopes have been given by Hoek & Brown (1980). De Mello (1977) discussed the implications of non-linear envelopes for rockfill. We have improved the non-linear concept proposed by De Mello (1977) by deriving normalized non-linear envelopes using the uniaxial compressive strength of the material. However, we agree that a linear envelope is often preferred for convenience, and it may be so that many geotechnical practitioners have a good feeling for such simplified c, ϕ parameters. Nevertheless, with modern numerical and computer facilities, replacing linear parameters with non-linear parameters can only improve design according to the current state-of-the-art.

We recognize the difficulties of testing rockfill particles significantly smaller than their prototype counterparts in the field. To overcome the effects of particle size, we have explained the relevance of parallel gradations (Fig. 2), realistic sample size ratios (Table 1), and the importance of similar laboratory and field porosities among other factors highlighted in Table 2. Although it is impossible to test prototype rockfill (1000–2000 mm) in the laboratory, one should be able to simulate prototype behaviour by establishing the appropriate similitude criteria (Fumagalli, 1969) applicable for triaxial testing as followed in this study.

Corrigenda

Equation (1) *should read*

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

Equation (2) should read

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

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