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Estimation of the mobilized shear strength of soft clays under different loading and unloading conditions

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ABSTRACT: In order to account for uncertainties encountered during the design process, the safety factor concept is widely used in geotechnical engineering. Despite this fact, a large number of earth structures continue to fail. The paper presents an empirical approach for estimating the mobilized shear strength at failure under different loading and unloading conditions in soft clays. The empirical approach, which is believed to lead to safer designs, is based on an accurate measurement of the preconsolidation pressure, probably the most important soil property.

KEYWORDS: Mobilized shear strength, preconsolidation pressure

1 INTRODUCTION

Although important progress has been achieved in understanding the fundamentals of clay behavior in recent decades, difficulties still exist in determining the mobilized shear strength of clays at failure under various loading and unloading conditions. Gue and Tan (2004) report that out of 55 cases of failure in soft ground that they reviewed in a period of four years, nearly 50% of the failures were caused by inadequacy in design. Therefore, confronted with problems in designing safe structures, geotechnical engineers continue to rely on semi-empirical methods. The following sections describe such a method, which is based on a fundamental soil parameter, and gives quite reliable results.

2 EMPIRICAL DETERMINATION OF THE MOBILIZED SHEAR STRENGTH

Several empirical methods were proposed to evaluate the mobilized shear strength of clays.

2.1 *Contribution by Bjerrum (1972)*

The reliability of stability calculations based on in situ vane strength values measured in soft cohesive soils has been a matter of concern since the introduction of the field vane apparatus into routine geotechnical investigations (Cadling and Odenstad 1950). In early applications, shear strength values measured by the field vane were believed to be representative of the "true" shear strength of cohesive soils. However, it was soon discovered that unexpected failures were occurring, involving embankments designed with relatively high factors of safety based on field vane results. Bjerrum (1972) back-calculated a number of these embankment failures and discovered that the vane strength values used in the designs either underestimated or overestimated the mobilized shear strength. By plotting the calculated factors of safety with respect to corresponding plasticity indices (Fig. 1a), he was able to establish, despite a certain scatter, a linear relationship between these two variables. He then proposed a correction coefficient (μ) to be applied to the measured vane strength values (c_u(vane)) as a function of the plasticity index (IP) of the soil (Fig. 1b).

As pointed out by Holtz and Wennerstrand (1972), this idea was not new, and a similar approach was already in use in Sweden for a long time (Broms 1969; Helenelund 1977). Other studies (Pilot 1972; Dascal et al. 1972) also confirmed the validity of Bjerrum's approach, and the method found general acceptance among practicing engineers because of its simplicity.

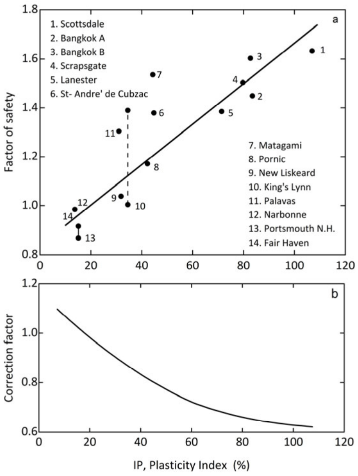

Fig. 1 (a) computed factors of safety versus plasticity indices of foundation clay (after Bjerrum 1972), (b) correction factor to be applied to the vane strength of the clay foundation (after Bjerrum 1972)

2.2 *Contribution by La Rochelle et al. (1974)*

La Rochelle et al. (1974) argued that progressive failure may be the most important factor affecting the shear strength of the clay. They proposed a method that would incorporate this effect, as well as the effect of time and anisotropy under embankment loading by employing shear strength values measured at large strains (about 15% axial strain) in the triaxial apparatus. This method was later renamed by Trak et al. (1980) as the "Undrained Strength At Large Strains" (USALS)" analysis and was shown to yield reliable stability calculations for Champlain clays and other fluvioglacial and lacustrine clays of Eastern Canada.

2.3 *Contribution by Ladd and Foott (1974)*

Ladd and Foott (1974) proposed the SHANSEP (Soil History And Normalized Soil Engineering Properties) method, which is based on the principle that the shear strength behavior of soils is best illustrated in terms of normalized soil parameters, The most common ones are c_u/σ'_{vo}, c_u/σ'_p and σ'_p/σ'_{vo} ratios, where c_u, σ'_{vo} and σ'_p are respectively the undrained shear strength, the effective overburden pressure and the preconsolidation pressure. The method is claimed to account for the effect of sample disturbance and of the rate of loading because of the derivation of the normalized soil parameters by reconsolidation of the clay to overconsolidation ratios greater than one. Ladd and Foott (1974) indicated that the laboratory testing required in the SHANSEP method could result in the collapse of highly structured quick clays and naturally cemented deposits but would be applicable to other clays.

2.4 *Contribution by Aas (1976a and 1976b)*

Aas (1976a and 1976b) introduced the ADP method to evaluate the undrained shear strength of soft Norwegian clays. ADP is an abbreviation for active, direct shear and passive analysis. This method is based on Bjerrum's (1973) proposal that the shear strength along a potential slip surface beneath an embankment can be represented by three types of laboratory tests: the compression, the extension and the direct shear tests.

2.5 *Contribution by Mesri (1975) and Trak et al. (1980)*

By modifying Bjerrum's (1973) diagrams showing relationships between the plasticity index and the $c_u(\text{vane})/\sigma'_{vo}$ and σ'_p/σ'_{vo} ratios (Fig. 2a and Fig. 2b), Mesri (1975) noted that $c_u(\text{vane})/\sigma'_p$ is a single curve for both aged and young clays (Fig. 2c). When Bjerrum's correction coefficient μ is applied to this curve, $\mu c_u(\text{vane})/\sigma'_p$ has a constant value of 0.22, and Mesri (1975) pointed out that the strength available at failure under embankments is therefore nearly independent of the plasticity index IP (Fig. 2d).

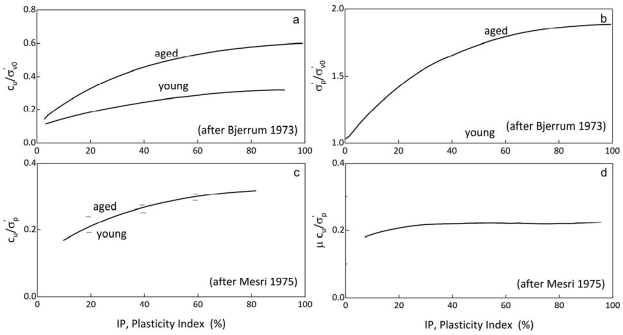

Fig. 2 Typical ratios for normally consolidated late glacial and postglacial clays (after Bjerrum 1973 and Mesri 1975)

The strength available at failure or the mobilized shear strength can therefore be expressed as c_u (mobilized) = 0.22 σ'_p. By analyzing a large number of failure cases, Trak et al. (1980) gave evidence that almost all of the methods proposed to determine the mobilized shear strength of clay deposits, namely the Bjerrum (1972), the SHANSEP (Ladd and Foott 1974) and the USALS (La Rochelle et al. 1974) methods make use of a practically constant strength value given by $c_{u\,(mob)} = 0.22\ \sigma'_p$ relationship. A similar conclusion was also reached by Larsson (1980).

The unique relationship between the mobilized shear strength at failure and the preconsolidation pressure was also confirmed by Chapuis (1982) who stated that "It is our opinion that two clay layers having the same σ'_p (z) profile will fail identically when identically loaded (short term condition), even if they have different c_u (z) profiles that may or may not be the result of different geological origins".

It is interesting to note that the above given expression for $c_{u\,(mob)}$ for overconsolidated clays is very similar to the empirical "rule of thumb" given by Puzrin et al. (2010): $c_u = 0.21\ \sigma'_{vo}$ for normally consolidated clays (Trak 2017).

The unique relationship between the mobilized shear strength and the preconsolidation pressure has a basis in the critical state concept, which was found to be applicable to natural clays (Trak 1980). This implies that a clay loaded up to failure will necessarily achieve its critical state either immediately (perfectly plastic clays) or after some strains (strain softening clays). This critical state strength can be used to characterize the minimum strength available under embankments at failure (Trak et al. 1980), or behind unstable slopes. In a clay deposit characterized by its preconsolidation (σ'_p) profile and the corresponding void ratio (*e*) profile, the critical state strength is a unique function of the depth. It can therefore be used in the same manner as an undrained shear strength parameter in a so called φ=0 analysis. Such an analysis is applicable to all cases where the void ratio of the clay can be assumed to be constant. This is essentially true for embankments built in one stage or the short-term stability of cut slopes or excavations (Trak 2017).

Bjerrum (1973) had also provided two "factor of safety vs IP" plots, one for cuts and unsupported excavations (Fig.3), and another one for footings and loading plates (Fig.4). It can be seen that excavation and bearing capacity failures may be represented by the same regression line as embankment failures. Since the $c_{u\,(mob)} = 0.22\ \sigma'_p$ expression was derived by making use of Bjerrum's correction coefficient (Fig. 1b) for embankments, it appears that the same expression could be used to estimate the mobilized shear strength in excavation and bearing capacity problems (Trak 1981). It was therefore decided to investigate the validity of this approach under other loading and unloading conditions. The case studies include an embankment in Bangkok clay, as well as two silos and one excavation in Ontario, Canada.

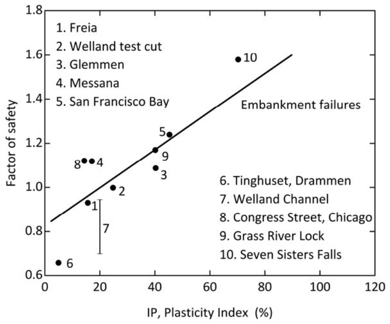

Fig. 3 Theoretical factor of safety at failure for cuts and unsupported excavations plotted against the plasticity index of the clay (after Bjerrum 1973)

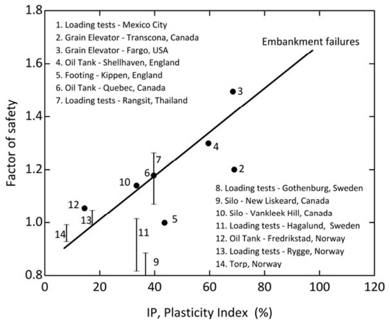

Fig. 4 Theoretical factor of safety at failure of footings and loading plates plotted against the plasticity index of the clay (after Bjerrum 1973)

3 CASE STUDIES

3.1 *Bangpli embankment*

An extensive research program was conducted by Brand et al. (1976) at a 400 x 400 m test site at Bangpli, Thailand. A large number of dutch cone and field vane tests were carried out under a full-scale embankment loaded to failure.

Bangpli is located near the coastline of the Gulf of Thailand, about 25 km southeast of Bangkok. Bangpli, like the city of Bangkok, is underlain by a soft clay deposit of marine origin, known as Bangkok clay. A typical subsoil profile at Bangpli is shown in Fig.5, which also gives field vane strength and preconsolidation pressure values. The test embankment was a compacted sand fill 3.3 m high, 9.2 m wide at the top, with a 1.2 m high and 10 m wide berm. Brand et al. (1976) observed no tension cracks in the fill at failure, and assumed c'=0, φ=37° as its shear strength parameters.

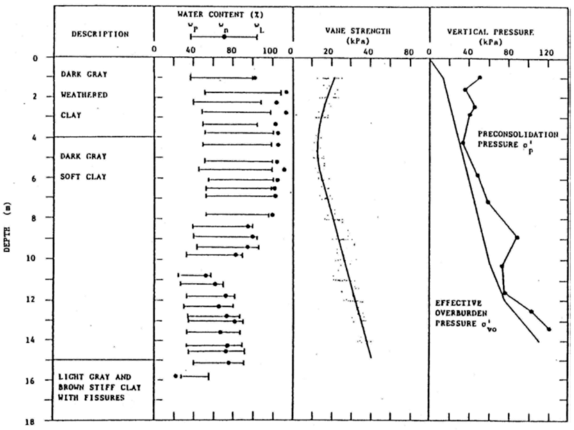

Fig 5. Soil profile at Bangpli (after Brand et al 1976)

Stability analyses performed by Brand et al. (1976) were based mainly on dutch cone measurements. An average field vane strength profile was also used in the analyses. All calculations were performed using the Fellenius' (1927) method, which gave a minimum factor of safety of 1.2 based on field vane results. Stability analyses performed in the present study included additional calculations based on in situ vane strength profile with different crust strength assumptions, and the profile for the relationship $c_{u\,(mob)} = 0.22\ \sigma'_p$ (Fig.6). The results of stability analyses are given in Table 1

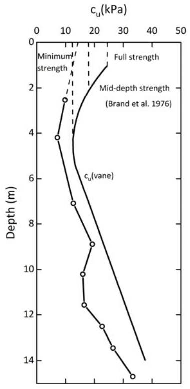

Fig 6. Strength profiles at Bangpli

Table 1. Computed factors of safety of Bangpli embankment – C = Critical circle, F = Observed failure arc, ( ) corrected value based on Bjerrum's (1972) method
0

Circle	Vane strength profile with different assumptions of crust strength			$c_{u\,(mob)} = 0.22\ \sigma'_p$
	Minimum	Mid-depth	Full	
C	*1.16*	*1.39 (0.97)*	1.57 (1.10)	0.93
F	*1.23*	*1.47 (1.03)*	1.66 (1.16)	0.97

Positions of the critical slip circle and the one that fits best the failure surface determined by in situ observations are shown in Fig. 7 for the $c_{u\,(mob)} = 0.22\ \sigma'_p$ profile. Results indicate that factors of safety values close to unity are obtained for the $c_{u\,(mob)} = 0.22\ \sigma'_p$ profile and the corrected vane strength profile, with a mid-depth crust strength assumption.

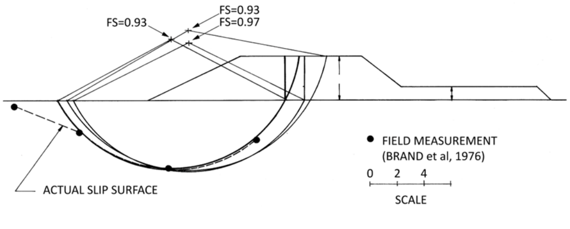

Fig 7. Critical circles corresponding to $c_u = 0.22\ \sigma'_p$, Bangpli embankment

3.2 *New Liskeard silo*

A silo located approximately 6.5 km north of New Liskeard in Northern Ontario, Canada, failed in 1961. The failure was studied by Eden and Bozozuk (1962) and Bozozuk (1977).

New Liskeard is on the edge of the vast pre-glacial Lake Ojibway-Barlow clay plain, deposited during the Pleistocene period. The subsoil consists mainly of varved silty clays. The soil profile in Fig. 8 also shows the field vane strength and preconsolidation pressure values. The silo was built of concrete staves; the height was 15 m and the inside diameter 6 m. Failure occurred one day after the silo was filled to capacity; the weight of the silo at failure was estimated to be 449 tons (grass silage: 314 tons, structures and foundation: 135 tons) (Bozozuk 1977). Analyses were performed by Eden and Bozozuk (1962) using Skempton's (1942 and 1951) and Meyerhof's (1951) bearing capacity equations, and the Fellenius' (1927) method. The shear strength value used in the analyses was the arithmetic mean of all measured vane strength values between the bottom of the footing and a depth below the footing equal to two thirds of its diameter. The average shear strength is 15.6 kPa, and the lowest factor of safety, corresponding to bearing capacity equations, is 0.97, whereas a value of 0.94 is obtained using the Fellenius' (1927) method (Table 2).

Table 2. Computed factors of safety of the New Liskeard silo, (1) taken from Eden & Bozozuk (1962), ( ) corrected values based on Bjerrum's (1972) method, (2) stability method without slices (Fellenius' 1927 method)

Method of analysis	Average vane strength	Vane strength with different crust strength assumptions		$c_u = 0.22\ \sigma'_p$
		Max strength	Min strength	
Bishop's method	*0.94*	*1.31 (1.12)*	1.26 (1.06)	1.02
Circular arc(2)	*0.94(1)*	*1.07 (0.97)*	1.03 (0.88)	0.89
Skempton (1951)	*0.97(1)*	*1.14 (0.97)*	1.10 (0.94)	0.94
Skempton (1942)	*1.10(1)*	*1.16 (0.99)*	1.12 (0.95)	0.96
Meyerhof (1951)	*1.09(1)*	*1.27 (1.08)*	1.23 (1.05)	1.04

Stability analyses performed in the present study included additional calculations based on in situ vane strength profile with different crust strength assumptions, using the Bishop's simplified method (1955), as well as bearing capacity equations. Similar analyses were also carried out using $c_{u\,(mob)} = 0.22\ \sigma'_p$ profile. These profiles are given in Fig. 9.

Results of stability analyses are shown in Table 2. It is seen that the $c_{u\,(mob)} = 0.22\ \sigma'_p$ expression gives in general satisfactory results, except perhaps for the Fellenius' (1927) method. Details of the silo base after failure are given in Fig. 10.

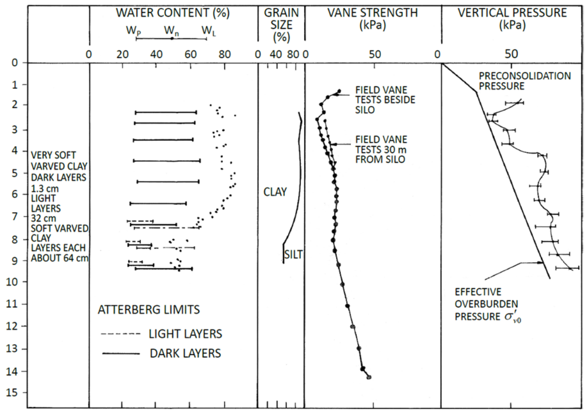

Fig 8. Soil profile at New Liskeard (after Eden & Bozozuk 1962)

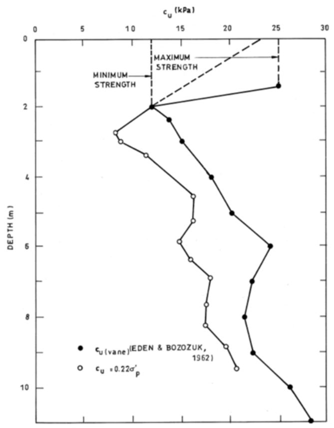

Fig 9. Strength profiles at New Liskeard

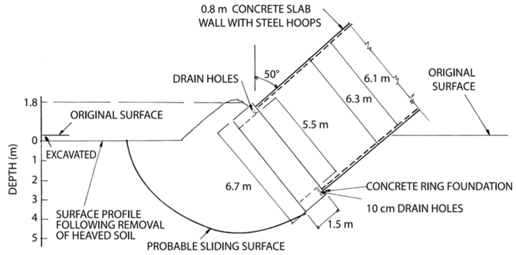

Fig 10. Details of the silo base after failure, New Liskeard (after Eden & Bozozuk 1962)

3.3 *Vankleek Hill silo*

A second silo failure occurred in Vankleek Hill in 1970 and was studied in detail by Bozozuk (1972). Vankleek Hill is located on the marine clay deposit that overlies a major part of the Ottawa-St Lawrence lowlands and is known as the Champlain Sea clay. The soil profile in Fig. 11 gives also field vane strength and preconsolidation pressure profiles. The silo was 21 m high and consisted of a 6 m inside diameter reinforced concrete tube with 16.5 cm thick walls, constructed on a concrete ring foundation.

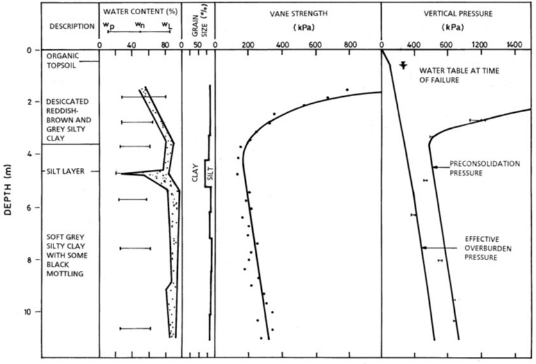

Fig 11. Soil profile at Vankleek Hill (after Bozozuk 1972)

The silo failure was analyzed by Bozozuk (1972) using three different shear strength profiles: 1) the field vane strength; 2) the field vane strength corrected for the inclination of the failure surface by means of laboratory vane tests at various inclinations, and 3) CAU (anisotropically consolidated undrained) strength. According to Bozozuk (1972), the total weight of the structure was estimated to be 672 tons. Factors of safety computed by Bozozuk (1972) using various bearing capacity formulae and average field vane shear strength values are given in Table 3.

Similar analyses were performed in this study, but the field vane strength values were taken as the weighted average along the slip surface determined after failure. In addition to bearing capacity formulae of Skempton (1942, 1951) and Meyerhof (1951), the failure was also analyzed by Bishop's (1955) simplified method. Results of the analyses are shown in Table 3. The same Table also gives the results of stability analyses using the $c_{u\,(mob)} = 0.22\ \sigma'_p$ profile given in Fig. 12. It is seen that, except for Meyerhof's (1951) formula, satisfactory results were obtained by using the $c_{u\,(mob)} = 0.22\ \sigma'_p$ expression to estimate the mobilized shear strength under this shallow foundation loading. Fig. 13 shows the silo foundation after failure.

Table 3. Computed factors of safety of the Vankleek Hill silo, ( ) corrected values based on Bjerrum's (1972) method, (1) stability method without slices, * taken from Bozozuk (1972),

	Vane strength with different crust strength assumptions			Average vane	$c_u = 0.22\ \sigma'_p$
	Maximum	Mid-depth	Minimum		
Bearing capacity equations					
Skempton (1951)	*1.52 (1.31)*	*1.12 (0.96)*	0.80 (0.68)	1.10*(0.95)	1.05
Skempton (1942)	*1.56 (1.34)*	*1.25 (1.08)*	0.82 (0.57)	1.13*(0.97)	1.05
Meyerhof (1951)	*1.71 (1.47)*	*1.37 (1.18)*	0.89 (0.76)	1.24*(1.07)	1.17
Limit equilibrium methods					
Circular arc$^{(1)}$	*1.38 (1.19)*	*1.12 (0.96)*	0.75 (0.64)		0.96
Bishop	*1.59 (1.37)*	*1.24 (1.07)*	0.76 (0.66)		1.05

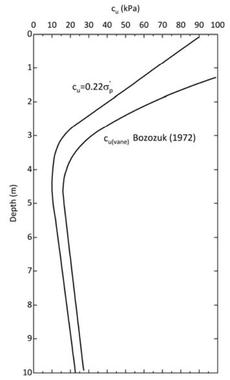

Fig 12. Strength profiles at Vankleek Hill

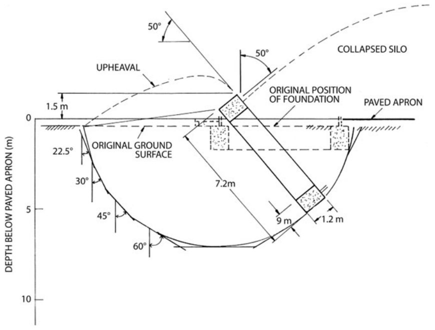

Fig 13. Silo foundation after failure, Vankleek Hill (after Bozozuk 1972)

3.4 *Vertical cut failure at Welland Canal*

A 13.4 km long cut for the realignment of the southern half of the old Welland Canal necessitated the excavation of very large volumes of soil. To ensure the safety of the excavation, a field test to study the behavior of deep, vertical cuts in soft clays was conducted and reported by Kwan (1971).

Welland is located in Ontario, Canada, about 17 km west of Niagara Falls. The Bypass Channel at Welland is 106.7 m wide with a water depth of 9.1 m. The Canal joins Lake Erie to Lake

Ontario. The whole region lies in an area called the Haldimand Clay Plain, which is of lacustrine origin. The soil profile in Fig. 14 also gives the field vane strength and the preconsolidation pressure profiles.

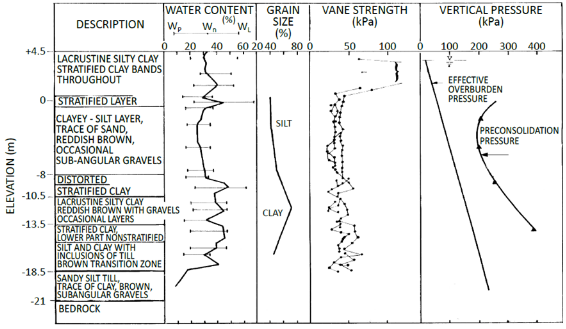

Fig 14. Soil profile at Welland (after Kwan 1971)

The test excavation was carried out in the late winter of 1967 and involved the removal of the top 5.2 m of desiccated layer, followed by instrumenting and excavating the trench. The final stage, when the trench was cut to an average depth of 9.1 m with a length of 15.2 m, took about four days. To ensure no "end effects", two vertical trenches were cut along the side of the instrumented trench. Failure occurred four days after the end of excavation. The final failure surface was located at a distance of about 4.5 m behind the vertical face; the width of the fissure was 1.5 m, and its depth 6.1 m.

Kwan (1971) carried out various back-calculations to analyze the vertical cut failure at Welland Canal, using the critical height ($H=4\ c_u/\gamma$) and conventional slip circle methods. Shear strength values used in these calculations were obtained from triaxial quick tests, and the results are shown in Table 4. Similar stability analyses were conducted in this study using the critical height equation and the simplified Bishop's (1955) method, and the results are also shown in Table 4.

The shear strength profiles were the field vane strength (no correction was necessary because I_p=20% for the Welland clay, Fig. 3), determined by Kwan (1971), and the $c_{u\ (mob)} = 0.22\ \sigma'_p$ (Fig. 15). The factor of safety corresponding to the last expression and the critical height equation was 1.01. The modified critical height equation ($H=3.85\ c_u/\gamma$) yielded a value of 0.98. The same expression was also used with the Bishop's (1955) simplified method and the factor of safety was 0.64 (Table 4).

Table 4. Computed factors of safety of Welland excavation, *tension crack is taken into account

	Triaxial quick test (Kwan 1971)	Field vane shear strength	$c_u = 0.22\ \sigma'_p$
Bishop	*0.81*	*0.61*	0.64*
Critical height method			
$H = 4c_u/\gamma$	*0.8*	*1.04*	1.01
$H = 3.85c_u/\gamma$	*0.77*	*0.95*	0.98

4 DISCUSSION

In the Bangpli test program, a number of dutch cone tests were conducted by Brand et al. (1976), who found that this apparatus overestimated the mobilized shear strength of the Bangkok clay. Further calibration may therefore be necessary when using this in situ testing method in analyzing the stability of embankments on soft clays.

It is interesting to note that in both the New Liskeard and Vankleek silo failures, limit equilibrium methods gave relatively low factors of safety compared to bearing capacity equations. Similarly, in the analysis of the Welland Canal failure, the limit equilibrium method gave very conservative factors of safety, whereas the critical height equations yielded consistently realistic results.

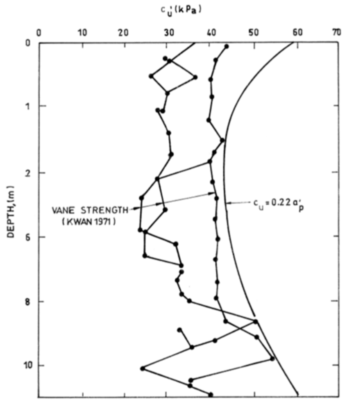

Fig 15. Shear strength profiles of Welland clay

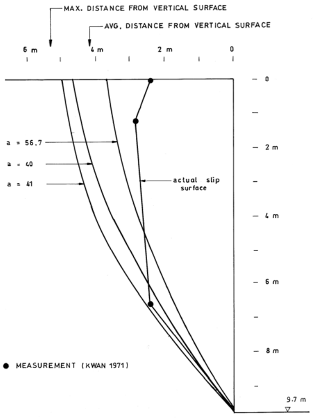

Fig 16. Comparison between actual failure surface and surfaces generated by cycloidal functions

After the failure of the Welland vertical cut, Kwan (1971) observed that the closest distance of the final tension crack was equal to 2.4 m, based on measurements in the northern side trench. According to Terzaghi (1941), the maximum tension behind a vertical cut occurs at a distance equal to half the critical depth. Therefore, in the Welland Canal failure, the location of the crack should have been located at about 4.9 m away from the vertical face of the cut.

Collin (1846) had observed that failure surfaces in most cohesive soils took the shape of a cycloidal curve instead of an inclined plane, as suggested by Coulomb (1773). A numerical solution was derived by Ellis (1973) to analyze the stability of vertical or near vertical slopes in homogeneous and isotropic soils. In the analysis of the Welland Canal failure, potential and actual slip surfaces were compared to curves generated by the following cycloidal function: $x = a\ (\theta - \sin\theta)$, $y = a\ (1 - \cos\theta)$. A certain correspondence was found between the slip surface described by Kwan (1971) and the generated curve (Fig.16). This indicates that the use of cycloidal arcs as potential slip surfaces in vertical cuts could be envisaged in stability analyses.

5 CONCLUSIONS

Several case histories dealing with failures under various loading and unloading conditions in soft clay deposits were reanalyzed using shear strength values determined by conventional field vane tests, and an empirical expression derived by Mesri (1975).

The Bangpli embankment failure analysis indicated that the $c_{u\,(mob)} = 0.22\ \sigma'_p$ expression provided a satisfactory measure of the mobilized shear strength in a Bangkok clay deposit. Similarly, both the New Liskeard and Vankleek Hill silo failures provided a good opportunity to investigate the applicability of the $c_{u\,(mob)} = 0.22\ \sigma'_p$ method to shallow foundation loading problems. Results indicate that this empirical approach leads to realistic value of the mobilized shear strength under such loading conditions.

Using the mobilized shear strength profile, it must be recognized that this expression gives the minimum value of the available strength of the foundation clay, achieved at complete failure. It would then be logical to proceed in the following manner when designing an embankment over a soft clay deposit:

a) Establish as accurately as possible the σ'_p profile of the clay foundation. This will require careful oedometer testing on undisturbed clay samples.
b) Derive values of the mobilized shear strength corresponding to the complete failure condition using the $c_{u\,(mob)} = 0.22\ \sigma'_p$ expression (for slightly overconsolidated clays). The $c_u = 0.21\ \sigma'_{vo}$ expression may be used for normally consolidated clays.
c) Carry out the $\varphi=0$ stability analysis to determine the embankment height H_f corresponding to failure condition (F=1.0).
d) Determine the embankment height H corresponding to the desired performance level, conserving the same geometry. If there is a need to define a safety factor, it can be defined as $F=H_f/H$, since the geometry is the same (Trak 2017).

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