Slope Failure Reanalysis of a Dilatant Dense Sand From CPTU and Laboratory Tests

U.F.A. Karim, A. Menkveld

Abstract. Geotechnical properties of dense to very dense sands at Channel Island Harbor in Oxnard, California, are re-evaluated after slope failures to select appropriate strength parameters for remedial and extension works. The revetment is excavated at a slope of 2:1 (horizontal: vertical) in estuarine and dune sands and covered with a geotextile filter and armor rock. Reanalysis of the slope stability explores laboratory and in situ strength conditions of peak, constant volume and residual angles for the sands. A stress-dilatancy theory provides the framework used in re-characterization of the soil and observed slope failure mechanisms. Effective friction angles are recommended for further slope stability analysis at that location.

Key words: slope stability, dense sand, site characterization, shear angle, dilatancy, case history.

1. Introduction

1.1. General

Re-evaluation of slopes and foundations that have collapsed provides insight into working frictional conditions, as long as the effective stress state in the soil over the life of the structure is reasonably well known. In this study, re-characterization of a slope failure in dense to very dense near surface sands for a harbor project at the Channel Island Harbor, Oxnard, California, led to improved assessment of geotechnical properties at the site, particularly effective stress friction angle, ϕ'. Selection of the appropriate friction angle (peak or large strain) is crucial in slope stability calculations, and this case study helped to provide some guidance for evaluating the available long-term strength of the soils.

After a brief description of the project, this paper will discuss results of laboratory and in situ tests analyzed under the critical-state framework adopted for soil re-characterization. Within that framework drained triaxial compression and direct shear testing are the most appropriate. Laboratory index tests of the sand are also used, primarily to compare sands at this site to standard reference sands and for the purpose of developing global correlations. The triaxial tests on reconstituted specimens placed at a range of relative densities of concern for this project are compared to stress-dilatancy relationship from Bolton (1986), to assess the applicability of that relationship as well as compressibility of the sand. Cone penetration test (CPT) data is used to evaluate profiles of relative density and peak effective stress friction angles, based on conventional correlations, for comparison with profiles of peak effective stress friction angles estimated from dilatancy relationships.

The use of peak or constant volume friction angles in re-design studies is further evaluated in limit-equilibrium calculations of slope stability. The friction angles obtained from critical state interpretations of laboratory and in situ test results that reproduce more closely the slope failure are recommended for use in further design.

1.2. Project background

The Channel Islands Harbor (CIH), located in the city of Oxnard, California, USA, was realized when two basins were constructed in the early 1980's by cutting slopes into the existing banks. The basins were excavated in the dry behind a temporary containment dike, and the dike was removed after all grades were established and slopes protection was in place. It is noted that during the grading of the site, sand dunes were flattened, creating an over consolidated profile of the upper sand layers. The slopes were originally designed at an angle of 2:1 (horizontal: vertical) as shown in Fig. 1. Slope protection was constructed by placing armor rock directly on top of a woven geotextile. After the construction of the basins was completed, the uplands soils were graded and improved for development of parking areas, sidewalks, landscaping and building improvements.

The original slopes were designed to a factor of safety of 1.5, which would correspond to using a friction angle of about 40° (Noble, 1996). By the mid-1990's portions of the slopes failed and sediments accumulated at the base. These sediments caused the floating dock sections to ground on the shoals at low tide. Redesign and stabilization for further site extension became necessary as a result of those events.

Selection of appropriate stabilization methods for the current slopes and identifying potential issues for future extensions of the harbor area necessitated geotechnical re-characterization of the old slopes. Evaluations of stratigraphy based on piezocone (CPTU) testing around the perimeter of the harbor identified 3 generalized profiles in the area of concern for stability analyses; (i) alternating sand and

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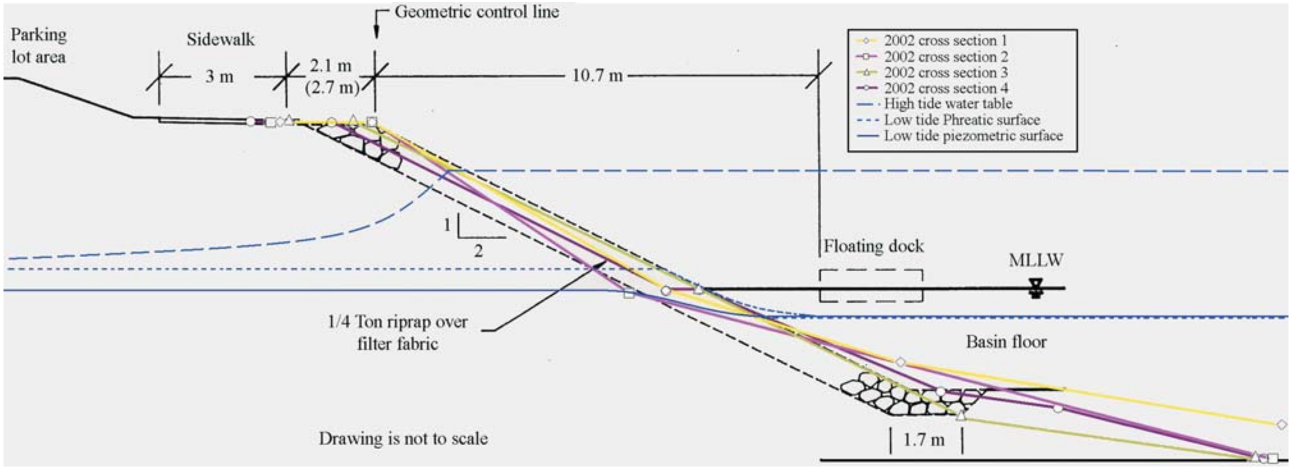

Figure 1 - Original slope design overlain by 2002 topography and range of tidal water conditions.

clay layers of approximately 2 to 3 m thickness; (ii) a continuous dense to very dense sand layer throughout the height of the slope; and (iii) interlayered sand and clay with thickness less than approximately 1 meter. The extents of these three generalized profiles are shown in Fig. 2. Since the continuous dense to very dense sand layers corresponded to areas of greatest slope distress, characterization of the dense to very dense sand and associated slope stability is the focus of this paper.

2. Site Characterization of Sand

Figure 2 - Extents of generalized soil profiles overlain on aerial photograph of site (Fugro, 2002).

2.1. Laboratory testing

The laboratory test program focused on evaluating index and shear properties of the sand for comparison to standard reference sand and interpretations using a critical state framework. Isotropic drained triaxial compression tests on specimens reconstituted at the appropriate relative densities D_r (50-100%) were conducted and the strength dilatancy theory as applied to this sand is assessed. The laboratory index tests are discussed in detail in Menkveld (2002) and include particle size analyses, microscopic evaluation of roundness, specific gravity, minimum index density, and evaluation of the ultimate state friction angle using simplified procedures (Cornforth, 1973; Santamarina & Cho, 2001). Average index properties for the CIH sands are presented in Table 1, as compared to those of standard reference sands. The simplified procedure for evaluating ϕ'_{cv} is shown in Figs. 3a and 3b.

The CIH sand is slightly finer grained than typical reference sands that form the basis of many CPT correlations for sands, but has a similar range of maximum and minimum void ratios. Additionally, the sand contains about 10% fines, which would classify the material as sand with silt, SP-SM under the Unified Soil Classification System (USCS). The higher fines content and smaller D_{50} (equivalent to average diameter for 50% of the sand) would generally lead one to consider the soil as having slightly higher compressibility than the reference sands, but the subrounded grains and predominantly sand matrix may lead the soil to behave in a similar manner to the low compressibility Monterey sand under drained penetration. The ultimate state friction angle (ϕ'_{cv}) determined by simplified procedures resulted in a value of approximately 34°, as shown on Fig. 3b. It was observed that some silt particles remained in suspension during the test, which may have resulted in slightly high value of ϕ'_{cv}. This may not be a significant problem in sands with silt, but in silty sands the simplified method is likely not representative of the actual

Table 1 - Grain characteristics of Chanel Islands Harbor sand to reference sands.

Sand[1,2]	D_{50} (mm)	D_{10} (mm)	Fines (%)	C_u	e_{max}	e_{min}	Angularity[3]	Compressibility
CIH_{mean}	0.25	0.07	10	3.4	0.92	0.51	SR	Low
Ticino	0.50	0.41	5	1.6	0.92	0.57	SA to A	Medium
Hokksund	0.39	0.21	5	2.2	0.88	0.54	SA to A	Medium
Monterey 0	0.37	0.25	5	1.6	0.82	0.54	SR	Low

[1]Properties for Ticino, Kokksund and Moterey 0 sand from Kulhawy & Mayne (1990).
[2]Chanel Islands Harbor (CIH) sand properties taken as the mean from laboratory testes. Varience of 5 to 10 percent was observed for D_{50}, e_{max} and e_{min}, and a varience or approximately 20 percent was observed for D_{10}, Fines and C_u.
[3]SR = Subrounded; SA = Subangular; A = Angular.

behavior, and ϕ'_{cv} values on the order of 27 to 30° could be expected (Bolton, 1986).

Bolton recognized that shear strength of cohesionless soils is related to the rate of dilation at failure, which in turn depends on the relative density D_r, the level of mean effective stress p' and soil compressibility. Therefore he developed an empirical method in which he uses Rowe's dilatancy concepts. Bolton showed that the peak secant friction angle ϕ'_{sec} for many sands could be estimated from triaxial tests using the expression:

$$\phi'_{sec} - \phi'_{cv} = 3\, D_r\, (Q - \ln p'_f) \tag{1a}$$

For plane strain he found that,

$$\phi'_s - \phi'_{cv} = 5\, D_r\, (Q - \ln p'_f) - 1 \tag{1b}$$

where p'_f (or p'_{ff}, as used in Bolton 1986) is the mean effective stress at failure, and Q is a constant with a value depending on the compressibility and mineralogy of the sand. Bolton suggested a general value of Q = 10 for most silica sands.

Drained triaxial compression tests on a dense and medium dense specimen were evaluated using the Bolton (1986) stress dilatancy theory (Eq. (1)). Figure 4 compares laboratory results of CIH sand to trends presented in Bolton (1986). Bolton recommended using a Q value, which is indicative of the soil compressibility, of 10 for quartz and feldspar sands, and lower values for more compressible materials. Triaxial test results for this study agreed well with the Bolton (1986) theory, and seemed to match best with a Q value of 11, indicating a low compressibility soil. It should be noted that a low number of triaxial tests were performed, which limited evaluation of the influence of confining stress, an important aspect of stress dilatancy theory.

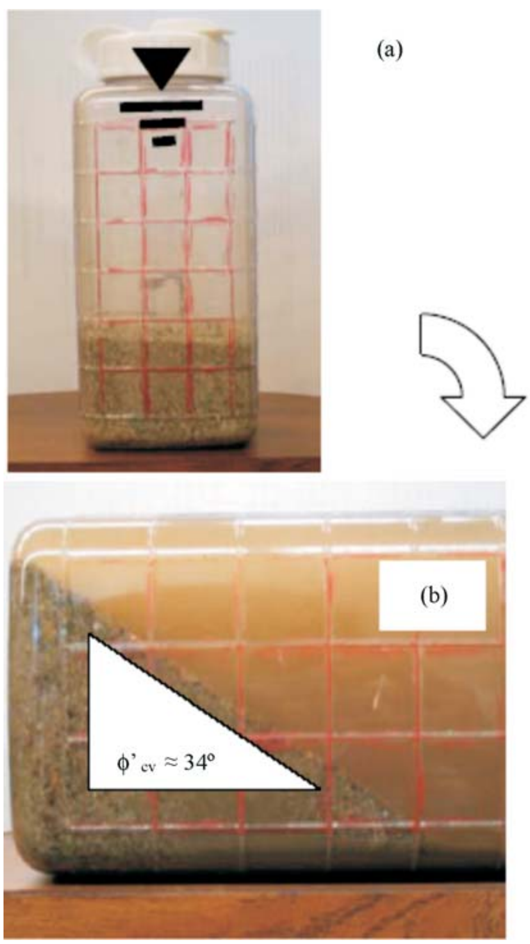

Figure 3 - Method for estimating ϕ'_{cv} based on Cornforth (1973).

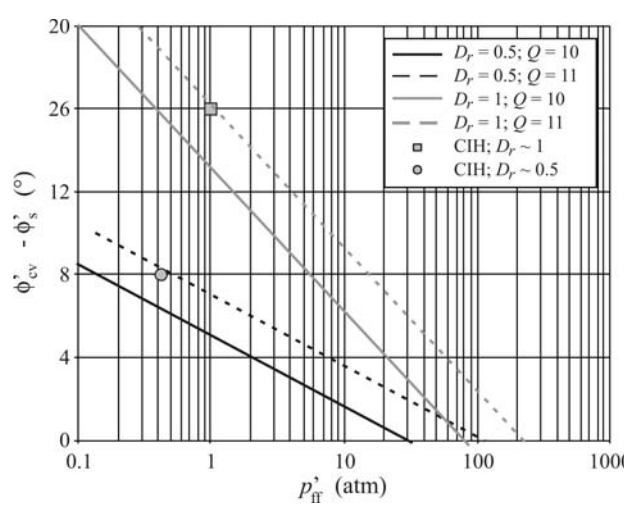

Figure 4 - Evaluation of CIH triaxial data using Bolton's (1986) stress dilatancy theory with ϕ'_{cv} = 34° (1 atm = 1 bar = 101.3 kPa).

2.2. *In situ* testing

Sixteen CPTs were performed around the perimeter of the site to generate the zones of generalized soil profiles presented in Fig. 2. As mentioned previously, this study focused on the dense to very dense sands of Typical Profile 2. Also, since the height of the sand dunes were reduced during grading operations, the upper sand profile is over consolidated, which was taken into consideration during analyses. The influence of over consolidation primarily influences cone penetration resistance through an increase in horizontal stress.

Five of the CPTs performed are included under Typical Profile 2, and, due to space constraints, only two of those CPTs will be presented in this section. CPTU-6 and CPTU-11 are located on the west side of the basins, and are separated by approximately 500 m. Analyses related to characterization of these profiles included assessment of relative density, and peak effective stress friction angle. The upper 7 m of the profile is of primary concern for assessment of slope stability issues, and the upper 1 meter is left off plots since it is primarily artificial fill.

Analysis of relative density of the profile was initiated by using correlations presented Baldi et al. (1986), which were primarily based on results of calibration chamber tests in Hokksund and Ticino sands. To account for over consolidation of the deposit, the relative density (D_r) is correlated to penetration resistance normalized by mean effective stress, and over consolidated profiles have an higher cone tip resistance at the same depth than normally consolidated profiles for an equivalent D_r. Profiles of CPT tip resistance (q_c) and estimates of relative density for the dense (D_r = 65%) and very dense (D_r = 100%) states are presented on Fig. 5. Despite being finer grained and having a higher fines content than the reference sands used for the Baldi *et al.* (1986) correlation, the recorded q_c values were 10 to 20 MPa higher than the Baldi *et al.* (1986) correlated value for 100 percent relative density. While at this point it is obvious that the upper sand is very dense, it is desirable to understand the large discrepancy for predicted and observed q_c values, and how this influences the interpretation of sand deeper layers.

Also shown on Fig. 5 are relative density correlations based on equations presented in Mayne & Kulhawy (1995) relating relative density to cone tip resistance normalized by horizontal stress. This correlation encompasses a much larger database of different sand types, with scatter often associated with the compressibility of the sand. The mean value of the correlation plots as the lower bound of the q_c values, with the mean plus 2 standard deviations plotting as the upper bound of the data. From this figure it is inferred that there is significant variability in evaluation of relative density from conventional correlations related to the compressibility of the soil. Additionally, without proper characterization of the soil compressibility, relative density may be significantly overestimated. The variability in correlations seems to be much more significant in dense to very dense soils.

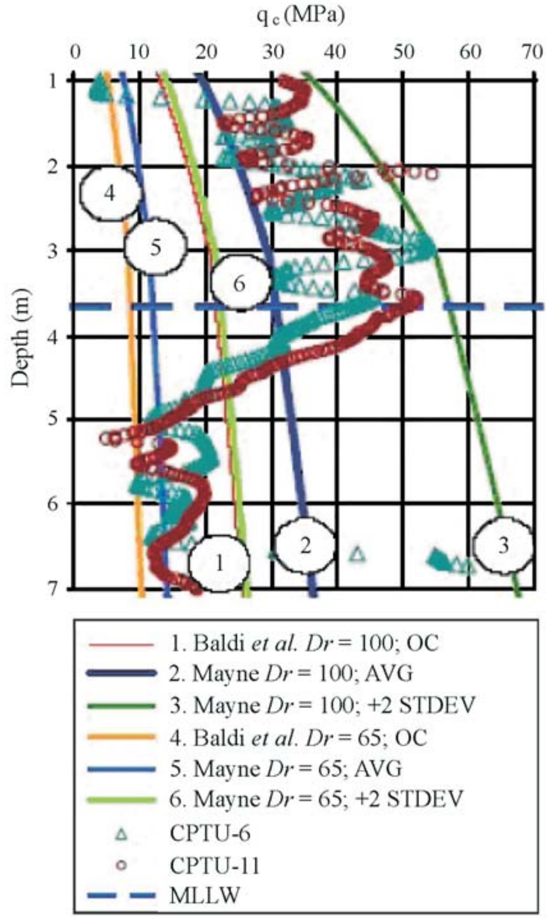

Figure 5 - Range of relative density estimates using typical correlations.

For design purposes, it is desirable to have an estimate of the peak effective stress friction angle. To develop peak strength profiles, correlations between ϕ'_{peak} and q_c as presented in Kulhawy & Mayne (1990) are utilized. Additionally, the influence of soil compressibility on cone tip resistance will be better matched by a change in effective stress friction angle than by a change in relative density (Robertson and Campanella, 1988). So the correlation between ϕ'_{peak} and q_c may be applicable over a larger range of sands than D_r correlations. With a knowledge of ϕ'_{cv} (Fig. 3b) and application in a stress dilatancy framework, profiles of effective stress friction angle can be used to assess relative density of a deposit. The dilatancy component of friction angle for the stress dilatancy theory was calculated by starting at the mean effective stress for a certain depth and moving towards the failure envelope along the 3:1 triaxial stress path. While it is acknowledge that the stress path during cone penetration will be different than triaxial compression, since the q_c-ϕ'_{peak} correlation is based on triaxial test data and the stress dilatancy theory is based on triaxial test data, this is a reasonable stress path to follow for the analyses performed.

Profiles of ϕ'_{peak} are shown for CPTU-6 and CPTU-11 on Fig. 6, along with profiles of ϕ'_{peak} based on stress dilatancy theory for dense and very dense states. The estimates of ϕ'_{peak} from q_c values tend to match well with stress dilatancy theory for a Q value of 11, if the upper soil is classified as very dense and the lower soil is classified as dense. Other than developing a profile of peak friction angle, the combined assessment of relative density and effective stress friction angle analyzed under a stress dilatancy framework, can be used to verify that a constant volume friction angle of 34° is reasonable for this deposit. It should be noted that the mean lower low water table MLLW shown in Figs 5 and 6 is based on high and low tide values (see Fig. 1). This table within the soil slope coincides approximately with the piezometric measurement and shows the relative position from which the cone tests were made at about 1.2 m above the high tide water table.

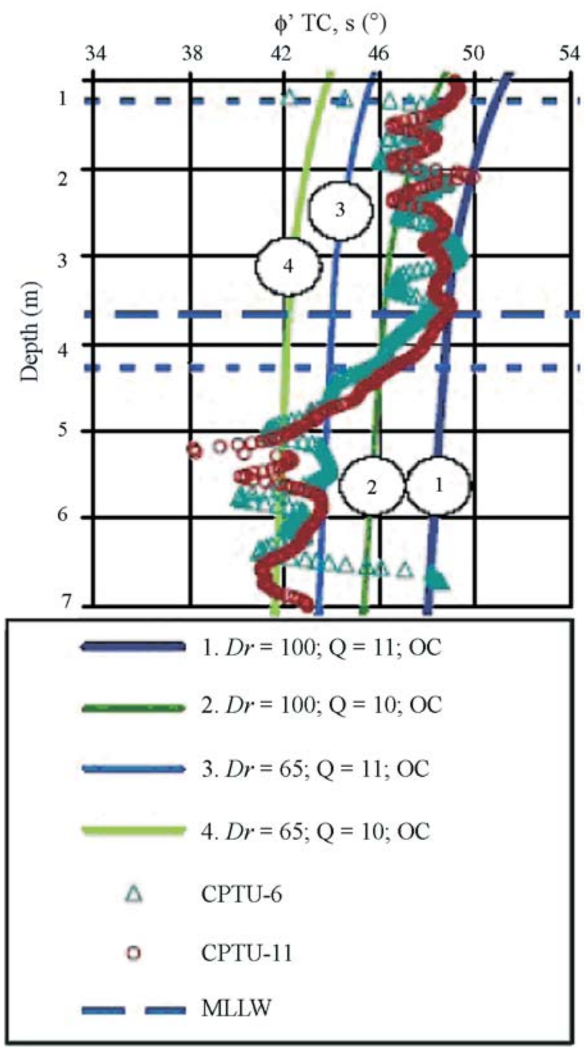

Figure 6 - Range of peak effective stress friction angle as a function of relative density (MLLW: mean lower low water is at approximate depth of low tide piezocone surface as shown in Fig. 1).

3. Slope Stability Analyses

Slope stability analyses are performed using a range of effective stress friction angles developed from the site characterization activities. Slope stability analyses were performed using plain strain and limit equilibrium theory as incorporated into the computer program SLOPE/W[1]. Strength parameters associated with computed critical slip surfaces that match observed failure surfaces are assessed within the previously mentioned stress dilatancy framework to determine the most suitable friction angle under working conditions.

SLOPE/W input parameters include range of friction angles, slope section, water table (high and low tide). The range of friction angles used as input are the peak friction angles obtained from triaxial and direct shear tests, the ultimate friction angles obtained with the Cornforth (1973) method (both the lowest, 31° and the mean value 34° are used). The top sand layer, being denser than the underlying sand, is therefore designated the higher values from the range of friction angles, except when both layers are at the ultimate state. Using secant effective stress friction angle of 50° in the very dense sand and 42° in the dense sand resulted in a factor of safety of approximately 2.0. When the effective stress friction angle was reduced to the ultimate state value of 34° and 31°, factor of safety values of 1.3 and 1.1 were calculated, respectively.

4. Discussion

Friction angles calculated from the dilatancy relationships presented by Bolton (1986) have led to a unified framework that matches laboratory test data, in situ test data, and field performance of a slope failure. As shown in Fig. 6 there is good agreement between the CPT estimates of ϕ'_{peak} and ϕ'_{peak} from stress dilatancy theory. It is therefore inferred from the CPT analyses that the ultimate state friction angle of approximately 34° can be a reasonable value for use in slope stability analyses. This is confirmed by the results of the laboratory tests presented in Figs. 3b and 4.

Due to the cyclic nature of the tidal loading on the slopes, design of remedial measures and future expansion based on slope stability in these sands, use of the ultimate state friction angle, ϕ'_{cv}, equal to 34° for the both the dense and very dense sands is recommended. This is a reasonable choice given the several complicating factors that need to be considered to accurately estimate working parameters for design. Cyclic tidal loading and seepage conditions are among the most important factors, as well as construction methods and materials (*i.e.*, armor rock and filter fabric) used.

1 Geo-Slope International, Calgary, Alberta, Canada, developed the SLOPE/W program.

The safety factor of the re-constructed slopes, with respect to a peak friction angle, may gradually decreased with time due to a number of factors. As noted earlier, failure took place after slope deterioration spanning 10 years. Ultimate state conditions may have been reached after some years because cyclic high and low tides (to factor of safety values ranging between 1.5 and 2.0), and seepage conditions have gradual effects on the initial slope properties. Numerous loading cycles generate enough inelastic strains so that a point on the post-peak part of the stress-displacement curve is approached. Consequently, the sands are eventually controlled by the ultimate state. The original design friction angles of about 40° are reduced due to these effects by 6° leading to reduction in the safety factor from 1.5 to 1.0. It is possible that the designers overlooked this long-term effect and therefore assumed higher friction angles.

5. Conclusions

The primary conclusions of this study are:

• Slope stability failures are possible in dense to very dense sand deposits designed to a factor of safety of 1.5 using peak effective stress parameters, especially if subjected to transient loads, such as tides.

• Relative density correlations that underestimate the compressibility of the soil will overestimate the relative density of the deposit, and possibly lead to unconservative analyses.

• The cone penetration resistance seems to be strongly related to horizontal stress and a dilatancy component, and the normalizing cone tip resistance by mean stress may lead to overestimates of relative density in over consolidated deposits.

• Evaluating relative density based on correlations to ϕ'_{peak} assessed under a stress-dilatancy framework with lab estimates of ϕ'_{cv} seem to be less sensitive to soil compressibility. These relationships can be further calibrated using a limited number of reconstituted triaxial compression tests to further evaluate the soil compressibility.

• Evaluation of CPT tip resistance using empirical correlations and simple laboratory tests can provide insight into the peak and large strain friction angle of the soil.

• Back calculation of slope failure conditions using limit equilibrium analyses provided mobilized friction angles consistent with ϕ'_{cv} values estimated from simplified laboratory index tests and verified using CPT data analyzed under stress-dilatancy theory. The analyses also highlighted that pore pressure conditions and construction materials and methods play an important role in the occurrence of shallow failures.

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Notation

Cu = undrained shear strength

D = grain diameter in mm (subscripts 10% passing, 50% passing)

D_r = relative density

e = void ratio (subscript min = minimum and max = maximum)

M = strength parameter

p' = hydraustatic mean effective stress (subscripts f or ff at failure)

q_c = cone tip resistance

Q = value indicative of soil compressibility (stress dilatancy theory, Bolton,1986)

ϕ'_{peak} = maximum effective stress friction angle (from laboratory shear tests)

ϕ'_{cv} = effective stress friction angle at constant volume (laboratory shear test)

ϕ'_{TC} = peak effective stress friction angle (from a CPT cone test)

ϕ'_{sec} = peak secant friction angle

Abbreviations

AVG: Average
CPT: Cone penetration test
CPTU: CPT with pore water pressure measurment
CIH: Channel islands harbour
MLLW: mean lower low water
OC: Over consolidated
STDEV: standard deviation