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EFFECTS OF MODE OF SHEAR ON YIELD SHEAR STRENGTH OF CONTRACTIVE SANDY SOILS

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ABSTRACT

The authors collected a database of 386 laboratory triaxial compression, direct simple shear, rotational shear, and triaxial extension test results to examine the yield strength ratio concept used to evaluate liquefaction triggering in sloping ground. Generally, triaxial compression exhibits the highest yield strength ratios, triaxial extension yields the lowest ratios, and direct simple shear/rotational shear shows intermediate responses. And for a given mode of shear, mobilized yield strength ratios vary inversely with state parameter. Lastly, these laboratory data envelope the yield strength ratios obtained from back-analyses of static liquefaction flow failure case histories.

Keywords: Liquefaction, yield shear strength, laboratory testing, mode of shear

INTRODUCTION

Liquefaction-induced slope failures can be manifest in many forms, but are primarily categorized as lateral spreads and liquefaction flow failures. Lateral spreads often occur on level to mildly sloping ground (i.e., ground with little or no static shear stress) during seismic shaking when the combined static and seismic shear stresses exceed the available soil resistance. Because lateral spreads occur on level to mildly sloping ground, the soil is subjected to a stress path that corresponds closely to direct simple shear (DSS; Figure 1a).

In contrast, liquefaction flow failures can occur below sloping ground (i.e., ground subjected to a static shear stress) under static, quasi-static, or dynamic conditions. And soils involved in liquefaction flow failures (and also liquefaction-induced slumps and bearing capacity failures) commonly are subjected to multiple modes of shear, including triaxial compression (TxC), DSS, and triaxial extension (TxE; Figure 1b). Olson and Stark (2003a) back-analyzed numerous static liquefaction case histories to estimate yield strength ratios mobilized during these failures. Based on both the static and dynamic slope failures, they suggested that the liquefiable materials in many of these failures were, on average, subjected approximately to DSS conditions. For clarity, the yield shear strength, s_u(yield), is defined as the peak shear strength available during undrained loading of a saturated, contractive, sandy soil (Terzaghi et al. 1996). And the yield strength ratio, $s_u(\text{yield})/\sigma'_{vo}$, is the yield shear strength normalized by the prefailure vertical effective stress, σ'_{vo}.

Olson and Stark (2003b) described a database of triaxial compression tests used to evaluate the yield strength ratio concepts and correlations. While these data generally support the yield strength ratio concept for saturated contractive sands, TxC test results generally plot above the upper range of case history data. Olson and Stark (2003b) anticipated that TxE and DSS tests would provide results at

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lower and intermediate ranges of responses, respectively. Additionally, they expected that the combined laboratory data would envelope the values back-calculated from the case histories.

To test these hypotheses, we supplemented the TxC database described by Olson and Stark (2003b) with available additional TxC, DSS and rotational shear (RS), and TxE tests on saturated, contractive sandy soils. The objectives of this study were to: (1) evaluate yield strength ratio concepts using TxC, DSS/RS, and TxE test data; (2) compare laboratory measured data to strength ratios back-calculated from field case histories; and (3) explore potential reasons for observed differences in response for different modes of shear.

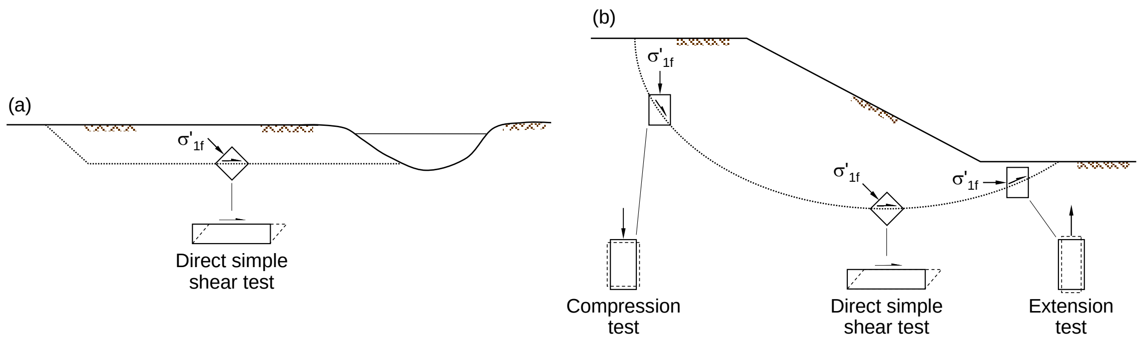

Figure 1: Relevance of laboratory shear tests to modes of shear on potential surfaces of sliding in the field. (a) Lateral spreading failure of level to mildly sloping ground. (b) Liquefaction flow failure (or slump) of sloping ground.

LABORATORY DATABASE FOR SATURATED, CONTRACTIVE SANDY SOILS

Table 1 summarizes the laboratory test data collected for this study, performed almost exclusively on reconstituted specimens. The table contains grain characteristics, maximum and minimum void ratios, and references for each of the sands. For each test, we collected the following data when available: end-of-consolidation void ratio (e_c), relative density (D_{rc}), and state parameter (ψ); major and minor principal effective stresses after consolidation (σ'_{1c} and σ'_{3c}, respectively); and yield shear stress conditions [s_u(yield) and Δu_{yield}], among other stress-strain values. State parameter, ψ, is defined as the difference between the void ratio at the end of consolidation, e_c, at a given mean effective confining stress and the void ratio at steady state, e_{ss}, for the same mean effective stress (Been and Jefferies 1985). For this study, we collected 275 individual tests results. In addition to these 275 tests, we included 111 test results from Olson (2001) for a total of 386 test data. Note that we only included data from Olson (2001) for cases where the complete stress-strain curve was available. Space limitations preclude us from including the entire database in this paper.

Classifications for stress-strain response

There are three typical stress-strain responses of saturated sandy soils to undrained monotonic loading. Figure 2 illustrates these three responses, using the labels suggested by Castro (1969). Purely contractive (Type A) response consists of the following: upon initial loading, the shear resistance of the sand increases until reaching its yield shear strength; upon reaching the yield shear strength, the sand structure collapses and undergoes strain-softening to a minimum shear resistance. In this case, the minimum shear resistance is also the steady state strength, as defined by Poulos (1981). Type A response occurs in very loose sandy soils and is of primary interest in liquefaction evaluation.

Contractive then dilative (Type B) response is similar to Type A response, except that after reaching a minimum shear resistance, the sand undergoes strain-hardening. Alarcon-Guzman et al. (1988) termed this minimum resistance the quasi-steady state shear strength. We divided Type B response into two subcategories: B1 and B2. Type B1 response exhibits a minimum shear resistance over a range of shear strain (arbitrarily defined as greater than 3%). Type B2 response exhibits a minimum shear resistance for a smaller window of shear strain (i.e., less than 3%) before strain-hardening. Type B2 response often occurs when the initial state of a specimen is near its steady state line. Like Type A response, Type B response (particularly B1) can occur in loose to medium dense sands and is relevant in liquefaction evaluation.

Dilative (Type C) response consists of strain-hardening behavior and reaches neither a yield nor a minimum shear resistance. As with Type B response, the sand often continues to strain-harden at large strain and may not reach a steady state within the displacement limitations of conventional testing equipment. Type C response occurs in dense sand and generally indicates that the soil is not susceptible to flow failure without significant void ratio redistribution or other changes to the soil.

Table 1: Laboratory database of sandy soils used to examine yield strength ratio concepts

Sand	Grain characteristics			Void ratio data				Reference
	D_{50} (mm)	FC (%)	C_U	e_{max}	test[1]	e_{min}	test[2]	
Erksak 300/0.7 (E330)	0.33	1	1.8	0.753	--	0.469	--	Been et al. (1991)
Monterey #0 (M0S)	0.35	0	1.7	0.86	A	0.53	a	Riemer (1992)
Fraser River 1 (FRS1)	0.30	0	--	1.00	B	0.68	b	Thomas (1992)
Nevada (NS)	0.25	8	--	0.887	--	0.511	--	Arulmoli et al. (1992)
Likan (LS)	0.24	0	1.9	1.239	--	0.756	--	Lee (1995)
Fort McMurray (FMS)	0.15	10	--	0.992	--	0.542	--	Konrad & Saint-Laurent (1995)
Kawagishi-cho (KCS)	0.32	1	--	1.054	--	0.793	--	Yoshimine (1996)
Hostun RF (HRF)	0.38	1	2.0	1.041	--	0.648	--	Doanh et al. (1997); Doanh & Ibraim (2000); Zhang (1997)
Unimin 1 & 3 (US1&3)	0.87	0	2.0	1.037	C	0.655	c	
Unimin 2 (US2)	0.87	0	2.0	1.037	C	0.655	c	
Syncrude (SYN)	0.17	12	2.9	1.04	C	0.61	c	Wride & Robertson (1999)
Fraser River 2 (FR2)	0.30	0	1.8	0.926	C	0.605	c	Vaid et al. (2001)
Silica #8 (SS8)	0.06	78	3.2	1.66	--	0.85	--	Wang & Sassa (2002)
Merriespruit tailings (FC = 0); (MTS0)	0.13	0	1.7	1.221	B	0.738	b	Fourie & Tshabalala (2005)
Merriespruit tailings (FC = 20); (MTS20)	0.12	18	4.3	1.326	B	0.696	b	
Merriespruit tailings (FC = 30); (MTS30)	0.10	30	24	1.331	B	0.577	b	
Merriespruit tailings (FC = 60); (MTS60)	0.06	60	25	1.827	B	0.655	b	
Toyoura (TS)	0.20	0	--	0.977	--	0.597	--	Yoshimine (1996)

[1]Tests to determine e_{max}: (A) dry pluviation; (B) ASTM D2049; (C) ASTM D4254
[2]Tests to determine e_{min}: (a) modified Japanese method; (b) ASTM D2049; (c) ASTM D4254

Yield strength ratios from laboratory data
For consolidated-undrained TxC and TxE tests, yield strength ratio is defined as:

$$\frac{s_u(yield)}{\sigma'_{vo}} \equiv \frac{(\sigma_1 - \sigma_3)_y}{2\sigma'_{1c}} \qquad (1)$$

where $(\sigma_1 - \sigma_3)_y$ is the peak deviator stress prior to strain softening and σ'_{1c} is the vertical effective stress at the end of consolidation (i.e., the cell pressure for isotropically consolidated specimens). For DSS and RS tests, the yield strength ratio is defined as:

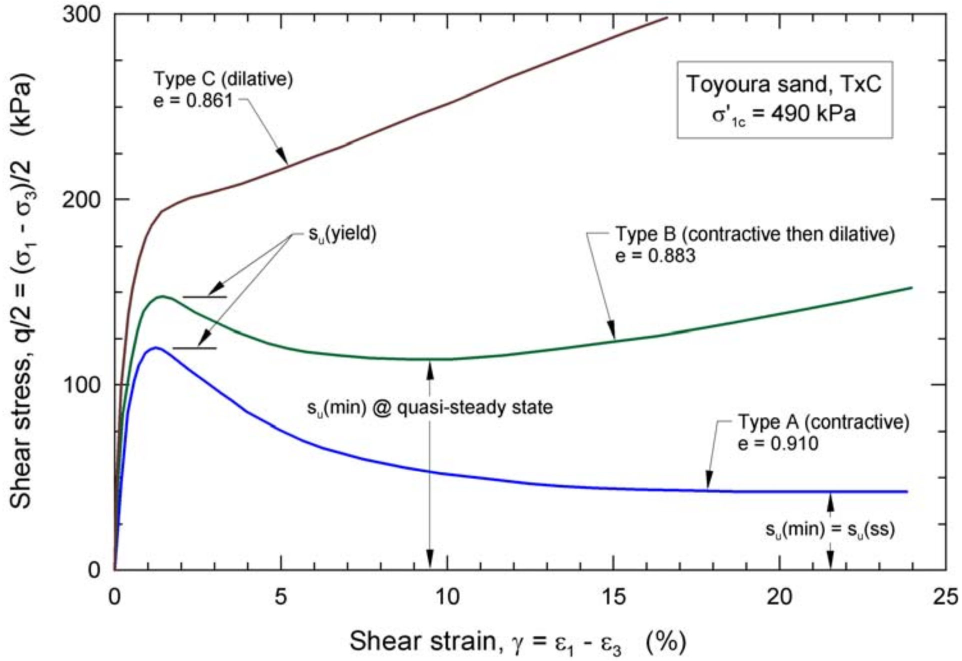

Figure 1: Type A, Type B, and Type C undrained stress-strain response of sandy soils (data from Verdugo 1992)

$$\frac{s_u(yield)}{\sigma'_{vo}} \equiv \frac{\tau_y}{\sigma'_{vc}} \qquad (2)$$

where τ_y is the shear stress at yield and σ'_{vc} is the vertical effective stress at the end of consolidation. We note that the DSS and RS specimens were consolidated under laterally constrained conditions (i.e., anisotropic principal stresses) while the TxC and TxE tests reported here involve equal all-around consolidation stresses (i.e., isotropic principal stresses). We also note that the yield shear strength in Eq. (1) is a deviatoric shear stress, while in Eq. (2) the shear stress at yield is not a deviatoric shear stress; it is a measured shear stress. While these factors complicate direct comparisons, these strength ratio definitions are widely accepted.

Figures 3 and 4 present yield strength ratio and yield friction angle data (in q-σ'_{mean} space), respectively, for the TxC tests. Yield strength ratios in TxC range from about 0.18 to 0.43 and yield friction angles (i.e., effective stress friction angles mobilized at yield conditions) range from about 16 to 26.5 degrees. Yield friction angles can be useful in analyses involving stress paths or triggering mechanisms where the effective stresses are known (or reasonably estimated), such as the case of a rising watertable. Yield conditions in q-σ'_{mean} space are converted to yield friction angles as follows:

$$\phi_y = \sin^{-1}\left[\frac{3\left(q/\sigma'_{mean}\right)_y}{6+\left(q/\sigma'_{mean}\right)_y}\right] \qquad \text{(Triaxial compression)} \qquad (3a)$$

$$\phi_y = \sin^{-1}\left[\frac{3\left(q/\sigma'_{mean}\right)_y}{6-\left(q/\sigma'_{mean}\right)_y}\right] \qquad \text{(Triaxial extension)} \qquad (3b)$$

where $q_y = (\sigma'_1 - \sigma'_3)_y$ and $(\sigma'_{mean})_y = (\sigma'_1 + \sigma'_2 + \sigma'_3)_y/3$. (Note that $\sigma'_2 = \sigma'_3$ in triaxial compression and that $\sigma'_1 = \sigma'_2$ in triaxial extension.)

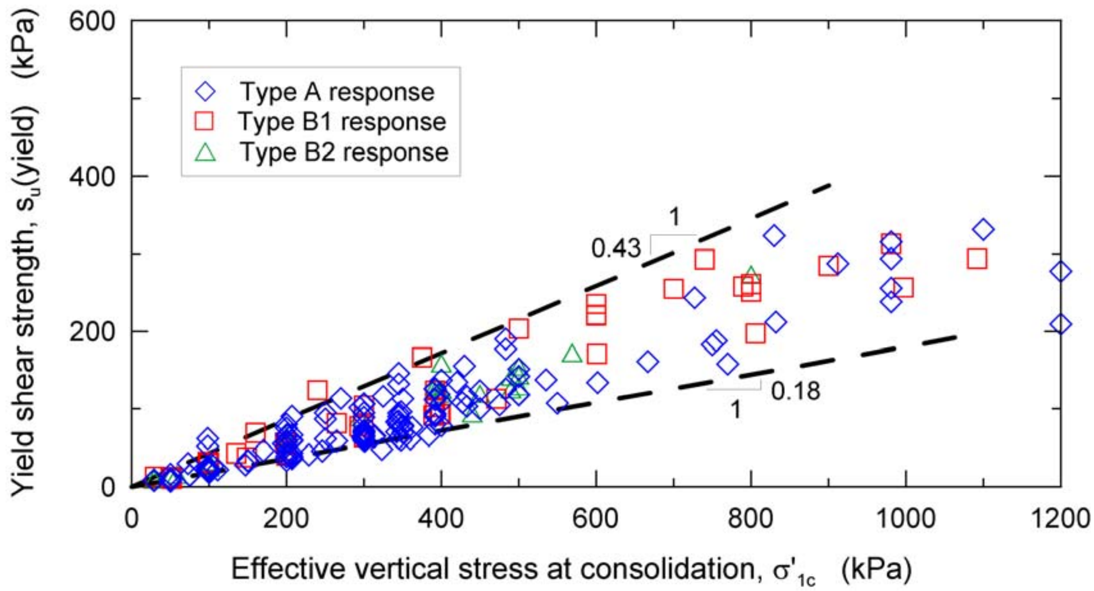

Figure 3: Summary of yield strength ratio data for triaxial compression

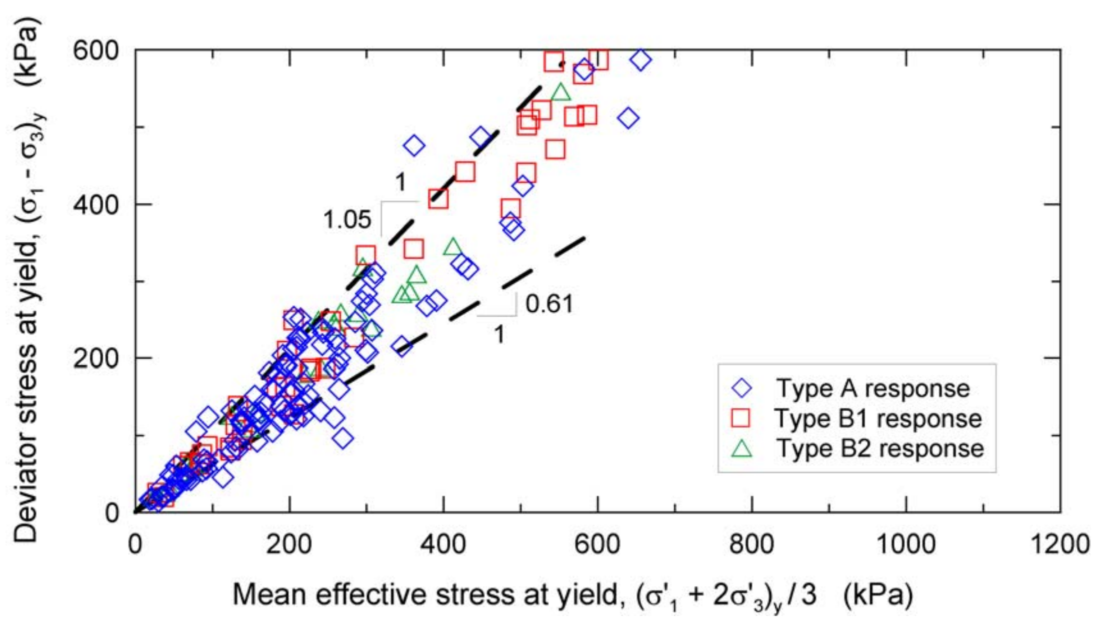

Figure 4: Summary of yield friction angle data (in q-σ'mean space) for triaxial compression

Similarly, Figures 5 and 6 present yield strength ratio and yield friction angle data, respectively, for the DSS and RS tests. Yield strength ratios in DSS/RS range from about 0.13 to 0.29 and yield friction angles range from about 12 to 23 degrees. Here, yield friction angles can be determined conventionally from τ_y-$\sigma'_{n,y}$ space, where $\sigma'_{n,y}$ is the effective normal stress at yield. Lastly, Figures 7 and 8 present yield strength ratio and yield friction angle data, respectively, for the TxE tests. Yield strength ratios in TxE range from approximately 0.11 to 0.24 and yield friction angles range from about 11 to 23 degrees. Yield friction angles for TxE are computed using Eq (3b).

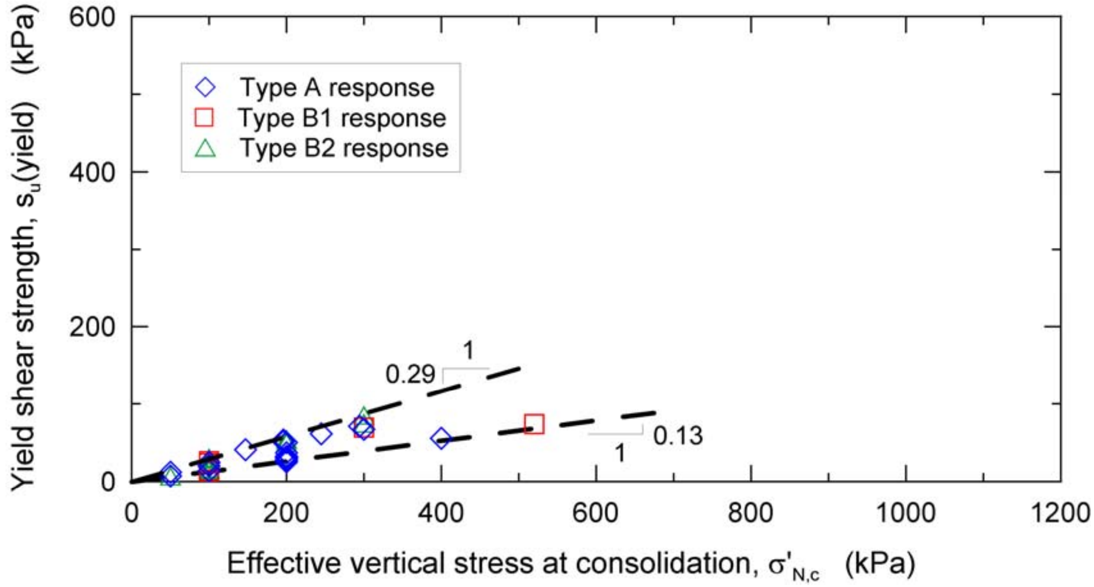

Figure 5: Summary of yield strength ratio data for direct simple shear and rotational shear

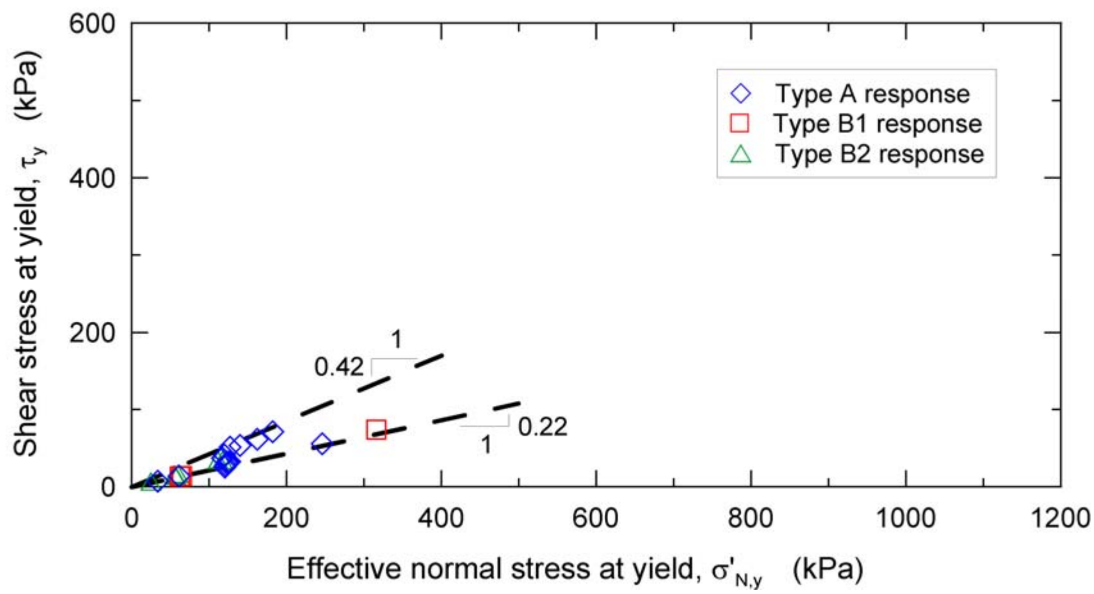

Figure 6: Summary of yield friction angle data for direct simple shear and rotational shear

Table 2 summarizes the TxC, DSS/RS, and TxE results. These data confirm that yield strength ratios and yield friction angles are strongly influenced by the mode of shear. Specifically, TxC generally exhibits the highest values of yield strength ratio, TxE generally provides the lowest yield strength ratios, and DSS/RS generally provides intermediate results. Furthermore, careful examination of available state parameter data indicate that yield strength ratio generally increases as state parameter decreases, regardless of the mode of shear.

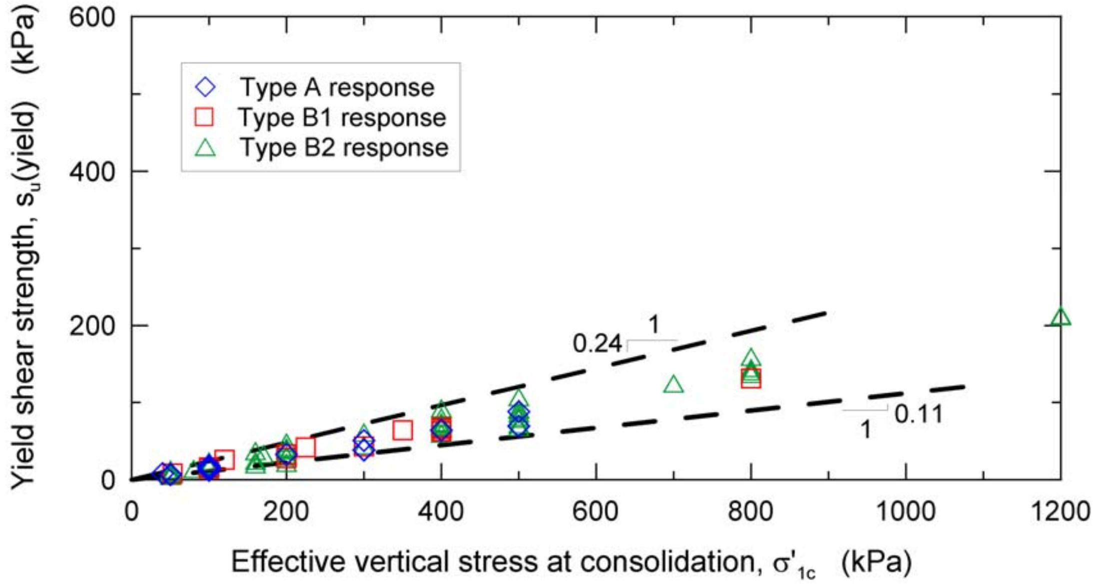

Figure 7: Summary of yield strength ratio data for triaxial extension

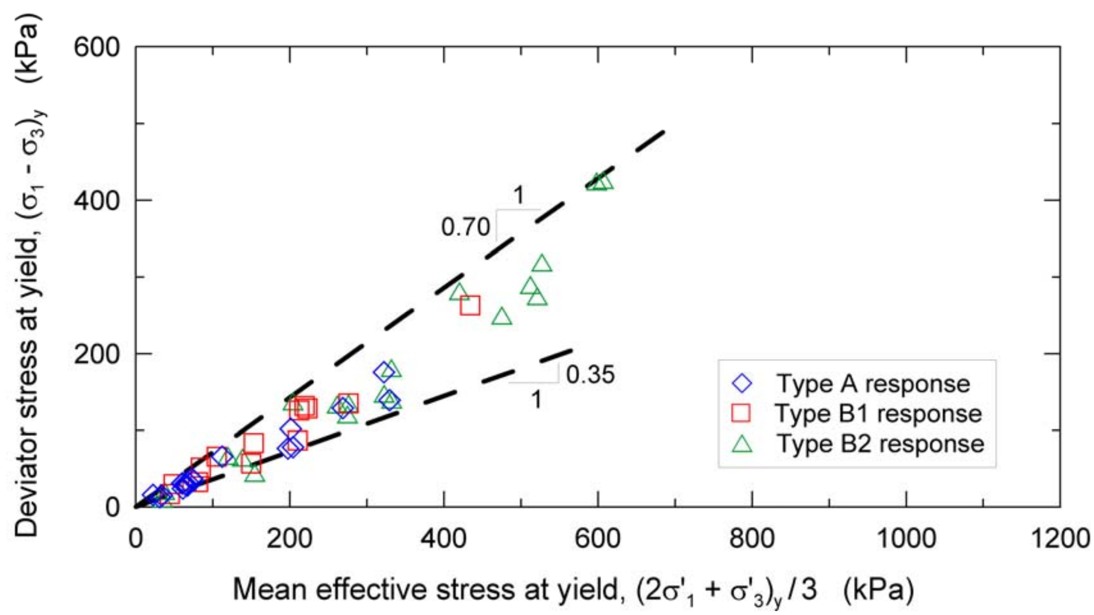

Figure 8: Summary of yield friction angle data (in q-σ'mean space) for triaxial extension

Table 2: Summary of yield strength ratios and yield friction angles measured in laboratory tests and back-calculated from field case histories

Parameter	Laboratory test results			Field case histories
	Mode of shear			
	Triaxial compression	Direct simple shear & rotational shear	Triaxial extension	
$s_u(yield)/\sigma'_{vo}$	0.18 - 0.43	0.13 - 0.29	0.11 - 0.24	0.23 - 0.31
Yield friction angle	16 - 26.5°	12 - 23°	11 - 23°	~ 13 - 17°

COMPARISON OF FIELD AND LABORATORY DATA

Olson and Stark (2003a) analyzed thirty static, deformation, dynamic, and seismically-induced liquefaction flow failure case histories to evaluate shear stress ratios mobilized at liquefaction triggering. Of these case histories, only the static flow failures provided mobilized shear stress ratios immediately prior to failure that are approximately equal to the yield strength ratio (see Olson and Stark, 2003a for further discussion). Olson and Stark (2003a) also suggested that three deformation and dynamically-induced flow failures exhibited mobilized shear stress ratios immediately prior to failure that are close to their yield strength ratios. In total, these eight liquefaction flow failures exhibited back-calculated yield strength ratios that ranged from approximately 0.23 to 0.31.

Figure 8 and Table 2 compare the yield strength ratios back-calculated from these eight field case histories as well as the seismically-induced flow failures to the values obtained from the laboratory database. We included the seismically-induced flow failures for comparison only. As illustrated in Figure 8, the laboratory data envelope the case histories. Although the field case histories involved multiple modes of shear, on average the back-calculated yield strength ratios correspond fairly closely to yield strength ratios measured in DSS tests. We also note that most of the cases fall above the upper bound for TxE, suggesting that a TxE mode of shear did not dominate any of the static flow failure case histories.

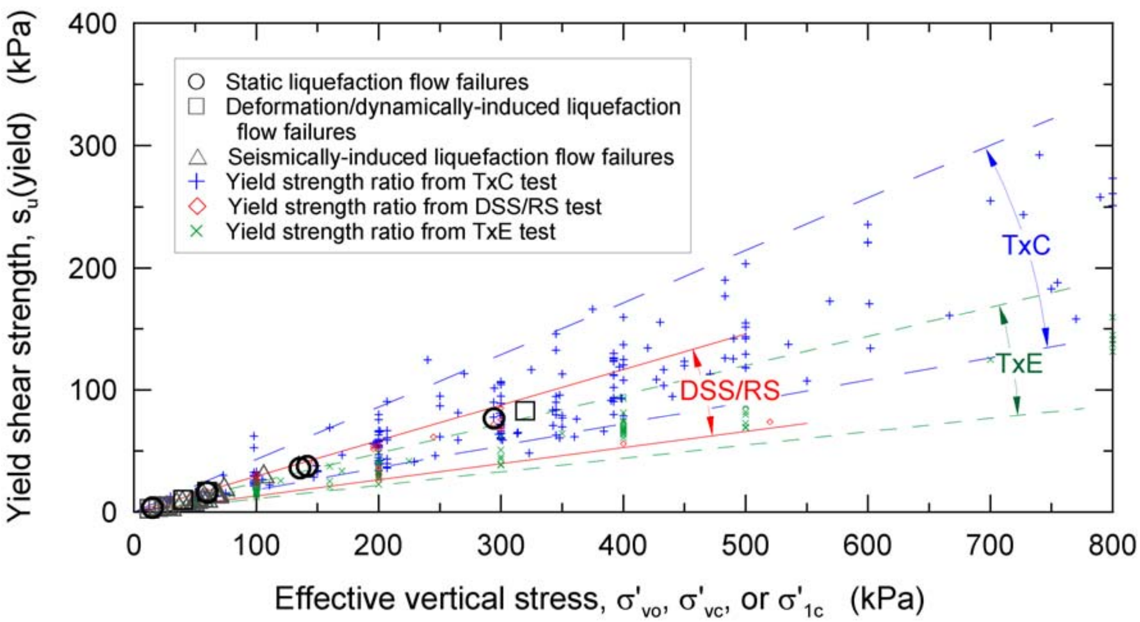

Figure 8: Comparison of yield strength ratios for field and laboratory data. Flow failure case histories from Olson and Stark (2003a).

INTERPRETATION AND DISCUSSION

Investigators have long recognized that the mode of shear strongly affects the yield shear strength (which is mobilized at small to intermediate shear strain) of individual, contractive sandy soils (e.g., Hanzawa 1980; Vaid et al. 1990; Uthayakumar and Vaid 1998; Yoshimine et al. 1999). Data presented here for a large number of sandy soils confirm these findings for individual sands. When considering a large number of sands, differences in yield strength ratio for a given mode of shear occur as a result of differences in state parameter, grain size distribution, grain characteristics, mineralogy, compressibility, consolidation shear stresses, among other factors. For a given sand, though, yield strength ratios differ for various modes of shear because of several factors, but primarily due to inherent and induced anisotropy (Casagrande and Carrillo 1944).

Casagrande and Carrillo (1944) defined inherent anisotropy as differences in fabric and the resulting particle contacts that develop during deposition. For example, during controlled horizontal pluviation non-spherical sand grains come to rest with their longitudinal axes primarily oriented in a nearly horizontal direction (Oda 1972; Uthayakumar 1996). Moist tamping, on the other hand, produces a more isotropic fabric by utilizing capillary effects to keep sand particles apart during placement (Vaid et al. 1999). Additionally, Oda (1972) found that particle contact normals initially are oriented in a direction parallel to the direction of deposition. As a result, a sand deposited horizontally in water generally will exhibit greater resistance to compression that shears across the bedding (and takes advantage of the soil fabric and particle interlocking) than to shearing that occurs parallel to the bedding (as in DSS) or that disrupts the soil fabric (as in TxE). It follows that different depositional environments (both natural and controlled) produce different soil fabrics, and lead to different stress-strain response at small to intermediate shear strains when subjected to different modes of shear.

Induced anisotropy occurs as elongated sand grains become aligned in a direction perpendicular to the major principal stress as a sand mass is sheared (Casagrande and Carrillo 1944; Oda et al. 1985). Additionally, contact normals become increasing oriented along the direction of the major principal stress during shear (Oda et al. 1985). These observations indicate that a major change occurs in the sand fabric when deposits are sheared in directions that are not parallel to the direction of deposition. These fabric changes affect stress-strain behavior at small and intermediate shear strains.

The differences exhibited in yield strength ratios for different modes of shear have practical significance. In some circumstances, potential failure mechanisms or slip surfaces may be dominated by a particular mode of shear. In these cases, the guidelines supplied in Table 2 and Figures 3 through 8 provide a designer with ranges of yield strength ratios and yield friction angles that can be anticipated for TxC, DSS/RS, and TxE modes of shear. Designers can also use these ranges to validate specific laboratory results for a given sand.

CONCLUSIONS

The laboratory data that we collected in this study confirm that mode of shear plays a significant role in the undrained stress-strain response of saturated, contractive sandy soils and that the yield and strength ratio concept described by Olson and Stark (2003a) is valid. Furthermore, the laboratory measured strength ratios agree with the values back-calculated from liquefaction flow failure case histories. Specifically, we conclude the following from our analysis and interpretation.

1. Yield shear strengths and strength ratios, which are mobilized at small to intermediate shear strains, are strongly influenced by soil fabric as qualitatively expressed by inherent and induced anisotropy.
2. For a given value of state parameter, triaxial compression tests generally exhibit the highest values of yield strength ratio, triaxial extension yields the lowest values, and simple/rotation shear yields intermediate results. We observed a similar trend for yield friction angles.
3. Yield strength ratios measured in laboratory tests envelope the values back-calculated from static liquefaction flow failure case histories. These field case histories generally fall above the triaxial extension results, suggesting that an extensional mode of shear did not dominate failure in these cases.

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