Correlations of Unconsolidated-Undrained Triaxial Tests and Cone Penetration Tests

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Unconsolidated-undrained (UU) triaxial and cone penetration test results were used to develop correlations between undrained shear strength and cone resistance for three soft to medium alluvial clays in the San Diego area. The cone factor relating the UU triaxial strength to the cone resistance, termed N_{kuu}, had an average value and range of 11.0 ± 2.0, 11.0 ± 2.5, and 12.4 ± 0.8 for the Lopez Ridge, Creekside Estates, and Rancho Del Oro sites in San Diego, respectively. A data base of additional values of N_{kuu} was compiled from the literature for nonfissured, normally to lightly overconsolidated clays (overconsolidation ratios ranged from 1 to 5). The data base of N_{kuu} values showed considerably less scatter than that observed in previous cone factors on the basis of field vane shear tests. The reduction in scatter is believed to be due to the uncertainty in interpreting vane shear tests and the repeatability of UU triaxial tests when high-quality samples are available.

In southern California the cone penetration test (CPT) is frequently used during initial site investigations to provide information for an efficient boring and sampling program. CPT provides quick insight into soil stratigraphy and also identifies soil layers that might be problematic and require additional testing during the remainder of the investigation. Laboratory testing programs are then designed to measure the engineering properties of those soil layers by using high-quality samples obtained from soil borings located by using the CPT results. Currently in San Diego, most of the geotechnical design is based on the results of the laboratory tests. In an effort to incorporate the CPT results into the geotechnical design process, correlations between cone penetrometer resistance and undrained shear strength are being developed for soil deposits in the San Diego area.

REVIEW OF EXISTING CORRELATIONS

The undrained shear strength for clays is derived from CPT results by using theoretical solutions or empirical correlations or both. Baligh et al. (*1*) present a comprehensive overview of the different theories that can be grouped into the following three main categories: (a) bearing capacity, (b) cavity expansion, and (c) steady penetration. Those three methodologies employ a form of the traditional bearing capacity equation:

$$q_c = N_c S_u + \sigma_{vo} \tag{1}$$

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where

q_c = cone resistance,
N_c = bearing capacity factor,
S_u = undrained shear strength, and
σ_{vo} = total vertical stress.

Each method incorporates a different expression for N_c and the total overburden stress, such as the horizontal or the octahedral stress, to determine the undrained shear strength.

EMPIRICAL CORRELATIONS

Owing to the difficulties in estimating the in situ horizontal stress and evaluating the various expressions for N_c, an empirical equation similar to Equation 1 is frequently used in practice to relate cone resistance to undrained shear strength. The empirical expression commonly used in practice is

$$q_c = N_k S_u + \sigma_{vo} \tag{2}$$

where N_k is the empirical cone factor.

The first empirical correlations relating q_c and S_u were developed in Europe, and, as a result, the reference undrained shear strength was usually determined from the results of field vane shear tests. Previous data collected by Lunne and Kleven (*2*) and Jamiolkowski et al. (*3*) showed that the empirical cone factor N_k decreases with plasticity index and ranges from 9 to 26 when S_u is measured by using a field vane shear test.

Bjerrum (*4*) reviewed 16 well-documented embankment failures on cohesive foundations and developed the field vane correction factor μ, as indicated in Figure 1. The correction factor reduces the measured strength to reflect the influence of anisotropy and strain rate effects on the undrained strength. Other researchers (*5–10*) have contributed additional data from other embankment failures for Figure 1. The additional data have increased the scatter about Bjerrum's recommended curve, leading some to question the use of the vane shear test for design.

If the vane shear strength values are corrected by using Bjerrum's field correction factor μ, the resulting corrected cone factor ($N_k^* = N_k/\mu$) appears to be independent of plasticity index and shows slightly less scatter than N_k. As indicated in Figure 2, the majority of the published N_k^* values are between 10 and 24, with an average of approximately 15. However, even after correcting the field vane shear strength, the values of N_k^* still show considerable scatter. The scatter shown in Figure 2 makes the determination of a design undrained shear strength very difficult.

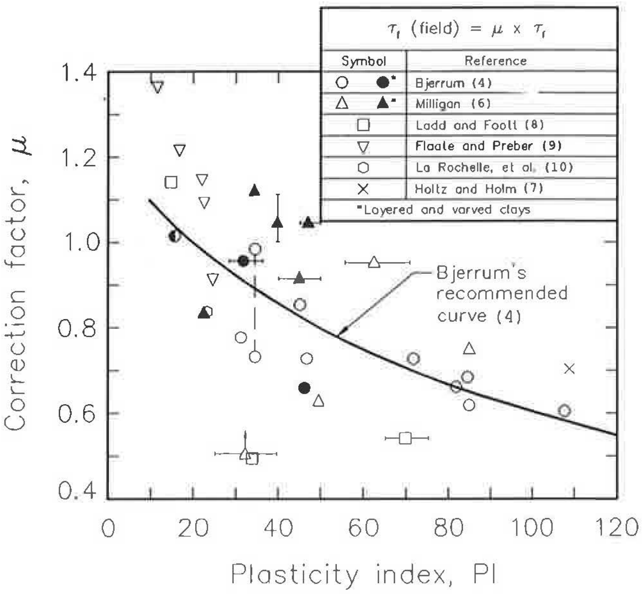

FIGURE 1 Correlation factor for the field vane test as a function of plasticity index, based on embankment failures [adapted from Holtz and Kovacs (*36*)].

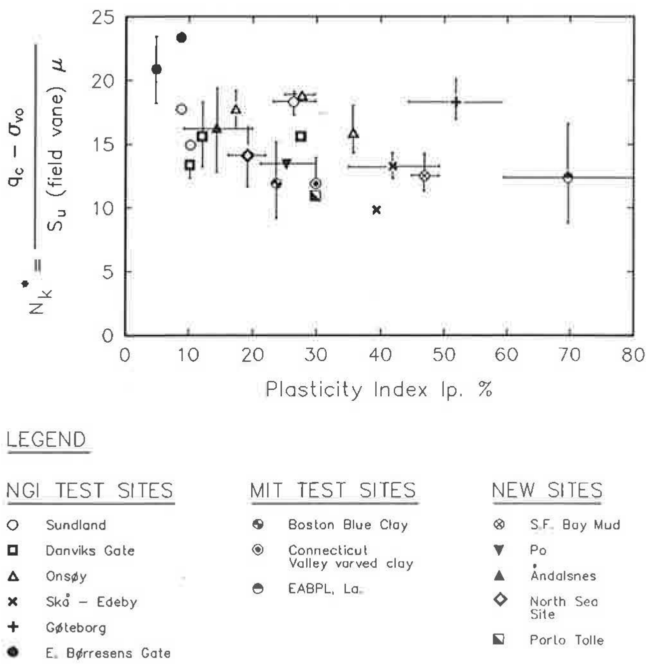

FIGURE 2 Previously published corrected cone factors [after Lunne and Kleven (*2*) and Meigh (*37*)].

NEW EMPIRICAL CONE FACTOR, N_{kuu}

The value of the empirical cone factor varies considerably depending on the type of cone, cone test procedure, the reference strength, and, most important, the soil deposit. The data bases of empirical cone factor currently available in the literature have not always been consistent because researchers have used different types of cones and different tests to measure the undrained shear strength. Researchers have also used the total horizontal stress or mean octahedral stress in Equation 2 instead of the total overburden pressure. The main objective of this research was to develop a new cone factor by using the tip resistance from standard electrical cones tested in accordance with ASTM standards, the total overburden pressure, and a consistent measurement of undrained shear strength.

A number of different techniques for measuring the undrained shear strength (field vane, isotropically consolidated-undrained triaxial, unconfined compression, anisotropically consolidated-undrained triaxial, unconsolidated-undrained triaxial, direct simple shear, and plane strain) were considered during this study. Despite the limitations of the unconsolidated-undrained (UU) triaxial test, the undrained shear strength obtained from this test is still widely used for design in the United States. The UU triaxial test provides repeatable results when high quality samples are available, does not require sophisticated laboratory equipment, and is very cost effective. Ladd et al. (*11*) also pointed out that the errors associated with UU triaxial tests are, "to some extent," self-compensating because disturbance decreases the strength while anisotropy and strain rate effects increase the strength. However, Ladd et al. warned that the effects of disturbance, anisotropy, and rate of loading are variable, and, therefore, considerable judgment should be used for cases where the factor of safety is "low."

Owing to the popularity of the UU triaxial test, the uncertainties in interpreting the vane shear test, and the difficulties in performing the other undrained strength tests mentioned, only values of S_u measured in UU triaxial tests were used in the correlations reported herein. Unconfined compression tests were not considered to be a UU triaxial test and were not used in the correlations. Therefore, the cone factors presented herein will be referred to as N_{kuu} and should be utilized to determine the undrained shear strength for use in total stress or end-of-construction stability analyses.

SAN DIEGO TEST SITES

To facilitate the use of CPTs in the San Diego area, a research program was initiated to develop cone factors for local soil deposits. To date, three sites—Lopez Ridge (*12*), Creekside Estates (*13*), and Rancho Del Oro (*14*)—have been studied. At each site a minimum of 10 cone soundings was performed by Earth Technology Corporation, using a standard electrical cone in accordance with ASTM D3441. Exploratory borings were drilled within 15 to 20 ft of selected cone penetration soundings to obtain high-quality, 3-in. diameter Shelby tube samples for laboratory testing.

All three sites are located within alluviated canyons that are proposed for development. The proposed Lopez Ridge project will necessitate the construction of a roadway embankment fill approximately 700 ft in length and varying in height from 10 to 30 ft. The proposed Creekside Estates and Rancho Del Oro projects involve the placement of compacted fills 10 and 25 ft deep, respectively. Those fills will be used to create building pads for single-family homes.

LABORATORY TEST RESULTS

Classification tests, a minimum of one consolidation test, and a minimum of three unconsolidated-undrained triaxial tests were performed in accordance with ASTM standards on each Shelby tube sample obtained from the various sites. The measured soil properties of the canyon alluvium at the three San Diego sites are presented in Table 1. The alluvium ranges flow a low to high plasticity clay at the Lopez Ridge and Rancho Del Oro sites to a high plasticity clay or silt at the Creekside Estates site. Geologically, the alluvial deposits are young and are normally to lightly overconsolidated. As presented in Table 1, the undrained shear strength measured in UU triaxial tests ranged from 0.30 to 0.63 ton/ft^2 and was the highest at the Rancho Del Oro site. All UU triaxial tests specimens had a degree of saturation greater than 97 percent.

TABLE 1 PROPERTIES OF CANYON ALLUVIUM AT LOPEZ RIDGE CROSSING, CREEKSIDE ESTATES, AND RANCHO DEL ORO SITES

Property	Lopez Ridge	Creekside Estates	Rancho Del Oro
Alluvium thickness, ft.	10-40	30-35	40-70
Alluvium Classification	CL	CH	CH-MH
Plastic Limit	18-20	24-32	28-30
Liquid Limit	38-40	60-80	60-68
Plasticity Index	20	36-48	30-40
Natural Water Content, %	28-30	45-60	40-45
Overconsolidation Ratio	1-1.3	1-1.2	1-1.6
UU Triaxial Shear Strength, tsf	0.30-0.37	0.42-0.54	0.50-0.63
Net Cone Resistance, tsf	3.2-3.8	4.6-5.7	5.3-7.3
UU Triaxial Cone Factor, N_{kuu}	11.0 ± 2.0	11.0 ± 2.5	12.4 ± 0.8

NEW CORRELATIONS OF UU TRIAXIAL TESTS AND CONE PENETRATION TESTS

Empirical cone factors were calculated by using the undrained shear strength from UU triaxial tests, the electrical cone resistance, and the total overburden stress at the depth of the sample. At the Lopez Ridge site, the average value of N_{kuu} was 11.0 with a range of ±2.0. The Creekside Estates site also had an average value of N_{kuu} equal to 11.0 with a range of ±2.5. The Rancho Del Oro site had an average N_{kuu} value of 12.4 with a range of ±0.8. The range of net cone resistance, $q_c - \sigma_{vo}$, and UU triaxial shear strength used in the calculations of N_{kuu} at each site are presented in Table 1.

DATA BASE OF UU TRIAXIAL TESTS AND CONE PENETRATION TESTS

An extensive literature search was conducted to create a data base of sites at which values of N_{kuu} could be determined to investigate the accuracy of the N_{kuu} values calculated for the San Diego sites. A total of 18 sites was collected for the data base, and additional sites were being sought. Only sites with undrained shear strengths measured in UU triaxial tests and test specimens having a degree of saturation at or near 100 percent were selected. Unconfined compression test results were not used in the correlations. In addition, only cone soundings, using a standard electrical cone advanced at approximately 2 cm/sec (0.78 in./sec) and in accordance with ASTM D3441, were used in the correlations. The electric cones all had an apex angle of 60 degrees and a projected area of 10 cm^2 (1.55 in.^{2}). The sites, sources of the data, and the symbols used to represent the data are presented in Table 2.

VARIATION OF CONE FACTOR WITH PLASTICITY INDEX

Figure 3 indicates the variation of N_{kuu} as a function of plasticity index (PI) for the 18 data-base sites and the three San

TABLE 2 LISTING OF SITES, SYMBOLS, AND REFERENCE NUMBERS USED IN CORRELATIONS BETWEEN UU TRIAXIAL AND CONE PENETRATION TESTS

SYMBOL	SITE	(Reference)	SYMBOL	SITE	(Reference)
	AUGUSTA	(15)	■	PORTO TOLLE	(3)
	BEAUMONT	(17)		RANCHO DEL ORO	(14)
	BEAUMONT	(18)		SAINT ALBAN	(25)
	BOSTON BLUE	(19, 20)		S.F. BAY MUD	(26)
	CRAN	(21)		S.F. BAY MUD	(27)
	CREEKSIDE	(13)		SANTA BARBARA (SOFT)	(28)
◣	HAGA	(22)		SANTA BARBARA (STIFF)	(28)
	LOPEZ RIDGE	(12)		SILTY HOLOCENE	(24)
▲	OTTAWA SEWAGE PLANT	(23)	●	TEXARKANA	(29)
	PLASTIC HOLOCENE	(24)	▼	VAL DI CHIANA	(30)

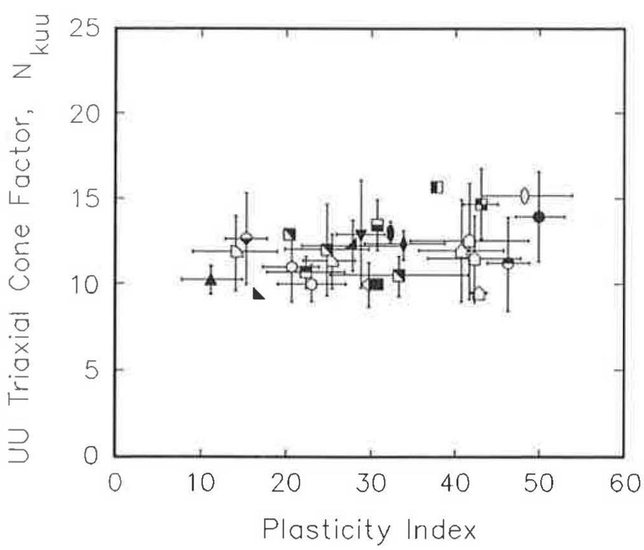

FIGURE 3 Variation of UU triaxial cone factor with plasticity index.

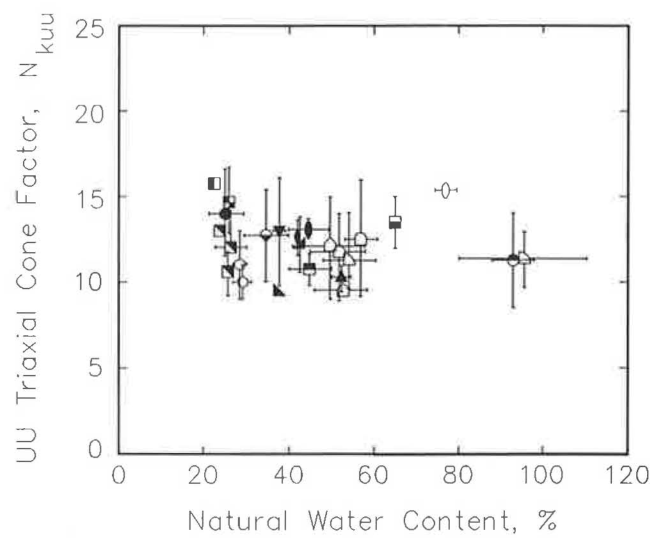

FIGURE 4 Variation of UU triaxial cone factor with natural water content.

Diego sites. It can be seen that the values of N_{kuu} range from 8.5 to 16.5, with an average value of approximately 12. Each data symbol represents the median value of PI and N_{kuu} calculated at each site, while the lines surrounding each point illustrate the range of PI and N_{kuu}. The symbols for the San Diego and San Francisco Bay Mud (*12*) sites correspond to the median value of N_{kuu} for a particular boring.

In a comparison of Figures 2 and 3, N_{kuu} shows considerably less scatter than the corrected cone factor N_k^*. The reduction in scatter is probably due to the use of tip resistance values measured by using only a standard electrical cone and the repeatability and simple interpretation of UU triaxial tests. Some of the scatter observed in N_k^* is probably due to soil anisotropy, strain rate effects, and the difficulties in interpreting and performing field vane shear tests.

VARIATION OF CONE FACTOR WITH LIQUIDITY INDEX

Figure 4 presents the variation of UU triaxial cone factor with the natural water content. The majority of the natural water contents ranges from 20 to 60. In a comparison of Figures 3 and 4, the range in natural water content for a particular site was significantly smaller than that observed in PI. In an effort to incorporate natural water content into the correlations, N_{kuu} was plotted against the liquidity index (LI).

The LI provides an index for scaling the natural water content and an insight into the engineering behavior of the deposit. It can be seen from Figure 5 that the majority of the sites had an LI ranging from 0.2 to 1.0, which indicates a plastic behavior during shear. This behavior is typical for the normally consolidated to lightly overconsolidated clays investigated during this study. Therefore, the use of the LI may provide a better index for N_{kuu} than PI because it incorporates information about water content, plasticity, and the engineering behavior of the soil. In addition, sensitivity S_t can be estimated from LI by using data presented by Eden and Kubota (*31*) and Bjerrum (*16*). Their data were used to derive the following equation for estimating sensitivity:

$$S_t = 10^{(\mathrm{LI} - 0.20)} \qquad (3)$$

To facilitate the determination of undrained shear strength, the data base was replotted in terms of net cone resistance $(q_c - \sigma_{vo})$ and undrained shear strength. It can be seen from Figure 6 that the majority of the data plots along a straight line corresponding to a value of N_{kuu} equal to approximately 12. The symbols in Figure 6 correspond to the median value of N_{kuu} for a particular site or boring. The scatter of N_{kuu} appears to increase slightly as the net cone resistance and undrained shear strength increase. This is probably due to the uncertainty of interpreting cone measurements in stiff clays.

VARIATION OF CONE FACTOR WITH UNDRAINED STRENGTH RATIO

Ladd and Foott (*8*) showed that the undrained shear strength of clays is controlled by the effective consolidation stress σ'_{vc}, or the overconsolidation ratio (OCR) or both. As a result, Mayne and Kemper (*32*) and Wroth (*33*) have suggested plotting the normalized net cone resistance $(q_c - \sigma_{vo})/\sigma'_{vc}$ versus the undrained strength ratio S_u/σ'_{vc}. The advantages of using the normalized net cone resistance are that it is dimensionless and it is directly related to the overconsolidation ratio as shown below:

$$\frac{(q_c - \sigma_{vo})}{\sigma'_{vc}} = \frac{(q_c - \sigma_{vo})}{S_u} \cdot \frac{S_u}{\sigma'_{vc}} = N_{kuu} \cdot \frac{S_u}{\sigma'_{vc}}$$

$$= N_{kuu} \cdot f(\mathrm{OCR}) \qquad (4)$$

It can be seen from Figure 7 that the normalized net cone resistance is directly related to the undrained strength ratio. Also indicated in Figure 7 is a line that corresponds to N_{kuu}

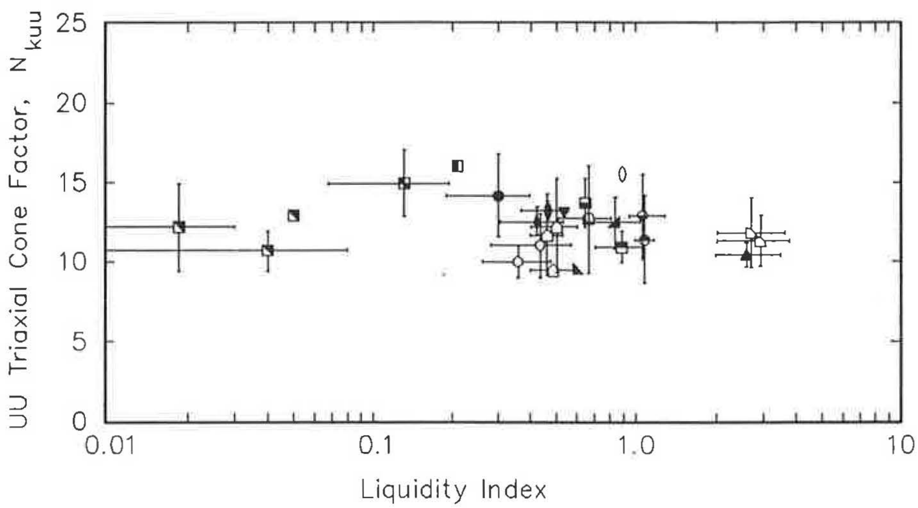

FIGURE 5 Variation of UU triaxial cone factor with liquidity index.

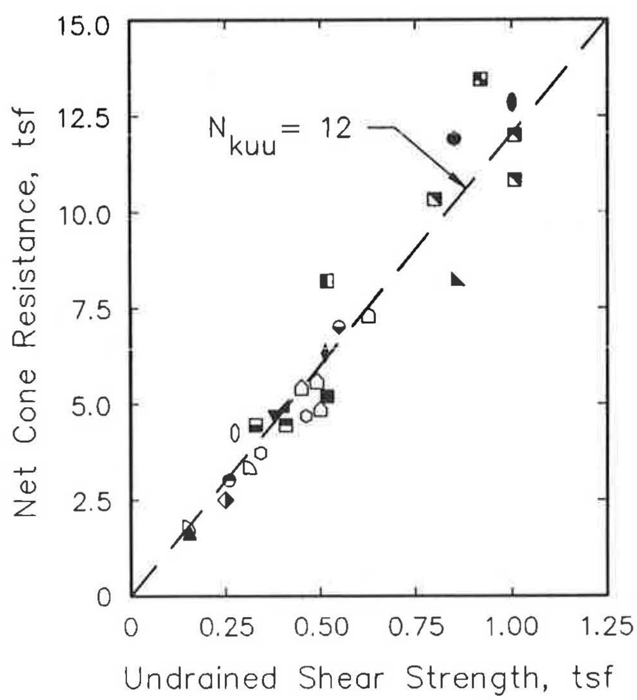

FIGURE 6 UU triaxial cone factor as a function of net cone resistance and undrained shear strength.

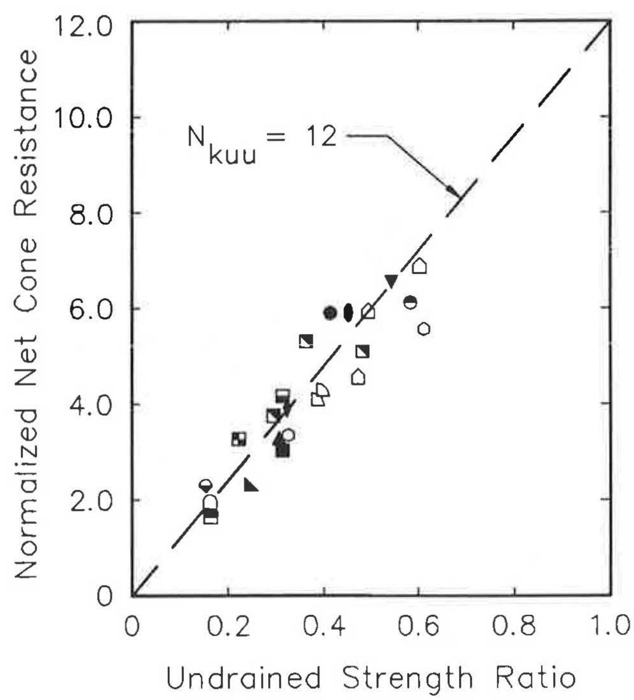

FIGURE 7 Variation of UU triaxial cone factor with normalized net cone resistance with undrained strength ratio.

equal to 12, which again is in good agreement with the data. Therefore, a reasonable estimate of undrained strength ratio for nonfissured, normally to lightly overconsolidated (overconsolidation ratios ranging from 1 to 5) clays can be obtained directly from values of normalized net cone resistance, using an N_{kuu} of approximately 12.

VERIFICATION OF UNDRAINED SHEAR STRENGTH

The undrained shear strength obtained from the design charts presented here should be verified by using previously published relationships for undrained strength ratio. One of the most widely used relationships was presented by Jamiolkowski et al. (*33*) and is shown below:

$$\frac{S_u}{\sigma'_{vc}} = (0.23 \pm 0.04) \cdot \mathrm{OCR}^{0.8} \quad (5)$$

This relationship is applicable to most soft sedimentary clays of low to medium plasticity and is frequently used to evaluate embankment stability. This relationship was developed primarily from the results of direct simple shear tests. Data presented by Ladd and Edgers (*35*) and Jamiolkowski et al. (*34*) have shown that triaxial compression tests yield slightly higher values of undrained shear strength than direct simple shear tests. As a result, to obtain an estimate of S_u that corresponds to the UU triaxial strength, the coefficient in Equation 5 can be increased to approximately 0.3 and the equation simplified to what follows for most clays:

$$\frac{S_u}{\sigma'_{vc}} = (0.30) \cdot \mathrm{OCR}^{0.8} \qquad (6)$$

SUMMARY

Values of the empirical cone factor vary considerably depending on the type of cone, cone test procedure, the reference strength, and, most important, the soil deposit. The main objective of this research was to develop a new cone factor N_{kuu} for nonfissured, normally to lightly overconsolidated clays (overconsolidation ratios ranging from 1 to 5) using the tip resistance from only electrical cones tested in accordance with ASTM Standard D3441 and values of undrained shear strength measured in UU triaxial tests. Undrained shear strengths measured by using isotropically consolidated-undrained, unconfined compression, anisotropically consolidated-undrained triaxial tests, vane shear, or other strength tests were not used in the correlations reported herein.

UU triaxial cone factors N_{kuu} were calculated for three soft to medium canyon alluviums in the San Diego area. The average value and range of N_{kuu} were calculated to be 11.0 ± 2.0, 11.0 ± 2.5, and 12.4 ± 0.8 for the Lopez Ridge Crossing, Creekside Estates, and Rancho Del Oro sites, respectively. An extensive literature search was conducted to locate 18 additional sites for which N_{kuu} could be calculated. Variations of the UU triaxial cone factor with plasticity index, natural water content, liquidity index, net cone resistance, and undrained strength ratio were developed from the data base. Those correlations show significantly less scatter than that observed in previous cone factors based on field vane shear tests. The reduction in scatter is believed to be due to the uncertainty in interpreting vane shear tests and the repeatability of UU triaxial tests when high-quality samples are available.

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