Evaluating Strength Tests from Foundation Failures

Rèsistance au Cisaillement-Evaluation des Essais

M.BOZOZUK Geotechnical Section, Division of Building Research, National Research Council of Canada, Canada

SYNOPSIS Three bearing capacity failures of tower silos have been analysed using in situ vane and triaxial strength tests on thin-wall piston tube samples obtained from the foundation clays. The investigations were carried out in Ontario, Canada, on a 15-m high silo on varved clay at New Liskeard, on a 21-m high silo on near-normally consolidated marine clay at Vankleek Hill, and on a 32-m high silo on overconsolidated marine clay near Richmond. Case records permitted evaluation of these test methods for measuring shear strength of the foundation soils.

Analyses showed that the NGI vane measured the average shear strengths adequately in situ and that these strengths can be used to predict ultimate bearing capacity. In comparison, triaxial strength tests underestimated the vane test strengths in the varved clays and in the fissured or desiccated marine clay. Below the desiccated soil the triaxial peak strengths exceeded the vane strengths, but the post-peak values were considerably lower. The unconfined tests consistently underestimated the in situ measurements. The reasons for the apparent satisfactory performance of the vane for predicting ultimate bearing capacity are discussed.

INTRODUCTION

For soil to support a structure, a shearing resistance equal to the applied stresses must be mobilized within the soil mass. The maximum resistance that can be mobilized at any point within the mass is equal to the peak shear strength. Both the peak strength and the strain at which it occurs usually vary from point to point and with direction of applied shear stress. To mobilize the required shearing resistance, therefore, differing strains must develop in progression along the critical plane. The soil may then be stressed to post-peak, peak, and pre-peak values, depending upon the magnitude of the applied load and location along the critical shear plane. Consequently, it is impossible to develop the peak shearing resistance simultaneously along the entire shearing surface. The ultimate bearing capacity of the soil is not related to the peak strength but to some "average" strength intermediate between the peak and post-peak or residual strength of the soil.

In analysing three bearing capacity failures of tower silos constructed on different clay soils in Ontario, shear strengths were measured in situ with a 55 x 110 mm NGI vane (Anderson and Bjerrum, 1956) and in the laboratory on undisturbed soil samples obtained with a 54-mm dia thin-wall piston tube sampler (Bjerrum 1954). These test results were used to predict ultimate bearing capacity, which could then be compared with the known bearing capacity of the soil at failure. The field vane test, severely criticized by Schmertmann (1975), LaRochelle et al. (1973, 1974) and Roy (1975), has provided the most reasonable value of in situ "average" shear strength for determining the ultimate bearing capacity of the soils. This paper compares the results of these strength tests and discusses the reasons for the apparent satisfactory performance of the vane.

NEW LISKEARD

In July 1961 a 15.24-m high, 6.10-m dia precast concrete stave silo 6 km northeast of New Liskeard had just been filled for the first time with hay silage when it failed suddenly as a result of a bearing capacity failure (Eden and Bozozuk, 1962). The results of field and laboratory tests on soils from the site are summarized in Figure 1. The soil profile (Figure 1a) showed a weathered crust to a depth of 1.5 m, underlain by layers of soft, sensitive brown clays and grey varved clays to a depth of 6.7 m. Below this the sample boring traversed grey varved clays to a depth of 14 m. These clays are deposits of glacial Lake Barlow.

Classification tests (Figure 1b) on the soil at depths from 2 to 6 m show a plasticity index of about 40 per cent, a liquid limit of about 60 per cent, and a natural water content of about 80 per cent. These values decreased below 7 m.

The shear strengths shown on Figure 1c are from in-situ vane tests, laboratory unconfined undrained (UU), and consolidated isotropically undrained (CIU) triaxial strength tests on undisturbed soil samples. The UU and CIU tests were performed at strain rates of 1 per cent per minute and 2 per cent per hour, respectively. The confining pressures for the CIU tests were made equal to the effective overburden pressure. Proving rings were used to measure the applied axial loads.

The estimated total weight of the silo and its contents when filled was 449 tonnes (hay silage 314 tonnes, structure and foundation 135 tonnes). Using the equation for ultimate bearing capacity (Skempton, 1951):

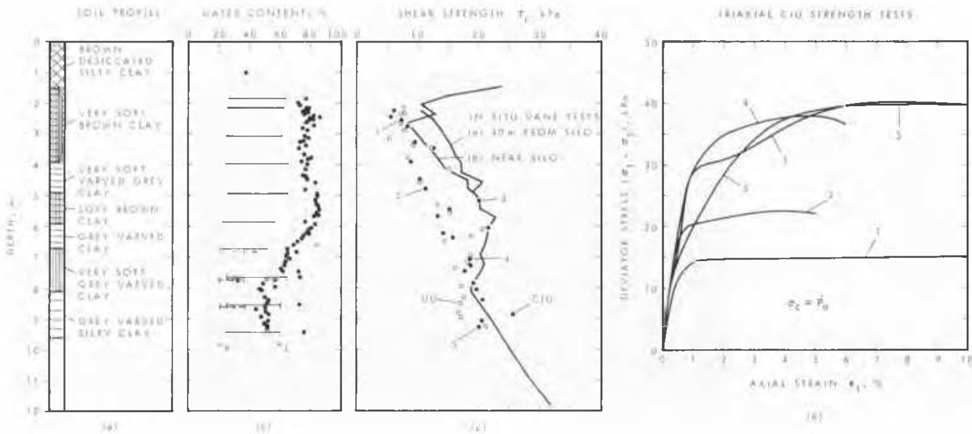


Figure 1 Engineering Properties of Soil at New Liskeard

$$q_u = cN_c + P$$

where q_u = ultimate bearing capacity of the soil

c = average shear strength of the soil to a depth below the foundation equal to 2/3 of the outside diameter of the foundation

N_c = shape factor

P = overburden pressure above the base of the foundation

and using also the average shear strength of the soil measured in situ with the vane, the ultimate bearing capacity of the soil was determined to be 447 tonnes. This gave a factor of safety of 1.00 at failure.

The two laboratory strength test methods produced shear strengths considerably below those measured in situ with the vane. Using these results the factor of safety at failure was about 0.8; that is, the ultimate bearing capacity would have been considerably underestimated. The reason for the low strengths may

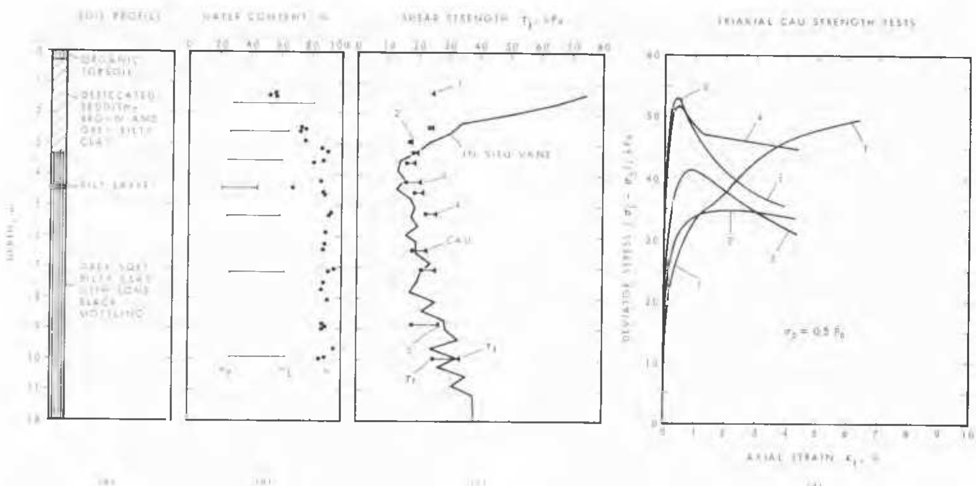


Figure 2 Engineering Properties of Soil at Vankleek Hill

be deduced from a study of the CIU stress-strain curves (Figure 1d). The deviator stress increased rapidly to a maximum value then levelled off. The absence of sharp peaks from the stress-strain curves may be due to disturbance from sampling and transit to the laboratory. Consequently these tests did not measure the peak strength of the soil but produced a measure of the post-peak residual strength.

VANKLEEK HILL

On 30 September 1970 a 21.34-m high, 6.10-m dia concrete tower silo about 5 km south of Vankleek Hill overturned suddenly due to a bearing capacity failure just after it had been filled for the first time with 492 tonnes of corn silage (Bozozuk, 1972). The soils are near-normally consolidated marine deposits of the Champlain Sea.

The soil profile (Figure 2a) shows a desiccated silty clay to a depth of 3.3 m overlying grey, soft, silty clay with some black mottling. This extends to a depth of at least 20 m.

Classification tests (Figure 2b) indicated a liquid limit of 82 per cent near the base of the silo footings, decreasing to about 60 per cent below a depth of 3 m. The average plasticity index was about 36 per cent. Natural water content was about 55 per cent at foundation level and increased with depth to almost 95 per cent, exceeding the liquid limit at each level by nearly 30 per cent. Grain size analysis indicated that 85 to 90 per cent of the soil particles were clay size.

In situ vane shear tests and laboratory triaxial CAU tests performed on undisturbed soil samples are plotted on Figure 2c. The CAU tests were performed at a strain rate of 2 per cent per hour, and the axial loads were measured with an electric force transducer. Consolidation pressures for these tests were $\sigma_1 = P_o'$, the cell pressure $\sigma_c = K_o P_o'$ where P_o' is the effective overburden pressure, and $K_o = 0.5$.

At the time of failure the combined weight of the structure, silage and foundations was 672 tonnes. Using the average in situ shear strength of the soil measured with the vane the estimated factor of safety was 1.10. Additional tests performed with a small vane on undisturbed soil samples at pre-selected inclinations from the vertical provided corrections for the slope of the failure surface (Bozozuk, 1972). When applied to the in situ field vane strengths, the factor of safety against a bearing capacity failure was reduced to 0.99. This correction agreed with Bjerrum's (1972) factor based on the plasticity index of the soil.

CAU triaxial strengths are compared with in situ vane shear strengths on Figure 2c. In the desiccated crust the CAU values fall below those measured in situ with the vane. The stress-strain curves in this soil formation (curves 1 and 2 in Figure 2d) do not display a sharp peak but level off at the maximum deviator stress, giving some measure of the "residual" strength of this soil.

In the normally-consolidated silty clay below the desiccated formation, the CAU shear strengths varied from 100 to 130 per cent of the in situ vane shear strengths. The stress-strain curves (curves 3, 4, 5 in Figure 2d) display a sharp peak at failure, followed by a rapid drop-off with increasing axial strain. Unfortunately the tests were not followed through to large strains when they were performed and consequently the minimum post-peak strengths τ_r were not determined. The trend in Figure 2c, however, indicates that τ_r could be smaller than the vane shear strengths.

RICHMOND ROAD

On 30 September 1975 a 32.3-m high, 9.14-m dia tower silo with 152-mm thick concrete walls overturned during filling with corn silage when the load reached 1797 tonnes. It was supported on a circular concrete ring foundation 61 mm thick, with outside and inside diameters of 11.89 and 7.62 m, respectively. The foundation contained no steel reinforcing. The total weight of the structure, its contents and foundation at the time of failure was about 2258 tonnes.

The silo was erected about 7 km north of Richmond on a clay plane of overconsolidated marine deposits of the Champlain Sea. The soil profile (Figure 3a) shows brown desiccated clayey silt to a depth of 2.4 m, changing to grey clayey silt to 3.1 m, and underlain by grey silty clays with black mottling. At 15.5 m this changed to a grey, brittle, silty clay.

The classification tests shown on Figure 3b indicate that the plasticity index and liquid limits were about constant for the soil profile at 20 and 40 per cent respectively. The natural water content was about equal to the liquid limits at a depth of 2 m, but it gradually increased to 60 per cent below a depth of 9 m. Grain size analysis indicated 37 per cent clays and 63 per cent silts at 2 m, changing almost linearly with depth to 58 per cent clay and 42 per cent silt at a depth of 16 m.

The shear strength of the soil plotted on Figure 3c was measured in situ with the vane, and in the laboratory with unconfined undrained (UU) and consolidated isotropically undrained (CIU) triaxial strength tests on undisturbed soil samples. The UU and CIU tests were loaded at constant rates of strain of 1 per cent per minute and 1 per cent per hour, respectively. A cell pressure of $\sigma_c = K_o P_o'$ where $K_o = 0.75$ was used in the CIU tests.

The ultimate bearing capacity of the soil for the silo, based on in situ average vane shear strength, was 241 kPa. The factor of safety at the time of failure was difficult to calculate because the non-reinforced concrete footing had cracked during filling of the silo, reducing the contact area for distribution of the load to the foundation soil. For the original contact area of 111.0 m^2, the factor of safety was 1.2; for a contact area equal to that of the 9.14-m dia tower, i.e., 70.1 m^2, it was 0.8. A factor of safety of 1.00 would have been obtained if the actual contact area at the time of failure had been 91 m^2. Consequently, it is reasonable to assume that the ultimate bearing capacity of the soil was adequately determined using the average shear strength measured in situ with the vane.

CIU and UU strength test results are compared with the in situ measurements on Figure 3c. In the desiccated crust the CIU values were lower than those measured in situ and the triaxial tests curves (1 and 2 , Figure 3d) do not display a sharp peak. Below the crust the triaxial test curves (3, 4, 5) developed a sharp peak at failure. The measured maximum strengths generally varied from 100 to 130 per cent of

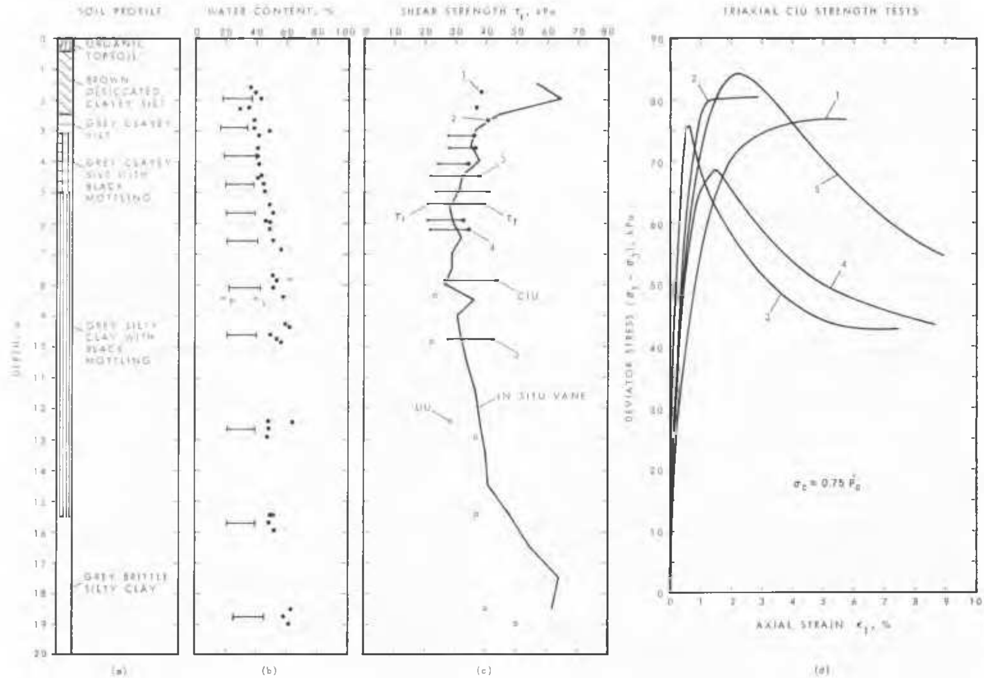
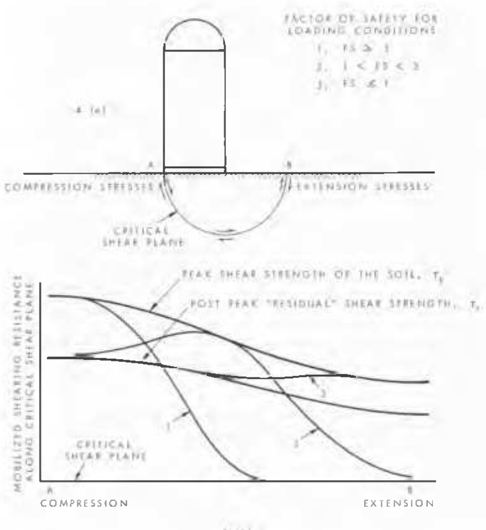


Figure 3 Engineering Properties of Soil at Richmond



in situ values. The post-peak strengths, τ_r, measured at large axial strains were generally 70 to 90 per cent of the vane strengths. The maximum unconfined (UU) strengths measured on undisturbed soil samples from below a depth of 8 m were consistently lower that the field vane strengths.

DISCUSSION

The bearing capacity of a soil required to support a structure (Figure 4a) is related to the shearing resistance that can be mobilized along a critical shear plane. The shearing resistance must equal the sum of the applied shear stresses, which vary with direction along the circle of shear. Beneath the structure the applied stresses are in compression, and at the other end of the critical circle they are in extension. Thus the peak shear strength of the soil, τ_f, along the critical plane may vary as shown on Figure 4b.

To mobilize shearing resistance, a shear strain (which is a function of applied stress) is required. Applied stresses are maximum beneath the structure, shear strains are therefore also greater. As the applied

Figure 4 Shearing Resistance Mobilized Along Critical Shear Plane at Various Factors of Safety for Bearing Capacity loads increase, the strains progress non-uniformly along the critical plane, so that when the factor of safety against a bearing capacity failure is $\geqslant 3$, the mobilized shearing resistance along the plane may be represented by curve 1 (Figure 4b); when $1 < FS < 3$, by curve 2; and when $FS \leqslant 1$, by curve 3. Because of the progressive development of the strains that must take place, it is impossible to mobilize τ_f simultaneously along the whole critical plane. The minimum shearing resistance that can be mobilized is the post-peak "residual" shear strength, τ_r. Hence, the ultimate bearing capacity of the soil must be related to some "average" shear strength that is less than τ_f and greater than τ_r.

When a vane is inserted in the ground it displaces some of the soil. In so doing it develops shear strains that disturb the soil and affect its strength (Law et al*). Furthermore, at the start of the test when a torque is first applied, the shear stresses are greatest just ahead of the blades and zero immediately behind them. The shear strains, therefore, are not uniform around the cylindrical shearing surface. As the vane is rotated further, shear strains and, hence, shearing resistance is mobilized progressively; part of the soil may be at post-peak, peak and pre-peak shear stresses. The maximum shear strength that can therefore be measured by the vane is some "intermediate" value between the peak and post-peak strengths. These "intermediate" shear strengths, used with Skempton's (1951) bearing capacity equation, correlated well with the observed ultimate bearing capacity for the failed silos.

Triaxial strength tests on both varved clays and desiccated clays gave soil strengths smaller than those measured in situ with the vane. The stress-strain curves had no sharp peak, probably because the soils were disturbed during sampling. The strengths determined from these tests appear to be indicative of the residual values.

The peak strengths of the soils below the desiccated crust, determined from CIU and CAU tests, were higher than the in situ vane test strengths. These strengths, however, would not normally correlate with the ultimate bearing capacity of the soil at failure because they can never be simultaneously mobilized along the critical shearing surface. If peak strengths are to be used for design purposes, the bearing capacity theories should be refined to accommodate them. The maximum unconfined test strengths were considerably lower than in situ values and hence provide very conservative estimates of ultimate bearing capacity of these clay soils.

CONCLUSIONS

Analysis of three bearing capacity failures on different clay soils has led to the following conclusions:

1. The ultimate bearing capacity of the clay soils agreed with the calculated value based on "average" strength as measured in situ with the NGI field vane.

2. The field vane does not measure the peak strength but rather some intermediate value between the peak and post-peak strength of the soil.

3. Peak strength cannot be used to predict ultimate bearing capacity with existing theories because it cannot be simultaneously mobilized along the entire critical shearing surface.

4. The unconfined strength test consistently underestimated in situ shear strength of the soil. These measurements would provide very conservative estimates of the ultimate bearing capacity.

5. Bearing capacity is related to some average strength that is less than the peak and greater than the residual value.

* Measured Strengths under Fills on Sensitive Clay. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, 11-15 July 1977.

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