7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Stability of Embankments in Potentially Liquefiable Soils: A Complement to Seed's Approach

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ABSTRACT

An important factor in evaluating the stability of hydraulic fills against flow sliding is the undrained steady state strength mobilized in the field. This paper proposes an empirical relationship among the undrained strength back-calculated from fills which failed by flow sliding, equivalent clean sand normalized blow count values, and soil specific parameters from steady state laboratory testing. It is shown that S_{uo}, which is a reference value of steady-state strength at maximum void ratio, is an important soil parameter. The proposed method offers an explanation for the performance of many artificial sand islands in the Beaufort Sea, indicates the extreme sensitivity of S_{uo} to soil type and the usefulness of S_{uo} for assessing the potential strength loss of soils for use in safety assessments of existing hydraulic fills.

INTRODUCTION

The problem of assessing the safety of loose deposits of freshly deposited sand masses subjected to shear stresses such as in embankments is complex and consists of two separate issues: (1) recognizing the triggering event(s) and (2) evaluating the average undrained strength mobilized in the field during flow failure.

The first issue, related to triggering of strength loss, is by far the most complex. Furthermore, it must be emphasized that shear failure must occur in undrained conditions. Only soils that tend to decrease in volume during shear, i.e. contractive soils, can suffer the loss of shearing resistance that results in large flow slides when the driving shear stress is considerably larger than the minimum undrained strength of the soil mass. Considering that undrained conditions prevail, possible triggering mechanisms are, for example, (i) rapid static loading loading (fills with steep slopes), (ii) earthquake loading, (iii) foundation movements leading to undrained creep in the sand fill, (iv) a combination of two or several of these individual mechanisms.

The second issue is discussed in details in this paper. In view of the above discussion, it is considered that the stability of soils that are loose enough to present a substantial risk for flow failures can be directly evaluated using the average undrained strength that is mobilized in the field assuming that strength loss has been triggered by the relevant mechanism or a combination thereof. As stated by Seed (1987), it may be adequate and economically advantageous to simply ensure the stability of the embankment against major sliding after liquefaction has occurred, at least for cases where large deformations can be tolerated. Seed (1987) proposed an approach for estimating the undrained shear strength during flow failure based on field case-histories. The average undrained strength, referred to as the residual strength by Seed, is obtained from a relationship between some in situ soil characteristics, such as standard penetration resistance, and back-calculated strength from case-histories.

It is the objective of this paper to demonstrate that Seed's approach can be improved by including a soil specific parameter describing its behaviour during undrained shear.

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SUMMARY OF CASE STUDIES ON RESIDUAL STRENGTHS

The original results of the analysis of twelve earth structures are presented in Figure 1 in terms of residual strength and equivalent clean sand blow count SPT $(N1)_{60}$. The undrained strengths were back-calculated using limit equilibrium analyses for the final geometry of the slide mass. In order to account for a lower penetration resistance in silty sands, an equivalent clean sand value was used by correcting the SPT blow count value as a function of fines content. The penetration values were also normalized with respect to effective overburden pressure and energy efficiency of the system. As noted by Seed, there is a considerable scatter in the results, possibly reflecting variable soil properties and intrinsic difficulties in the back-analysis of the failures. For instance, the residual strength back-calculated from a limit equilibrium analysis is in fact an average strength that may not fully consist of the lowest possible strength of the liquefied earth mass. Scatter in the results may also be caused by the stress path dependency of undrained strength in liquefied sand, as evidenced by recent laboratory tests by Vaid et al. (1990). Failure in partially drained conditions as suggested by Stark and Mesri (1992) may explain high values of back-calculated residual strength.

Despite the difficulties associated with the analysis of flow failures, it is clear that the relationship proposed by Seed is still widely used in practice, at least as a reasonable guide for evaluating the stability of earth structures after liquefaction has been triggered. Wroth (1984) correctly stated that any successful relationship (empirical) should ideally be:

(a) based on a physical appreciation of why the properties can be expected to be related;

(b) set against a background of theory, however idealized this may be.

A careful examination of Seed's empirical relationship in light of the above points may prove relevant for the practice.

FRAMEWORK FOR THE RELATIONSHIP BETWEEN UNDRAINED STRENGTH, SOIL TYPE AND SPT $(N1)_{60}$

Steady-state lines from laboratory tests

The behaviour of saturated loose cohesionless soils during undrained monotonic triaxial compression loading is characterized by strain-softening, which means that loose sand suffers significant strength loss after reaching a peak strength at relatively small strains. For identical void ratios and same initial mean stress conditions, the undrained strength reaches a unique ultimate value at large strains, at least for CIU and CAU tests (Castro, 1969; Konrad, 1990a). This ultimate strength has been referred to as the critical strength by Casagrande (1936), the steady-state strength by Castro (1969), the residual strength by Seed (1987) and post-liquefaction strength by others.

According to Castro (1969) the steady state line is the relationship between void ratio and minor principal effective stress during steady-state deformation, i.e. at constant shear stress and constant void ratio. The combination of values of e and S_u obtained from isotropically consolidated undrained triaxial tests on reconstituted samples defines the soil's characteristics during flow failure for laboratory conditions and for very loose to loose states (Fig. 2). In general, for these loose states, the relationship between Log S_u and void ratio is linear . However, for denser states, grain crushing may occur during shear and the slope of the steady state line becomes steeper reflecting changes in gradation and grain shape. It is emphasized that for the problem under consideration, the earth structures are in a relatively loose state with mean stresses ranging between 10 and 300 kPa. Consequently, the influence of grain crushing during shear may be considered as negligible.

The position of the steady state line in the void ratio-undrained strength plane can be described by the magnitude of the steady-state strength at a reference void ratio or by the magnitude of void ratio at a reference steady-state strength. Rather than selecting an arbitrary value of either void ratio or steady-state strength, it is proposed to consider the maximum void ratio as the reference void ratio for a given soil since it is an index property which can be determined independently. The steady-state characteristics of a given soil are then fully represented by three parameters: (1) the slope of the steady state line, λ, (2) the maximum void ratio, e_{max}, and (3) the value of the undrained steady-state strength at the reference void ratio, S_{uo}, as depicted on Fig. 2.

The steady state strength is thus obtained as:

$$Log\frac{S_u}{S_{uo}} = \lambda(e_{max} - e) \quad (1)$$

It is emphasized that one of the main advantages of using the proposed steady state characteristics lies in the fact that the reference void ratio is not arbitrarily choosen but has a physical meaning, i.e. it corresponds to the loosest packing of dry soil.

Idealized Field conditions

Steady-state parameters which perfectly reflect steady-state characteristics in the field are identified with an asterisk in Figure 3a. Unfortunately, methods to obtain truly undisturbed samples are very expensive. Consequently, indirect methods based on penetration resistance are widely used in practice to estimate in-situ relative density or void ratio. Studies by Marcuson and Bieganousky (1977) show that the penetration resistance of cohesionless soils is dependent upon many variables, two of which are the relative density and the applied overburden pressure. For a given effective vertical stress, the penetration resistance increases almost linearly with increasing relative density, at least for loose to medium dense sands, as shown in Fig. 3b.

Thus, the relationship between undrained shear strength during flow failure and normalized blow count for the idealized field case can be obtained by combining the relationships shown in Figures 3a and 3b. Because the relationship between Log Su and void ratio is linear and the relationship between relative density (i.e. void ratio) and normalized SPT blow count is also linear, at least for loose to medium dense states ($(N_1)_{60} < 15$), it follows that the expected relationship between logarithm of undrained strength during flow failure and SPT $(N_1)_{60}$ should also be linear and should be expressed as:

$$\text{Log}\frac{S_u^*}{S_{uo}^*} = \chi^* (N_1)_{60} \quad (2)$$

where S_{uo}^* = undrained strength during flow failure for e_{max}^*, χ^* is a parameter reflecting both steady-state characteristics and relationship between density and normalized blow count.

Proposed approach for estimating the undrained strength during flow failure in the field

Actual field conditions are more complex than the idealized case discussed above owing to, for example, soil inhomogeneity within the earth structure and differences in effective stress paths of soil elements along potential failure surfaces. The proposed approach seeks to define a relationship similar to eqn (2) obtained for an idealized field case for the more realistic field cases. First, the actual residual strength, Su^*, should be replaced by the average undrained shear strength mobilized during flow failure as back-calculated by Seed (1987), $\overline{S_u^*}$, however questionable its value may be. Second, since S_{uo}^* can only be determined from truly undisturbed samples, it is proposed to replace S_{uo}^* with S_{uo}, which can be obtained from standard laboratory tests on reconstituted specimens of the most representative soil in the failed zone or in the expected failure zone. Third, representative equivalent clean sand blow counts within the potential failure zone are used to reflect the in situ density state.

With thes approximations, eqn (2) can be modified as:

$$\text{Log}\frac{\overline{S_u^*}}{S_{uo}} = \chi (N_1)_{60} \quad (3)$$

where $\overline{S_u^*}$ is the average undrained shear strength mobilized during flow sliding, S_{uo} is the undrained shear strength at e_{max} inferred from laboratory tests on reconstituted soil specimens and χ is a parameter reflecting both steady-state characteristics of reconstituted specimens and the relationship between density and normalized blow count in the field. As shown on Fig. 4, χ is the the slope of the line passing through Points 1 and 2 and is not equal to χ^* because Point 1 is inferred from data on soil with a different fabric from that in the field.

Specific correlations between relative density and SPT penetration resistance are not available for most flow-sliding case-histories and would not be available for the majority of projects where S_u (field) is required for stability analyses. Hence, some other approach is needed to derive the value of χ for each soil. It is proposed to relate χ solely to λ by means of an empirical correlation obtained from the analysis of case-histories.

Thus, the proposed relationship between $\overline{S_u^*}$ and $(N_1)_{60}$ is given as:

$$\text{Log}\,\frac{\overline{S_u^*}}{S_{uo}} = \chi(\lambda)\,.\,(N_1)_{60} \qquad (4)$$

ANALYSIS OF CASE-HISTORIES

To use eqn (4) to predict $\overline{S_u^*}$, a relationship between χ and λ must be established from case-histories where sufficient field and laboratory data are available. Current Seed (1987) derivative approaches are based on about 20 case-histories involving flow slides or lateral spreads, for which three only include complete laboratory testing programs with data on steady-state lines on reconstituted specimens and magnitude of maximum void ratio of dry soil. These three case-histories are (1) Fort Peck Dam (Marcuson and Krinitzsky, 1976) (2) the Lower San Fernando Dam (Seed et al., 1975; (Marcuson et al., 1990), and (3) the Nerlerk Berm (Mitchell, 1984; Sladen et al. 1985a, b, 87; Been et al., 1987). Two flow-slide events are added to the above three cases: (4) a flow failure in uncompacted beach sands at the upstream face of Tar Island Dyke (Alberta) (Plewes et al. 1989) and (5) a road embankment failure at Asele (Sweden) following submergence during impounding a reservoir ((Ekstrom and Olofsson, 1985; Konrad, 1990a).

Because of uncertainty in input parameters, a sensitivity analysis was used to estimate upper and lower bounds for χ as a function of λ. The results of the analysis are summarized in Table 1.

Structure	Material characteristics from laboratory tests			Field data		χ
	emax	Suo (kPa)	λ	(N1)60	Su (kPa)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Nerlerk	0.94-0.98 0	0002-0.0015	0.044	8-10	3.0 - 5.0	0.33-0.55
Tar Island Dyke	0.98 - 1.03	0.0024-0.024	0.057	8-10	8.0-10	0.25-0.45
Fort Peck	0.97-1.05	0.008 - 0.12	0.087	10-12	12.5-22.5	0.17-0.34
L.S.F. > 50%	0.9-1.0	0.01-.01	0.10	15-17	15.0-25.0	0.13-0.23
Asele	0.8-0.87	0.3-0.7	0.145	8-10	5-7.5	0.085-0.17

Table 1. Summary of case-histories analysis

Figure 5 presents the relationship between an average value of χ and λ for the five case-histories analysed above. These few data points can be fitted to a power relationship as:

$$\chi = 0.0138\,\lambda^{-1.075} \qquad (5)$$

EXAMPLE APPLICATIONS: ARTIFICIAL ISLANDS IN THE BEAUFORT SEA

Hydraulic sand fills placed underwater have been used to support waterline penetrating caissons for hydrocarbon exploration in the Canadian Beaufort Sea since 1972. About 20 artificial sand islands have been constructed. Most have performed successfully but a few have not. The extensive geotechnical data base of these artificial islands reported in the literature allows an illustration of the proposed method. The results of geotechnical investigations on four subsea berms, Uviluk, Nerlerk, Kogyuk, and Alerk, were summarized by Sladen and Hewitt (1989). Flow slide failures were reported at two of these fills, namely Nerlerk and Alerk. The other fills did not fail.

STEP 1 Fill characterization

Sladen and Hewitt (Op.Cit.) have shown that placement technique has an important influence on the in-situ density of hydraulically placed sands. Material placed by the bottom-dumping technique is significantly denser than pipeline-placed material. Analysis of CPT data indicates that ECS $(N1)_{60}$ would be higher than 20 for hopper placed sands while pipeline-discharged materials generally display ECS $(N1)_{60}$ values around 10 in the upper 10 m of the fills.

STEP 2 Laboratory test results

Soil index properties for each site are summarized in Table 2.

Sand	D_{50} (% fines)	e_{max}	λ	S_{uo}, kPa	Reference
Ukalerk	0.35 (2%)	0.82	0.1	6.0	Klohn Leonoff Ltd(1983)
Erksak	0.355 (3-6%)	0.963	0.054	0.005	Been et al. (1987)
Kogyuk	0.36 (5%)	0.866	0.095	2.3	Been and Jefferies (1985)
Nerlerk	0.28 (12%)	0.96	0.044	0.0007	Konrad (1991)

Table 2. Index properties of some Beaufort Sea sands

STEP 3

The value of χ are calculated from eqn (5) for each soil and summarized in Table 3.

Sand	χ (eqn 6)	Su (field) kPa Pipeline placed
Ukalerk	0.16	240
Erksak	0.32	8
Kogyuk	0.17	115
Nerlerk	0.40	5

Table 3. Values of undrained strength during flow failure for pipeline-placed sands

STEP 4

Figure 6 shows the graphical determination of undrained shear strength for each sand. S_u is respectively 240 kPa, 115 kPa, 8 kPa and 5 kPa for sands from Ukalerk, Kogyuk, Erksak and Nerlerk.

Discussion of Results.

Extremely large differences in minimum undrained strength mobilized during potential undrained and unconfined failure are predicted despite fairly close grain size distributions of the Beaufort Sea sands. For instance, Ukalerk sand and Kogyuk sand have minimum undrained strength about one to two orders of magnitude higher than that for Erksak and Nerlerk sands. This may explain why liquefaction did not occur at Kogyuk in the zone placed with the pipeline-discharged method although CPT tip resistance profiles were about the same as those at Nerlerk where flow slides occurred as discussed above. Kogyuk was built with a sand that has, according to the proposed approach, a much larger undrained shear strength for equal ECS $(N1)_{60}$ values, and hence is less susceptible to flow sliding in the field.

The results discussed above indicate that subtle variations in soil characteristics are well encapsulated by S_{uo} and λ, which is not the case for CPT soundings or SPT borings. The proposed approach, which has the merit of combining adequate soil characterization with respect to a given mode of failure (here strain-softening during undrained shear) and field averaged data from case-histories, should provide an improved means of assessing the safety of hydraulically-placed fills.

CONCLUSIONS

When saturated cohesionless fills are placed in a loose to medium dense state, flow failure can be triggered by events such as earthquake loading, foundation deformation, undrained static loading or a combination of these events. As stated by Seed (1987), it may be adequate and economically advantageous to simply ensure the stability of the earth structure against major sliding after strength loss has been triggered rather than to prevent triggering. The geotechnical engineer must therefore assess the undrained strength that would be mobilized in the field during flow failure. This paper suggests that this can be done by using an empirical relationship between the undrained strength back-calculated from field performance studies and equivalent clean sand normalized blow count values, which incorporates soil specific parameters (S_{uo} and χ).

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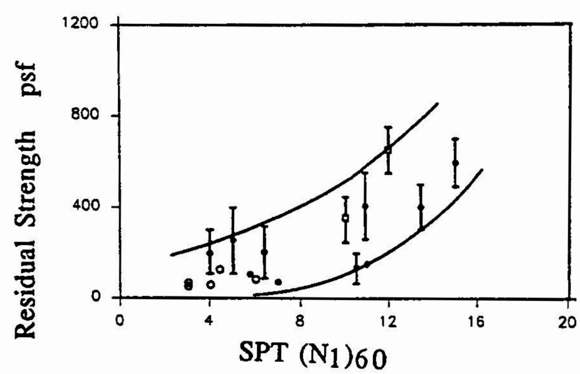

Fig.1 Residual strengths from field data

(After Seed, 1987)

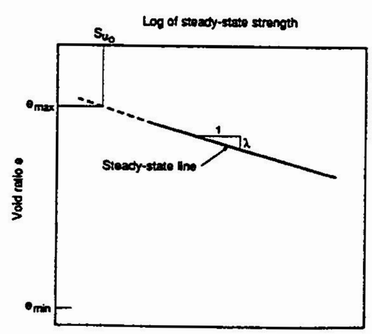

Fig.2 Steady-state characteristics of loose soils

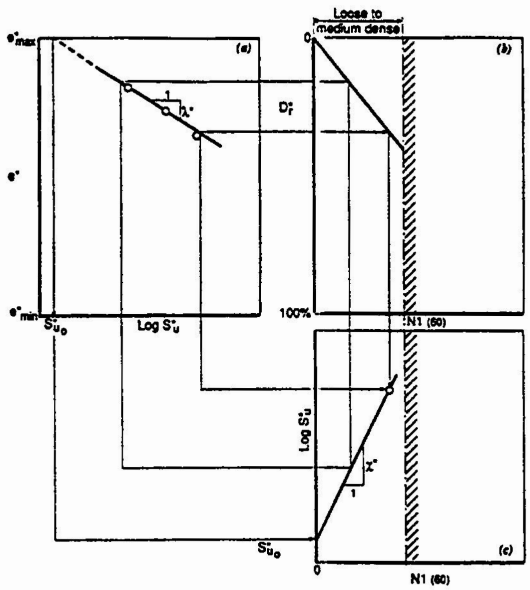

Fig. 3 Relationship between undrained strength during flow failure and (N1)60 for an idealized field case

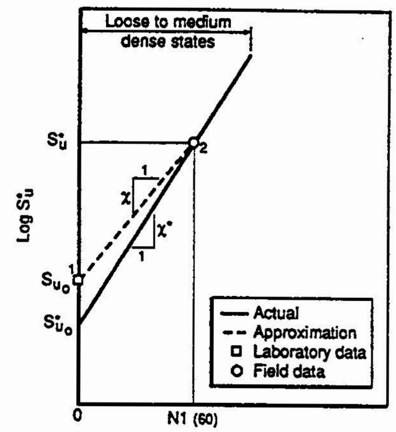

Fig.4 Approximations for the field

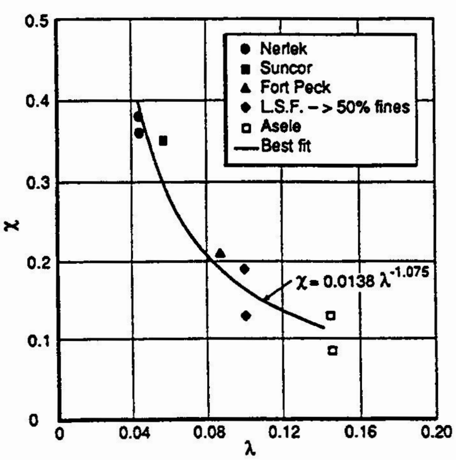

Fig.5 Relationship between χ and λ obtained from field cases and laboratory tests

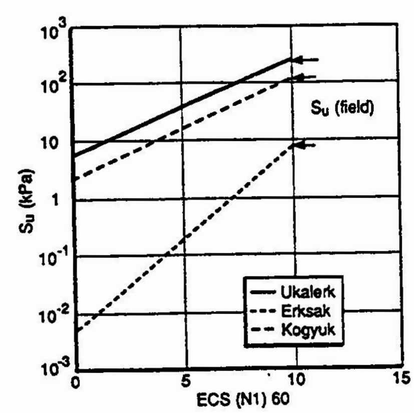

Fig.6 Application of proposed method to Beaufort Sea sands