FUNDAMENTAL AND PRACTICAL ISSUES CONCERNING UNDRAINED STRENGTH ANISOTROPY

QUESTIONS FONDAMENTALES ET PRATIQUES CONCERNANT L'ANISOTROPIE DES RESISTANCES NON DRAINEES

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INTRODUCTION

This discussion concerns the undrained strength anisotropy of soft sedimentary clays having a one–dimensional strain history (K_0 consolidation and rebound). The changes in strength with variations in the major principal stress direction (δ angle to vertical direction) during monotonic shearing can be divided into two components. An *inherent* anisotropy arises from the "soil structure" developed at the microlevel (preferred particle orientations and interparticle forces) and also at the macrolevel for certain soils such as varved glacial lake deposits. The second component, called ***initial shear stress*** anisotropy, occurs whenever undrained shearing starts from a $K_0 \neq 1$ condition, as first predicted by Brinch–Hansen and Gibson (1949). The combined effect of both components on the undrained strength ratio for low OCR clays sheared in compression ($C, \delta = 0°$) and extension ($E, \delta = 90°$) can be expressed as

$$\frac{q_f(C)}{\sigma'_{vc}} = \frac{[K_0 + (1-K_0)A_f]\ \sin\varphi'}{1 + (2A_f-1)\ \sin\varphi'} \qquad \text{(1a)}$$

with $A_f = (\Delta u - \Delta\sigma_h)/(\Delta\sigma_v - \Delta\sigma_h)$ since $\Delta\sigma_h = \Delta\sigma_3$

$$\frac{q_f(E)}{\sigma'_{vc}} = \frac{[1 - (1-K_0)A_f]\ \sin\varphi'}{1 + (2A_f-1)\ \sin\varphi'} \qquad \text{(1b)}$$

with $A_f = (\Delta u - \Delta\sigma_v)/(\Delta\sigma_h - \Delta\sigma_v)$ since $\Delta\sigma_v = \Delta\sigma_3$

where $q_f = 0.5(\sigma_1-\sigma_3)_f$ and σ'_{vc} = vertical consolidation stress.

Assuming *isotropic* material properties and $K_0 = 1 - \sin\varphi'$, Eq. 1 gives $K_s = q_f(E)/q_f(C) = 0.167/0.333 = 0.50$ for $\sin\varphi' = 0.50$ and $A_f = 1.00$ and $K_s = 0.233/0.300 = 0.78$ for $\sin\varphi' = 0.40$ and $A_f = 0.75$. As subsequently shown, these values are typical of the undrained strength anisotropy of lean and plastic OCR = 1 clays, respectively, even though $\sin\varphi'$ and A_f also vary due to the effects of inherent anisotropy.

EXPERIMENTAL DATA ON UNDRAINED STRENGTH ANISOTROPY

Measurement of undrained strength anisotropy must rely on laboratory K_0 consolidated–undrained (CK_0U) shear tests since: existing in situ tests cannot shear soil in different directions; and laboratory UU type tests ignore ***initial shear stress*** anisotropy. Fig. 1 plots peak undrained strength ratios from CK_0U triaxial compression/extension (q_f/σ'_{vc}) and Geonor direct simple shear (τ_h/σ'_{vc}) tests run on a variety of normally consolidated (NC) cohesive soils. Excluding the varved clay, the data show: $q_f/\sigma'_{vc} = 0.32 \pm 0.03$ in TC and having no trend with I_p; generally much lower DSS strengths that tend to decrease with lower plasticity; and even smaller ratios for shear in TE, especially at low I_p. Thus anisotropy is generally most important in low plasticity soils, especially if also sensitive. The varved clay (solid symbols) represents a special case since horizontal (DSS) shearing

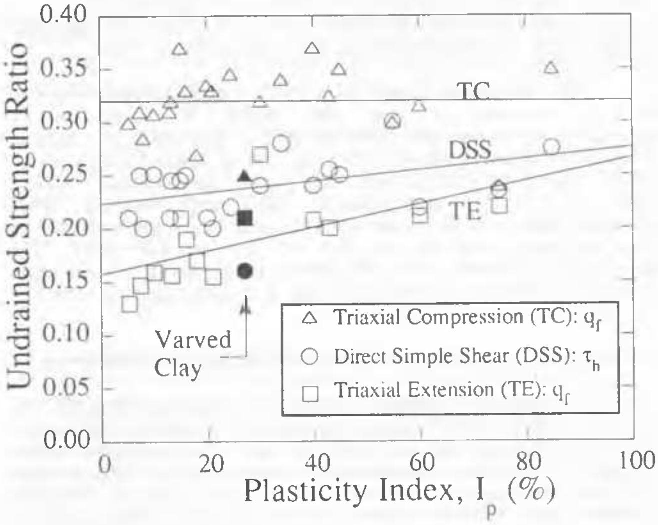

Fig. 1 Undrained Strength Ratios from CK_0U Tests on Normally Consolidated Clays and Silts (from Ladd 1991).

Clay	I_p (%)	I_L	Test Program	Triaxial Compression				Sym.
				S	m	No. Tests	r^2	
Natural BBC	28 ±5	0.6 ±0.2	SHANSEP TC/TE	0.280	0.68	24	0.990	B(S)
			Recomp. TC/TE	0.298	0.675	23	0.921	B(R)
AGS (N.J.)	43	0.6	SHANSEP TC/TE	0.325	0.78	11		AGS
James Bay B6	13 ±4	1.9 ±0.5	Recomp. TC/TE	0.45	0.86	10		B6

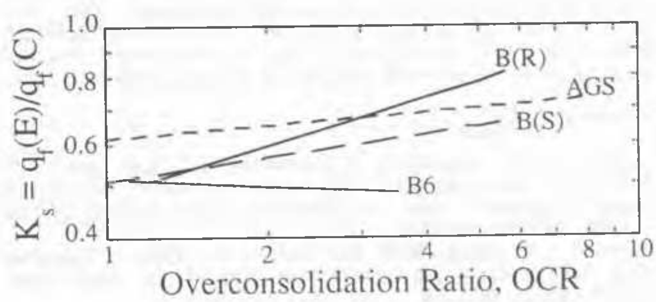

Fig. 2 Undrained Strength Anisotropy vs. OCR from Recompression and SHANSEP CK_0U Triaxial Tests.

gives the lowest strength.

Overconsolidated clays also exhibit significant undrained strength anisotropy as illustrated by the data in Fig. 2. It plots K_s vs. OCR from CK_0U triaxial tests on three natural clays. The reconsolidation technique used to test overconsolidated natural BBC has a large effect on anisotropy at high OCR. Recompression to the in situ OCR shows less anisotropy than SHANSEP, which first normally consolidates the soil and then unloads to the in situ OCR. Note: Fig. 2 shows values of S and m from linear regression on the TC tests using the SHANSEP relationship $q_f/\sigma'_{vc} = S(OCR)^m$.

Modeling of undrained strength anisotropy (i.e., determining how q_f varies with δ) from results of CK_0U triaxial (TX) and direct simple shear (DSS) tests faces two problems.

1) The triaxial tests involve a change in the σ_2 condition and measure strengths that are too low for plane strain (PS) conditions. Ladd (1991) quotes:

- At $\delta = 0°$, $q_f(TC)/q_f(PSC) = 0.92 \pm 0.05$ (several clays).
- At $\delta = 90°$, $q_f(TE)/q_f(PSE) = 0.82 \pm 0.02$ (only four clays)

2) The state of stress in DSS tests is unknown. Judgement suggests that τ_h lies between q_f and $\tau_{ff} = q_f \cos \varphi'$ and $\delta = 45 \pm 15°$.

The Directional Shear Cell (DSC) has the unique ability to vary δ between 0° and 90° under PS conditions by application of normal and shear stresses to four sides of a cubical sample contrained between two rigid end plattens (Arthur et al. 1981). Fig. 3 plots data from DSC and "conventional" CK_0U tests run on NC resedimented BBC, with Bishop's (1966) equation being used to model strength anisotropy for both sets of data (assuming $\tau_h = q_f$ at $\delta = 45°$ for the DSS test). The dashed line is about 15 ± 5% lower than the "true" strength. Research is needed to determine if this is typical since the DSC is not ready for use in practice.

UNDRAINED STRENGTHS FOR SLOPE STABILITY ANALYSES

This section discusses selection of anisotropic strengths for use in a "total stress" analysis involving a saturated foundation clay using Spencer's method of slices and a sophisticated search routine for the critical non–circular (irregular shaped) shear surface such as available in the UTEXAS3 slope stability program (Wright, 1991). Since "$\varphi = 0$," $c = c_u$ = the undrained shear strength. Even neglecting the adverse influence of progressive failure (e.g., Section 4.9 of Ladd, 1991) and the beneficial influence of 3–D "end effects" (e.g., Azzouz et al. 1983), which tend to cancel each other, there are two important issues: how to define c_u and what is the relationship between the direction of σ_{1f} (δ angle) and the inclination of the shear surface (α angle to horizontal plane)? Some engineers may select $c_u = q_f$ and $\delta = 45° - \alpha$, believing this to be theoretically consistent with a "$\varphi = 0$" analysis. For the data in Fig. 3, this assumption gives lines A and B in Fig. 4 for the DSC and TX&DSS data, respectively. In contrast, suppose that the critical shear surface closely approximates the actual most likely failure surface through the clay foundation. If true, the writer would then select $c_u = \tau_{ff} = q_f \cos \varphi'$ and $\delta = \theta - \alpha$, where $\theta = 45 + \varphi'/2$ = angle between failure plane and σ_1 plane. For the DSC data in Fig. 3, failure planes were observed at $\theta \approx 62°$, which is consistent with the measured $\varphi' \approx 34°$. This assumption gives line C in Fig. 4. The different assumptions have practical significance. For α = 45° to −30°, the average c_u/σ'_{vc} = 0.255, 0.225 and 0.185 for lines A, B and C, respectively. If line C were correct, then line A is unsafe by almost 40% and line B by about 20%.

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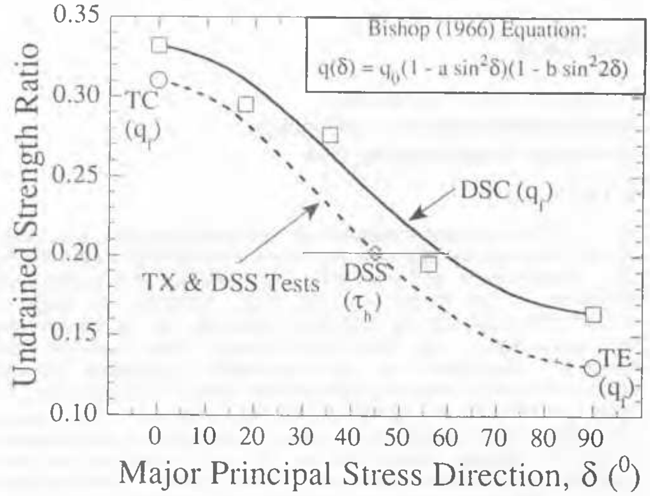

Fig. 3 Undrained Strength Ratio vs. Stress Direction from CK_0U Tests on NC Resedimented BBC.

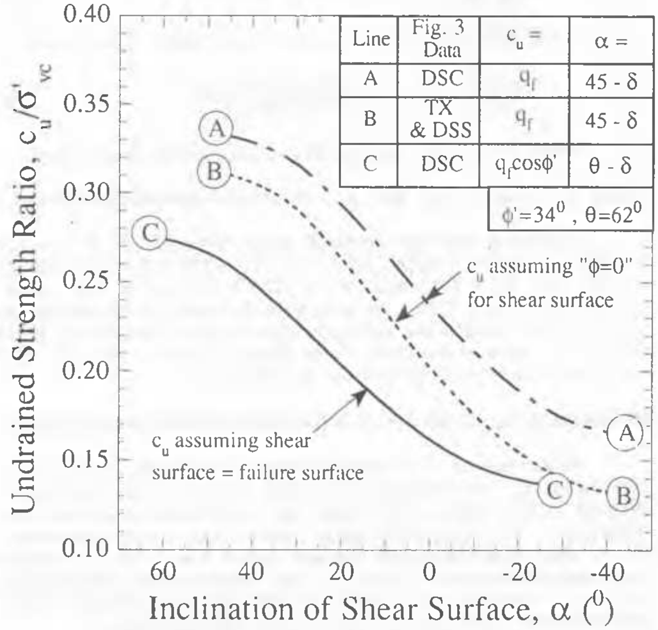

Fig. 4 Selection of Design Strengths from Experimental Data in Fig. 3.