PREDICTION OF EFFECTIVE STRENGTH PARAMETERS FROM RESIDUAL STRENGTH TESTS

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SUMMARY

The concept of partial safety factors in geotechnical design is reviewed. Sets of comparative data from both simple and more time consuming shear tests are presented and the relevance of partial safety factors to slope stability and bearing capacity discussed. Methods for the prediction of effective strength parameters for silts and clays are suggested.

INTRODUCTION

Partial safety factors provide a means of acknowledging the greater reliability attributable to the frictional component of shear strength when compared to the lesser confidence with which the cohesion intercept can be established. This leads to the suggestion that simplified methods for estimating the strengths of soils can produce results that are close to correct values, provided the frictional characteristics are reliably established. Means for prediction of effective strength parameters for silts and clays, without the need for time consuming triaxial testing, are put forward in view of the present use of other methods which are based on strength determined in terms of total stresses (shear vane and Scala penetrometer).

PARTIAL SAFETY FACTORS

The concept of partial safety factors in geotechnical design has been advocated principally by Meyerhof (1970, 1982, 1984), as a consistent approach to give more uniform margins of safety for different situations. The current loadings code (NZS 4203:1984), utilises partial safety factors for structural members (by separation of the total safety factor into a load factor and a strength reduction factor) in the strength method of design. However for soil strength, the code recommends that the total safety factor be derived from the combination of a load factor and a partial safety factor of 1.8 on the ultimate capacity of the soil. The background to this is discussed by Taylor (1976). Meyerhof, however, emphasises that soils may be treated similarly to other construction materials, in that strength reduction factors may be assigned to them. With addition of appropriate load factors, the customary total safety factors for geotechnical purposes are then obtained. Meyerhof's primary recommendations for slopes (other than dams) and shallow foundations are shown in Table 1. (Modification factors are required for earth retaining structures, excavations and deep foundations. Also it is apparent that the load factors shown are somewhat less than required by NZS 4203, hence by implication use of the listed strength reduction factors in this country will be somewhat conservative. These aspects require further evaluation but are not the object of the following discussion.)

Table 1. Meyerhof's values of minimum partial safety factors.

Category	Item	Load factor	Strength factor
Loads	Dead	1.25 (0.85)	
	Live etc.	1.5	
	Water pressures	1.25 (0.85)	
Shear strength	Cohesion (stability, earth pressures)		0.65
	Cohesion (foundations)		0.5
	Friction		0.8

Note: load factors in parentheses apply to dead loads and water pressures when their effect is beneficial.

The effective shear strength equation, using strength reduction factors becomes:

$$s_f = f_c c' + (p - f_u u) f_0 \tan \text{Ø}' \qquad (1)$$

where s_f is the factored shear strength, f_c and f_0 are strength reduction factors applied to effective cohesion (c') and friction (Ø') and f_u is a load factor applied to the pore water pressure, u.

The significance of the unfavourable reduction factor accorded to cohesion, when compared with the value for frictional strength is of particular note. The uncertainty of the 'cohesion' value (being simply an intercept found by extrapolating test data) has commonly been reported and is most clearly demonstrated by setting confidence limits on the test parameters. For normally consolidated, lightly overconsolidated and recompacted soils, the 99% confidence statistics will not infrequently indicate a negative cohesion, leading to recommendations for c' = 0 in design of so called 'cohesive' soils. Doubts regarding the long term reliance on effective cohesion even in some intact moderatedly overconsolidated clays have been expressed by Skempton (1977). Accordingly, determination of the frictional characteristics of saturated soils might be considered most appropriate for establishing reliable soil strength parameters from either triaxial tests or simplified approaches.

PEAK SHEAR STRENGTH CORRELATIONS

In a previous paper (Salt, 1984) the comparative simplicity and reliability of establishing residual strength parameters (in terms of effective stresses) was discussed. The ring shear apparatus (Bromhead, 1979) allows tests to be completed most rapidly but both ring shear and multiple reversal direct shear methods provide similar results with sandy or silty soils. When saturated, these materials exhibit very low or zero residual cohesion with linear strength envelopes. In finer grained clayey soils, where the friction angle is less than about 20 degrees, curvature of the strength envelope (concave downward) may be significant, in which case friction plus a 'pseudo' cohesion may be adopted over a specified normal stress range.

However if testing is carried out at sufficiently low normal stresses it appears that all saturated soils exhibit no true residual cohesion.

In order to extend the applicability of residual strength testing from its usual function (determination of landslide mechanisms and appropriate type and extent of remedial measures), correlation of residual strengths with recompacted properties was suggested in the earlier paper. As considerably more data are now available, these are presented.

Figure 1a shows the ratios of the peak shear stress to residual shear stress of recompacted soils, plotted as a function of the effective normal stress on the failure plane. This method of comparison has been used in order to show specific shear strengths rather than some assessed combination of cohesion plus friction.

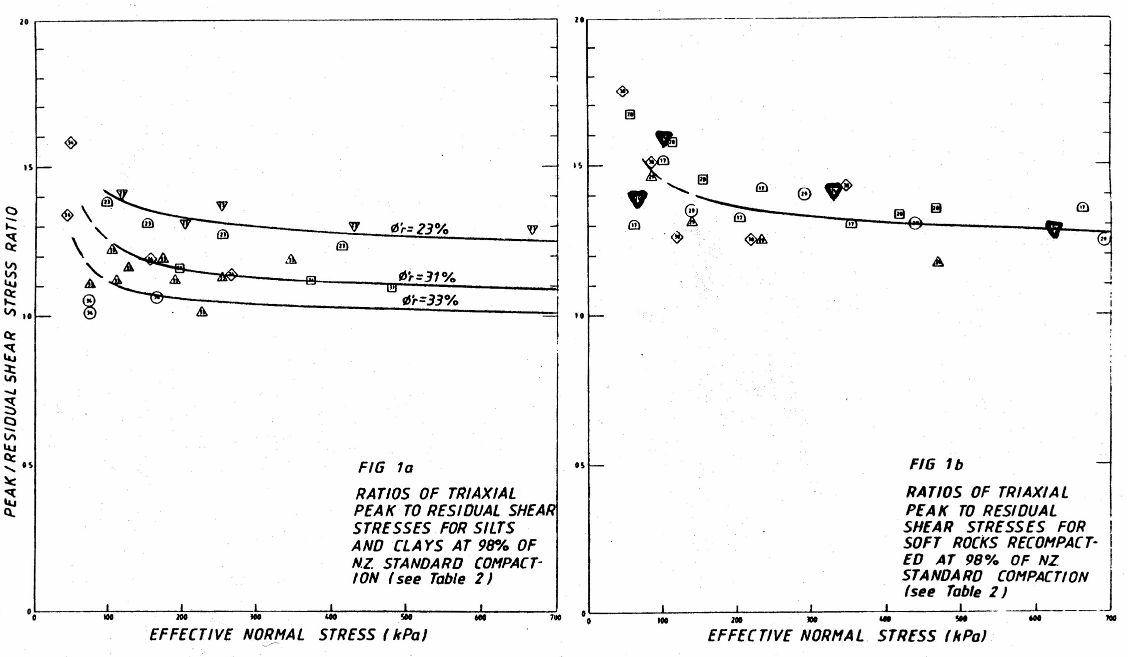

FIG 1a
RATIOS OF TRIAXIAL PEAK TO RESIDUAL SHEAR STRESSES FOR SILTS AND CLAYS AT 98% OF NZ STANDARD COMPACTION (see Table 2)

FIG 1b
RATIOS OF TRIAXIAL PEAK TO RESIDUAL SHEAR STRESSES FOR SOFT ROCKS RECOMPACTED AT 98% OF NZ STANDARD COMPACTION (see Table 2)

A variety of NZ soils was tested and corresponding residual friction angles are shown. All samples were compacted to between 97 and 100% of NZ Standard (Proctor) compaction, back-pressure saturated and the triaxial peak strengths were determined using the maximum stress ratio failure criterion (Bjerrum, 1961). In situ, these soils were all normally consolidated or lightly overconsolidated and were readily disaggregated for recompaction. These data could be crudely approximated by a single curve, but soils with different residual strengths may be considered independently and this results in the correlations as shown.

Further data are shown in Figure 1b. These were all heavily overconsolidated soils, i.e. soft rocks, (papa) which had been broken down to pass through a 19 mm sieve, rather than being reduced to their component grains. In this case a single correlation appears to fit the data sufficiently well for practical purposes. Details of the soil characteristics are shown in Table 2.

Table 2. Characteristics of recompacted fine grained materials.

Sample No.	Material	LL	PI	USC	Residual friction (degrees)
		Soils (Fig. 1a)			
2088	sandy silt	41	NP	ML	23
2069	clayey silt	45	18	ML-CL	23
7340	silt	26	5	ML	31
7358	silt	26	5	ML	31
153	sandy silt	44	25	CL	33
8440	clayey silt	27	9	CL	34
215	clayey silt	29	10	CL	34
		Recompacted soft rocks (Fig. 1b)			
2105	clayey silt	50	27	CL-CH	17
2717	silt-clay	56	33	CH	24
2818	sandy silt	47	26	CL	28
2718	clayey silt	45	24	CL	29
2047	clayey silt	52	23	MH-CH	29
2819	sandy silt	54	31	CH	30

Much of the scatter of the data in Fig. 1 is attributable to variations within an individual series of test samples. For a linear Mohr envelope in a c'-∅' soil, each series (shown by different symbols) should lie on a curve which steepens at low normal stresses.

Results come from three independent laboratories but most are from the one source. Variations resulting from the same soil tested at different laboratories are likely to be considerable from apparatus and preparation techniques alone. Additional sources of variation are selection of representative samples, determination of optimum moisture content, and which side of optimum the sample is compacted. Compaction factors also varied between 97 and 100%.

Although more data are required it is suggested that sufficient correlation exists to allow preliminary estimates of peak compacted strengths from residual strength data. For soils with residual angles less than 23 degrees no data are available as yet.

STRENGTH - COMPACTION RELATIONSHIPS

For soils for which Proctor compaction is applicable, little information has been found regarding the relationship of peak triaxial strength to any parameter. It appears reasonable to expect that for densities close to the Proctor maximum, strengths might be linearly related to the compaction factor.

One of the samples in Table 2 (No. 215) was compacted at densities ranging from 89 to 99% of the maximum at optimum moisture using NZ Standard

Compaction. Again to avoid the need to consider cohesion and friction independently, the ratio (R) of peak triaxial to residual shear stress appears the most simple parameter to consider. Inspection of the very limited available data showed that for preliminary estimates the strengths of homogeneous normally consolidated or recompacted soils with compaction factors in the range of 89 to 99% might be approximated independently of normal stress, simply by:

$$R_{cf} = 1 + (R_{98}-1) \times (CF/98)^{6} \qquad (1)$$

Where R_{cf} and R_{98} are the peak to residual shear stress ratios at compaction factors (CF) at percentages of CF and 98% respectively. The standard value of 98% has been arbitrarily suggested as this is frequently specified as a practical target (e.g. NRB F1) and is applicable to the data shown in Figure 1.

SUGGESTED SHEAR STRENGTH PREDICTIONS

Determination of effective strength parameters is frequently regarded as so time consuming that codes suggest other methods for design. For example, NZS 4404 suggests that in most cases "it is impracticable to measure the factor of safety of a slope against shear failure". (A precedent basis is suggested. Where possible this is sometimes the best approach, but often precedents do not exist or are unduly conservative.)

The concept of partial safety factors for cohesion and friction minimises the difference between analyses based on either a simple residual shear test (friction only) or more rigorous triaxial data. As no NZ codes focus on safety factors for the stability of slopes other than for dams or retaining structures, the partial safety factors proposed by Meyerhof might be adopted in the interim for all other cases (fill embankments, cut slopes and stabilised or modified natural slopes. For shallow foundations, serviceability criteria (settlements) often govern design rather than bearing capacity and also, safety factors are stipulated in NZS 4203. However, the following simplified approaches are suggested.

Initially, for undisturbed or recompacted soils, the peak strength may be taken conservatively as equal to the parameters determined from residual strength tests. Analysis (giving due regard to local as well as general shear criteria) may then confirm that serviceability or dynamic considerations govern, and no more elaborate testing for the determination of peak strength is required.

If static strength governs (assuming residual conditions), actual strength for silts and clays may be used in conjunction with ratios interpolated from Figure 1, to determine peak shear strengths (s_{cf}) at the appropriate compaction factor for several normal stresses (p') in the relevant range of field stresses, using simply:

$$s_{cf} = R_{cf} \times p' \times \tan Ø'_{r} \qquad (2)$$

From these pairs of shear and normal stresses the estimated Mohr envelope may be plotted and effective strength cohesion and friction parameters read in the usual way. The values so determined may be used either for preliminary design or as a means of deciding whether significantly higher strengths are likely to be justified in the event that more time consuming triaxial testing is carried out.

An example showing typical residual strength test results and inferred triaxial strength is shown in Figure 2. Note that this shows the full inferred cohesion assuming the sample remains well compacted, and at low normal stresses a conservative partial safety factor for effective cohesion must be applied. If softening can take place, ie the material is near-surface and can become saturated or unconfined, zero cohesion should be adopted.

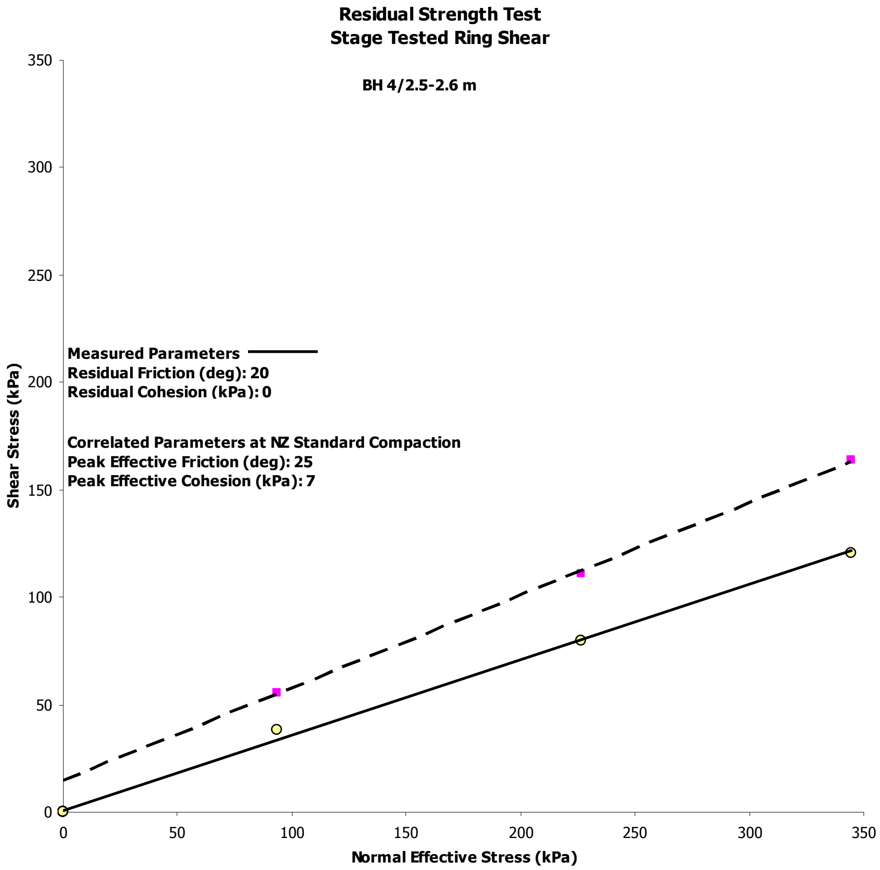

Figure 2. Example test data for residual and peak effective strength parameters.

CONCLUDING REMARKS

1. The use of partial safety factors rather than a total safety factor in geotechnical design allows a consistent approach, giving more uniform margins of safety for different situations. Where appropriate partial factors are applied to cohesion and friction, due regard may be given to the confidence placed in each parameter. Future revisions of existing codes would provide for more realistic geotechnical design if the partial safety factor approach (Meyerhof, 1984) is incorporated optionally.

2. As no NZ codes currently focus on the question of appropriate safety factors for slope stability (other than dams), Meyerhof's recommendations (Equation 1) might be adopted. Shallow foundations designed using the load factors required by NZS 4203, with Meyerhof's strength reduction factors

(Table 1) should produce conservative total safety factors. The application of residual strength parameters in conjunction with appropriate partial safety factors will often demonstrate that criteria other than static strength govern design.

3. Simple procedures for predicting effective strength parameters of silts and clays in either their undisturbed or recompacted states are presented. Collection of further data is in progress to refine the interim correlations. However, even at this stage it is suggested that the effective strength parameters predicted by these relatively simple techniques, should be regarded as preferable for long term design, to commonly used devices such as the shear vane and Scala penetrometer which produce strength estimates which are limited to much less definitive (undrained strength) concepts.

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