Failure and analysis of a concrete silo

Analyse de la rupture d'un silo en béton

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SYNOPSIS: A concrete corn silo, founded in a desiccated stiff clay deposit, failed upon first filling. A detailed soil investigation in the field and in the laboratory followed. This paper reports on the events leading to the failure, the extensive in situ and laboratory tests which were carried out and on the subsequent bearing capacity analysis considering the varying strength parameters of the soil deposit.

1 INTRODUCTION

Bearing capacity failures in clay deposits are not uncommon. Failure may occur due to overstressing of the underlaying soil or by disintegration of the foundation. In many cases excessive settlement, total and differential, may also make the structure unserviceable. Failure due to soil shear or an inadequate foundation will cause a rather sudden collapse of the structure. The poor performance of several silos founded on clay deposits has led to several investigations e.g. Bauer & Tanaka 1988, Bozozuk 1977 and Tanaka 1986. Tower silos are large structures and are made generally of steel of reinforced concrete sections erected at the site from the roof down. The roof is raised by hydraulic jacks as the new ring sections are added underneath. For many of these structures, the companies supplying and erecting the silo, require the owner to provide the foundation. This practice has led to many problems in the future performance, inasmuch as most of the owners are farmers who built the concrete raft with little or no investigation of the subsoil.

This paper reports on the failure of such a concrete tower silo. The tall silo overturned due to soil shear when it was filled for the first time. An extensive field investigation was carried out to determine the soil properties and the extent of the failure surface. In situ tests were performed with such devices as pressuremeter, dilatometer, shear vane, static cone, borehole shear apparatus and screw plate. The results from these tests are presented, evaluated and discussed with regards to shear strength. Several theories were employed in which the layered soil deposit was considered. A slope "stability analysis" was also carried out and the result was compared to the corresponding values from the bearing capacity analyses and to the strength values derived from the back analysis of the failed silo.

2 SOIL CONDITIONS

The clay deposit, in which the silo was founded at a depth of 0.6m, is generally known as Leda clay or Champlain Sea clay. The soil is characterized by its high water content, soft consistency and its extremely sensitive nature.

This marine deposit, which covers large portions of Eastern Canada, can exhibit high shear strength due to desiccation. Due to post-depositional changes, such as drying, weathering, groundwater fluctuations and removal of overburden, the upper portion of this deposit can be stiff, overconsolidated and highly fissured. This desiccated crust exhibits a low water content and a high shear strength. Therefore, it is advantageous to found a structure as close as possible to the surface in order to make use of this strength. At the site of this study, near Ottawa in Canada, the thickness of this stiff crust was between 3 and 4 metres. Below the crust, the soil strength is quite low and the natural water content is above its liquid limit. At the surface, the overconsolidation ration (OCR) was more than 4, whereas at a depth of 4.5m, this ratio was only 1.2 (Bauer, Scott and Shields 1973 and Eden and Crawford 1957). For soil samples within the crust, specimens of soil were tested in unconfined-undrained (UU) and in isotropically consolidated-undrained (CU) triaxial compression tests.

3 EVENTS LEADING TO THE COLLAPSE OF THE SILO

The silo sat empty for several months before it was observed that when the silo was three-quarters full with green corn silage, the concrete ring foundation started to crack. Filling continued and the cracks widened. The cracked portions of the foundation heaved upwards and the silo started to tilt and filling was stopped. The tilting increased and the silo collapsed quite suddenly. As the base rotated due to a circular bearing capacity failure, the extreme edges of the concrete foundation settled and heaved by 6m and 1.5m respectively, as indicated in Figure 1.

4 LABORATORY AND FIELD TESTS

A detailed description of the equipment and the testing procedure was given by Tanaka (1986) and only a few comments will be given here for the cases where the test procedures used differed from those normally adopted or recommended.

SPT: This test (carried out every 0.3m) is not very meaningfull in clay, but was used in

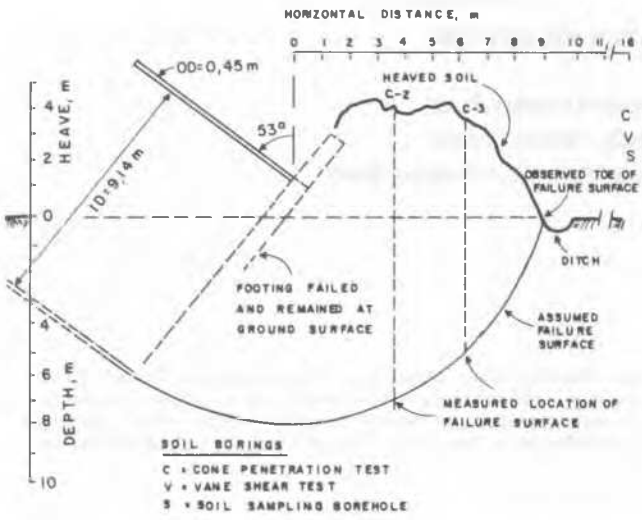

FIGURE 1 : FAILED CONCRETE SILO

the stiff crust to retrieve disturbed samples and to verify the soil profile.

CPT: High and low capacity electric cones were pushed through the stiff crust and the soft underlying soil respectively. The cones were of standard dimensions with apex angles of 60 degrees and a cross-section area of $10cm^2$. They could not measure any porewater pressure.

SVT: The shear vane was a standard tapered vane (45 degrees), having a 76mm diameter.

SCT: Several sizes of screw plates were employed up to 300mm in diameter. Due to the extremely stiff crust and the limitations of the drill, a 76mm helicoidal plate was used. Testing procedure was according to Selvadurai et al (1980), and carried out at every 0.5m.

DMT: This was the first time, according to the knowledge of the authors, that this device was used in highly fissured clay (at every 0.2m) underlain by soft saturated clay. The equipment, testing procedure and method of analysis have been widely described in literature, e.g., Marchetti (1980).

PMT: This mono-cellular pressuremeter was developed by Briaud & Shields (1979) and the testing procedure and analysis of results given by various researches were adopted. The only exception was that the hole was prepared by pushing a thin-walled shelby tube into the soil having the same diameter of 39mm as the probe.

BST: The borehole shear apparatus consists of an expandable probe which is lowered into a predrilled hole. The shear plates of the apparatus were pushed into the wall of the hole and the probe was pulled vertically. From the known expansion pressure, the pulling force and displacement, the shear strength parameters were obtained for every 0.3m.

5 BEARING CAPACITY OF LAYERED SOILS

Most foundations for tower silos are located at shallow depths and for uniform soil deposits their bearing capacity can be calculated readily by conventional theories such as proposed by Terzaghi, Meyerhof or Brinch-Hansen, for example. For stratified or layered clays, there are also several methods available. In this study three methods of analysis were employed and the objective was to back-calculate the applied silo load which caused soil rupture using the parameters derived from the field and laboratory tests:

1. q (ult) = CiNm + q .....(Vesic, 1975)
   where:
   Cl = the undrained shear strength of the upper layer;
   Nm = the modified bearing capacity factor which depends on the strength ratio of the layers, r = C2/Cl, the relative thickness of the upper layer as well as the foundation shape.

Brown and Meyerhof as discussed by Vesic (1975) proposed a slight modification to Vesic bearing capacity factor Nm for the case of a stiff clay stratum overlaying a softer layer. A computer program incorporating this modification, was used in this analysis for various combinations of soil parameters.

Reddy and Srinivasan as given by Vesic (1975) presented a solution for a two-layered clay system also considering the anisotropic characteristics of the layer underlying the foundations. Reddy and Srinivasan's solution is given as:

2. q (ult) = ClNc' (1 + Sc' + dc') + qNq ....(Reddy & Srinivasan as given by Vesic,1975)
   where:
   Nc' = bearing capacity factor considering the anisotropy of the bearing soil;
   Sc' and dc' = shape and depth factors respectively.

3. The problem was analysed by assuming circular slip surfaces and dividing the failed soil mass into a number of vertical slices. In this analysis, Bishop's simplified method of slices was employed. Sounding in the field had shown that the failure surface of the silo foundation could indeed be approximated by a circular arc (Figure 1). In order to accomplish a comparison between this two-dimensional analysis and other methods, the circular raft was transformed into a strip footing of equal width.

6 PRESENTATION AND DISCUSSION OF RESULTS

The undrained shear strength profiles obtained from the various in situ tests as discussed in the previous section, are shown in Figure 2. Each point in this figure represents the average of several tests. It should be realized that a direct comparison between strength values derived from the various in situ tests and a prototype foundation failure suffers from serious restrictions. First and foremost is the effect of scale and of second importance is the mode of strain to which the soil is subjected in the different testing techniques.

The back calculated shear strength from the silo failure is also indicated in Figure 2. The shear strength estimated from the actual failure of the structure represents an "average" value for the failed zone. The failure surface was determined from cone tests as shown in Figure 3. The problem was then analysed as a two-layer soil system knowing the extent of the shear surface and the applied load at failure. The back calculated strength of the crust was quite sensitive to the value of the shear strength which was chosen for the soft clay. Despite this interdependence of strength for these two layers, the crust strength estimated from silo failure was

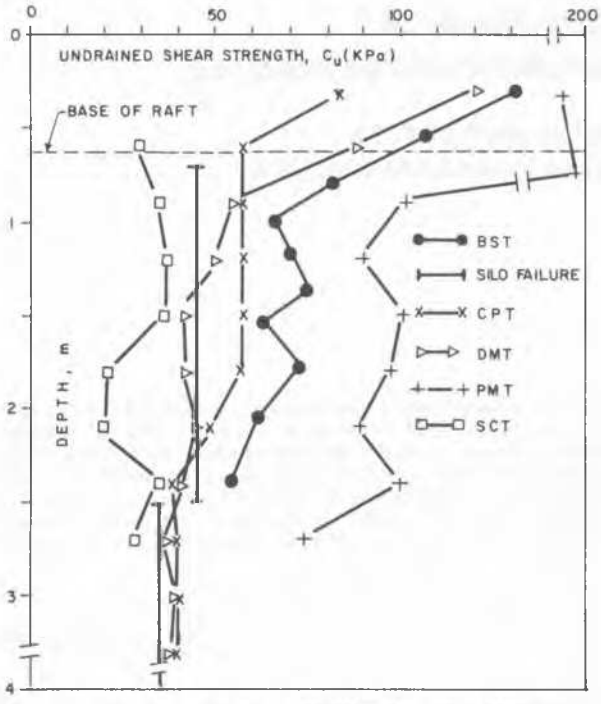

FIGURE 2 : IN-SITU SHEAR STRENGTH VS. DEPTH

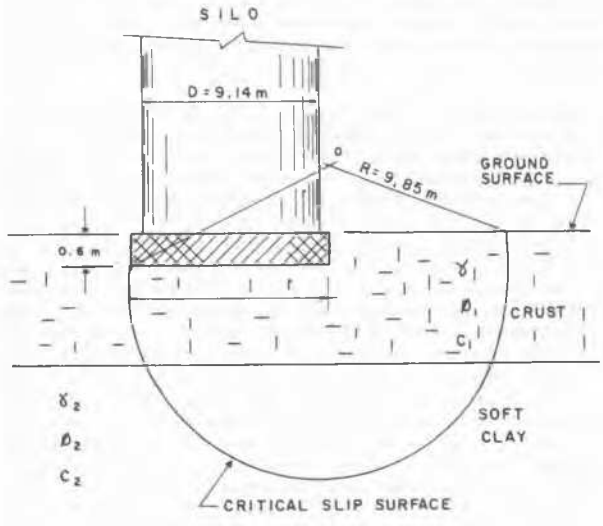

FIGURE 3 : SECTION OF SILO AND SOIL PARAMETERS

45 kPa for a strength value of 35 kPa for the soft clay.

Circular Raft: The bearing capacity was calculated assuming a rough circular footing located at a depth of 0.6m. For the softer clay, two values for the undrained shear strength were used, 35 kPa and 45 kPa. Table 1 summarizes the results of this analysis.

The two methods show reasonable agreement with each other and with the observed failure pressure

Table 1. Comparison of ultimate bearing capacity for circular raft in layered clay. Numbers in brackets are for a strip footing in layered clay.

Method	Shear Strength C1 in kPa	Ultimate Bearing Capacity C2=35,kPa	C2=45,kPa
Vesic	40	251.5(230.8)	- (282.2)
	50	257.9(234.0)	319.3(285.4)
Ø = 0	60	264.4(227.2)	325.6(287.6)
	70	270.6(240.3)	332.0(291.8)
	80	276.3(243.5)	338.3(294.9)
Ø = 5	40	318.6(314.1)	- (367.6)
	50	325.0(317.3)	405.6(370.8)
	60	331.1(320.5)	411.9(373.9)
	70	337.7(323.6)	418.3(376.1)
	80	344.1(326.8)	424.7(380.3)
Reddy &	40	253.3(294.6)	343.1(277.4)
Srinivasan	50	261.0(229.3)	357.3(281.3)
	60	267.6(235.4)	370.0(284.4)
r = 1.0	70	280.9(244.7)	375.7(295.3)
Ø = 0	80	287.1(249.1)	381.6(299.6)
r = 1.2	40	281.6	362.5
	50	288.7	372.6
Ø = 0	60	294.4	391.7
	70	307.1	407.1
	80	318.5	421.4

of 300 kPa. The results based on Reddy and Srinivasan's theory allow for anisotropy in the bearing layer. Increasing the anisotropy by 20 percent increased the calculated bearing value by approximately 10 per cent. It was felt that anisotropy in the relatively thin crust layer had little effect on the overall bearing capacity since the major portion of the shearing surfaces was located in softer isotropic clay. Scrutinizing the results in Table 1, it can be concluded that the average mobilized shear strengths in the lower and upper clay layers were in the order of 40 kPa and 60 kPa respectively. There is also the possibility that some drainage might have taken place along the fissures of the crust material as the silo turned over.

Strip Foundation: In a second analysis the problem was analysed as a strip footing of width equal to the diameter of the silo raft. This had an advantage that the shear strength could be varied with depth as indicated by Figure 3. The strength was decreased linearly from 80 kPa at the base of the footing to 35 kPa at a depth of 2.5m. Below that depth, the strength was assumed to be constant as 35 kPa. The uniformly applied constant stress was increased until the analysis yielded a factor of safety of unity against failure. The most critical slip circle had a radius of 9.85m and cut the ground surface at the edge of the footing. The critical contact stress was 276.4 kPa. Considering an internal friction angle of 5 degrees for the crust yielded a critical circle of 10.4m and a failure contact stress 288.1 kPa. The results of the other second analysis are indicated in Table 1.

7 CONCLUSIONS

The various in situ testing techniques employed in this study were able to provide qualitative and quantitative information for the variation of shear strength of the crust and the underlying soft clay, despite subjecting the soil to differ-

ent loading and deformation conditions, strain rates, etc. Only two devices, the shear vane and the borehole shear apparatus, are able to measure the strength of a soil directly. The cone test depends on a cone factor which must be obtained from other test techniques. The pressuremeter overpredicted the shear strength compared to the results from other devices and compared to the back calculated strength from the failed foundation.

The three methods chosen to predict the bearing capacity at failure, yielded values of reasonable agreement with one another and with the estimated bearing stress of 300 kPa at failure of the silo.

The method of slices is quite versatile inasmuch as a multi-layered soil system could be considered, whereas the other two methods, e.g., Vesic and Reddy and Srinivasan, can accommodate a two-layered soil only. The analysis comparing the results of these three methods yielded a reasonable agreement and it can be concluded that either theoretical analysis is satisfactory.

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REFERENCES

Bauer, G.E., Scott, J.D. and Shields, P.H. (1973). The deformation properties of a clay crust. Proc. 8th ICSMFE, Moscow,Vol.1, pp. 103-112.

Bauer, G.E. and Tanaka, A. (1988). Foundation performance of two tower silos based on in situ tests. Simpósio Sobre Novos Conceitos em Ensaios de Campo e Laboratório, Rio de Janeiro, Vol. 2, pp. 537-548.

Bozozuk, M. (1977). Evaluation strength tests from foundation failures. Proc. 9th ICSMFE, Tokyo, Vol. 1, pp. 55-59.

Briaud, J.L. and Shields, D.H. (1979). A special pressuremeter and pressuremeter test for pavement evaluation. Geot. Testing J., ASTM, Vol. 2, No. 3, pp. 143-151.

Eden, W.J. and Crawford, C.B. (1957). Geotechnical properties of Leda clay in the Ottawa area. Proc. 4th ICSMFE, London, Vol. 1, pp. 22-27.

Marchetti, S. (1980). In situ test by flat dilatometer, J.G.E.D., ASCE, Vol. 106, pp. 299-321.

Selvadurai, A.P.S., Bauer, G.E. and Nicholas, J.J. (1980). Screw plate testing of a soft clay, C.G.J., Vol. 17, No. 4, pp. 465-472.

Tanaka, A. (1986). In situ measurement of shear strength and deformation characteristics of a fissured clay soil.Ph.D.Thesis, Civil Engrg., Carleton University, Ottawa, Canada.

Vesic, A.S., (1975). Bearing capacity of shallow foundations, Ch. 3, Foundation Engineering Handbook, Eds. Winterkorn, H.F. and Fang, H.Y., Van Nostrand Reinhold Co., New York.