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FOUNDATION PERFORMANCE OF A STEEL TOWER SILO AT RICHMOND, ONTARIO

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SOMMAIRE

Les auteurs étudient le comportement d'une dalle circulaire en béton supportant une tour d'ensilage en acier construite dans un dépôt d'argile molle, afin de déterminer la possibilité d'appliquer des méthodes de fondations conventionnelles à ce genre de bâtiment. La capacité portante admissible du sol déterminée sur place au scissomètre est plus qu'adéquate pour assurer un comportement satisfaisant des fondations. La valeur observée du tassement indiquent que la compression maximale du sol se produit directement sous les fondations malgré la surconsolidation élevée du sol sous l'effet de la dessiccation. L'action du gel s'étant fait sentir, les auteurs étudient la nécessité de prévoir une protection contre le gel lorsque la semelle d'un silo s'étend audelà de ses rives, à peu de profondeur.

FOUNDATION PERFORMANCE OF A STEEL TOWER SILO

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The foundation performance of a steel tower silo founded on a concrete circular raft in a deposit of soft clay was monitored in order to determine the applicability of standard engineering foundation methods to this type of structure. The allowable bearing capacity of the soil determined from field vane shear tests was more than adequate for the satisfactory performance of the foundation. The measured settlement of the silo was about one third of that predicted. The settlement gauges showed that the maximum soil compression occurred immediately below the foundation although the soil was highly overconsolidated by desiccation. Because frost action occurred, the need to design for frost protection when silo footings extend beyond the silo wall at a shallow depth is discussed.

INTRODUCTION

On 30 Sept. 1975 a new concrete tower silo 32.3 m high, 9.14 m in diameter, overturned when it was being filled for the first time, because of a bearing capacity failure in the underlying soil (Bozozuk 1977; 1979). Other similar foundation failures of tower silos have occurred (Bozozuk 1972; Eden and Bozozuk 1962) and numerous cases of tilting of large tower silos because of excessive soil settlements can be seen.

The concern created among farmers and silo contractors about poor foundation performance resulted in the initiation of research programs at the Division of Building Research of the National Research Council of Canada and at several universities, to study the foundation design and performance of large tower silos. This paper reports on the results of one such investigation.

The foundation performance studied is for a steel tower silo founded on a rigid circular concrete raft erected in 1976 adjacent to the failed concrete silo. The engineering properties of the soil at the site were used to specify the dimensions for the foundation for the new silo. The performance of the foundation was then monitored during loading in order to confirm the application of engineering foundation theory and practice to this type of structure.

CHOICE OF FOUNDATION

A concise summary concerning the essentials for good foundation design was given by Sowers (1970): (1) it must be placed at an adequate depth to prevent frost damage, heave, undermining by scour, or damage from future construction nearby; (2) it must be safe against breaking into the ground; and (3) it must not settle enough to disfigure or damage the structure.

The first requirement involves a number of decisions based to a large extent on the sound judgement and experience of the designer or builder. For the other two requirements, i.e., the allowable bearing capacity and settlement of the structure, an understanding of the principles and methods of foundation engineering is essential.

In general, the selection of foundations for large tower silos should follow the same procedure as that for other earth-supported structures such as buildings, warehouses and bridges. The soil conditions at the silo site should be ascertained and its strength and compressibility characteristics determined. These properties may be determined by field or laboratory tests on undisturbed samples of the soil. The procedure for calculating the allowable bearing capacity for tower silos founded in clay from these soil properties is given by Bozozuk (1974).

The second step in foundation design involves specifying the thickness of the foundation and providing the necessary steel reinforcement. The structural design of the foundation should be done in accordance with standard reinforced concrete design procedures, such as presented by Turnbull and Bellman (1977) for ring footings. Alternatively, for most soil conditions and common silo dimensions, information on the size of footing and on reinforcing can be obtained from design tables prepared by Canada Plan Services (Plans 7411 and 7412). These plans give in tabulated form the maximum sizes of tower silos that can be safely supported on soils with allowable bearing capacities varying from 75 kPa to 250 kPa.

SITE DESCRIPTION

The site, located 10 km south-west of Ottawa and 7 km north of Richmond, Ontario, is underlain by an overconsolidated deposit of marine clay. Known as Leda clay, the soil was deposited in the saline water of the Champlain Sea during the recent episode of continental glaciation (Gadd 1975). Detailed discussions concerning the geology of the clay are available elsewhere (Gadd 1975; Fransham and Gadd 1977).

The soil profile together with water contents, Atterberg limits, undrained shear strengths and preconsolidation pressures is presented in Fig. 1. The top 2.4 m consists of desiccated and oxidized brown clayey silt below which the soil is predominantly soft gray silty clay. The Atterberg limits are fairly constant with depth. The plasticity index and the liquid limit are approximately 20 and 40%, respectively. Within the oxidized layer, the liquid limit and the natural water content are about the same. In the softer clay layer below the desiccated crust, however, the natural water contents average 52% and are higher than the liquid limits.

Figure 1 also shows the undrained shear strength and the remoulded shear strength determined by field vane tests. Within the top layer, the average undrained shear strength was about 60 kPa. Below the crust and to a depth of 15 m, the undrained shear strength was fairly constant with an average value of 36.5 kPa. Then it gradually increased to 60 kPa at 18 m.

The remoulded strength of the soil gives an indication of the sensitivity of the clay to disturbance. The average remoulded shear strength of 5 kPa compared to the average undisturbed shear strength of 36.5 kPa indicates that the soil is very sensitive to disturbance.

From the distribution of the most probable preconsolidation pressure and the in situ vertical effective stress, it was concluded that the desiccated layer was overconsolidated by about 100 kPa, while the underlying clay was overconsolidated by only about 20 kPa.

From Bozozuk (1974), a soil shear strength of 36.5 kPa will allow a foundation bearing value of 81 kPa with a safety factor of 3 against bearing capacity failure for a circular foundation.

The possible settlement of a structure with a foundation pressure of this magnitude depends on the distribution of the maximum vertical stress in the soil mass in relation to the overconsolidation pressure of the soil. The increase in vertical stress at the base of the foundation would be 81 kPa minus the pressure from the excavated soil, and it would decrease with depth. For the soil conditions at the site, the soil in the upper 3 m would undergo little compression, while significant but tolerable settlements would occur in the soil beneath this depth. An allowable bearing value of 81 kPa, therefore, was chosen as the limit for the concrete raft to support the new silo.

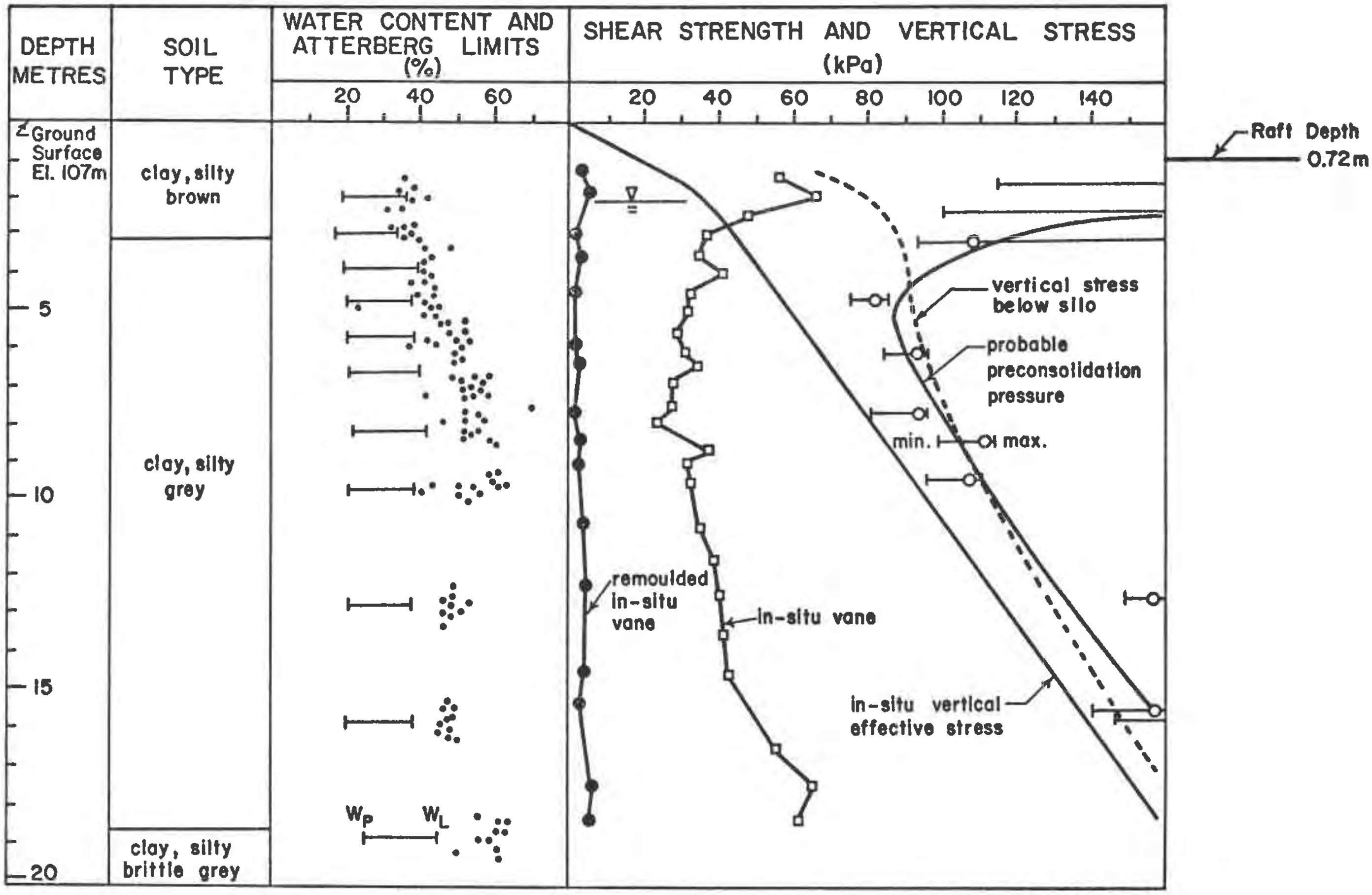

Figure 1. Soil profile at raft location.

DESCRIPTION OF STRUCTURE

The steel tower silo was erected in June 1976. It had an inside diameter of 7.6 m, height of 26 m excluding the roof, and a capacity of about 950 tonnes.

In order to ensure that the remoulded soil from the failed concrete silo did not affect the performance of the new structure, the edge of the new foundation was located about 6 m from the zone of disturbed clay. Fig. 2 shows the relative position of the raft foundation to the failed silo and the barn.

The raft foundation, 13.7 m in diameter, was founded at a depth of 0.72 m (Fig. 3). The figure also shows the concrete pedestal cast on top of the raft. The pedestal is the standard Harvestore foundation normally used to support the silo when soil conditions are adequate. The standard Harvestore foundation anchors the steel silo at its base and contains the trough for the unloading conveyor system.

Steel reinforcing for the raft was provided by adding 20 mm radial bars 3.1 m long placed 8 cm above the bottom of the excavation. The reinforcing bars were positioned 3.2 m from the center of the raft at a uniform spacing of 0.15 m around the silo perimeter. To hold them in position, they were fastened at both ends to 20 mm circumferential steel reinforcement.

The total dead mass of the foundation and silo was approximately 630 tonnes.

Figure 2. Raft foundation for tower silo. Portion of failed silo in the background.

INSTRUMENTATION AND OBSERVATIONS

The performance of the silo during loading and the reaction of the underlying soil to the load were monitored by the installation of instruments to measure the settlement and deformation of the raft, the zone of compression in the soil mass, the pore water pressures beneath the silo, and the contact pressure distribution on the base of the raft. The instrumentation consisted of a deep bench mark driven to the bottom of the clay deposit, 24 levelling points on the raft (12 around the circumference of the silo and 12 around the edge of the raft), six deep settlement gauges, two Gloetzl pneumatic and four Geonor standpipe piezometers, and eight Gloetzl hydraulic earth pressure cells. The locations are given in Fig. 3.

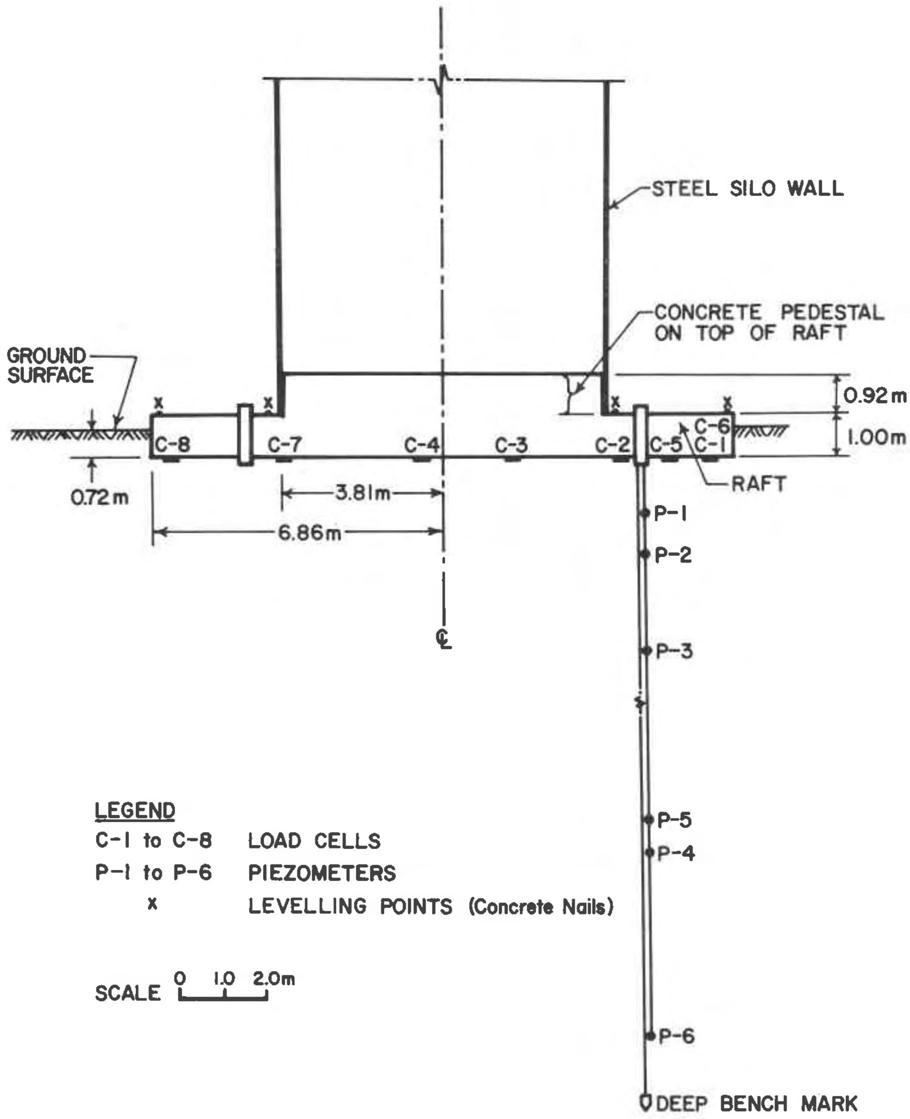

Figure 3. Section through silo, showing foundation and instrumentation.

The load-settlement relationship is plotted in Fig. 4. The load was applied continuously during construction and subsequent filling of the silo with haylage over a period of about 50 days, which raised the applied net pressure to about 55 kPa. The settlement due to this loading was about 7 mm. In the following 150 days when a small additional load was applied and removed, a further settlement of 4 mm took place. During the last 50 days of this period ending in December 1976, the settlement was less than 1 mm, indicating a very slow rate of settlement. Frequent settlement surveys were discontinued in February 1977, when it was discovered that the silo had heaved more than 3 mm due to frost penetration of the underlying clay through the concrete raft that extended beyond the silo wall.

By October 1978, when the silo had undergone two cycles of loading and unloading, and was loaded for the third time to a net applied pressure of 65 kPa, the settlement had reached a total of 21 mm.

The differential settlement of the raft monitored with the 24 levelling points showed that the raft was very rigid. The average differential movement was in the order of 1 mm, even when the entire structure was lifted by frost action on only a part of its base.

The settlement gauges showed that about 90% of the soil compression took place in the upper 14 m of the soil profile and almost one half of this settlement occurred in the 4 m of soil directly below the raft. This settlement pattern was fairly constant over the entire loading range and has not changed with time.

The excess pore water pressures were very low compared to the pressures applied during construction. The excess pore water pressures of approximately 6 kPa indicated that the silty clay drained rapidly during the construction and loading period of less than 50 days and that the observed settlements were due to both elastic soil movements and consolidation. The excess pore pressures were almost completely dissipated 60 days after the silo was filled.

The earth pressure cells at the base of the raft indicated that the contact pressure distribution across the foundation was nonuniform and consistent with elastoplastic theory for such foundations. When the average gross contact pressure from the dead and live loads was 72 kPa, the actual contact pressure was approximately 80 kPa at the edge, reducing to about 50 kPa towards the center of the raft.

SETTLEMENT OF SILO

Although reliable methods are available for estimating the allowable foundation pressure to ensure an adequate factor of safety against a bearing capacity failure, it is difficult to predict the settlement that this load will produce. The total settlement of a structure will be composed of: (a) an immediate settlement due to lateral deformation of the underlying soil under the influence of the applied load; and (b) long-term consolidation of the soil due to dissipation of excess pore water pressures.

The estimated theoretical immediate settlement was compared to the observed settlement for pressure levels of up to 55 kPa. The estimate was based upon the secant moduli obtained from triaxial stress-strain curves. At the higher pressures, the loads were applied slowly allowing consolidation to take place at the same time; consequently the immediate settlements were not determined.

Stress-strain curves for 11 isotropically consolidated undrained (CIU) triaxial tests at various depths were available. The compressible zone below the bottom of the raft was therefore divided into 11 soil layers and the calculated compressions of the layers were added to give the total settlement of the raft. The compression of each layer was calculated from: (a) estimated vertical and lateral stress increases due to the foundation pressure at the center of the layer from elastic theory, (b) the secant modulus determined from the particular triaxial stress-strain curve corresponding to the vertical stress increase estimated above and (c) the compression of the soil layer determined from elastic theory. Elastic formulae relating stress and strain are found in many soil mechanics textbooks. It is common to use a Poisson's ratio of 0.5 for calculations of immediate settlement in saturated clay soils.

In general, the estimated settlements were greater than observed. For example, corresponding to a net applied pressure of 55 kPa, the calculated and measured immediate settlements were 13.5 and 7 mm respectively. The main reason for the discrepancy was the low moduli obtained from the uniaxial tests. Low moduli are caused by disturbance in obtaining and preparing samples. The total immediate settlement at the maximum pressure applied

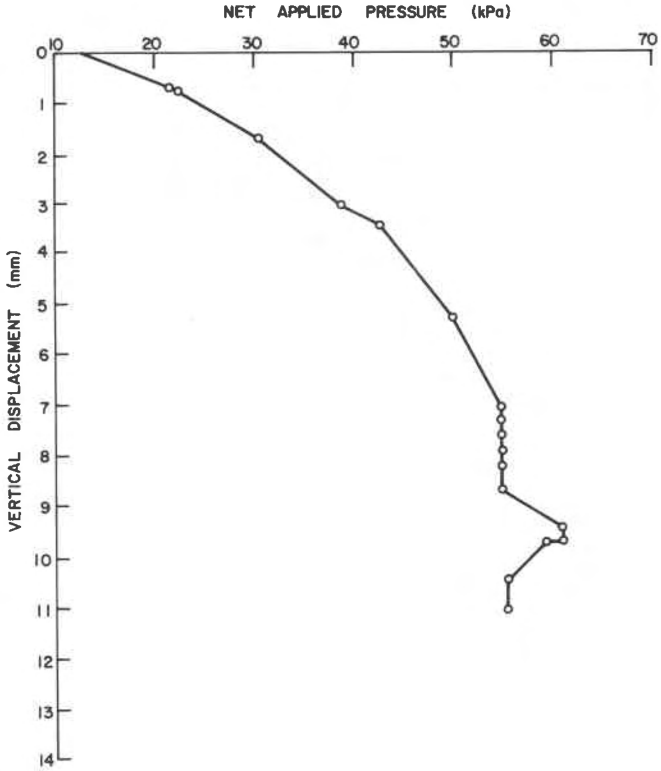

Figure 4. Settlement of silo raft foundation.

of 65 kPa was calculated to be 16 mm.

Based upon the maximum net applied pressure of 65 kPa, the predicted long-term primary consolidation settlement, determined from standard odometer test results, was about 46 mm. The total predicted settlement was therefore 62 mm. In addition, creep-type settlements could occur in the clay because the total vertical applied stress was about equal to or exceeded the preconsolidation pressure of the clay between 4 and 11 m of depth.

The eventual total settlements will, however, not be much greater than the 21 mm measured in October 1978. The immediate and consolidation settlements appeared to be complete but additional settlement will probably occur due to (a) loading and unloading of the silo, and (b) soil creep.

Both the calculated and measured total settlements discussed previously were based on the maximum net applied pressure of 65 kPa. The raft was, however, dimensioned to exert a net increase in average vertical pressure of 72 kPa. If the silo was filled with heavier silage so that the applied pressure would be greater than 65 kPa, both the predicted and measured settlements would be larger.

FROST ACTION

The unexpected frost heave of the silo raft indicated a relatively minor problem with this shallow foundation. Other silos normally have their foundations placed deeper than the depth of frost penetration. A deeper foundation in this case would have meant a substantial increase in material costs and changes in design to allow the normal unloading of the silo. In general a tall, heavy silo such as this one may require a wide footing extended well beyond the silo wall which would not be protected by the insulating and heating effects of the silage.

Frost heave pressures under the toe could break the footing. When the silo raft heaved, the earth pressure cells showed that the central part of the foundation was lifted clear of the underlying soil by the uplift pressures acting around the edge. The considerable thickness and rigidity of the raft prevented it from being damaged but a normal footing would probably have been cracked and damaged. Such footings therefore should be placed below the depth of frost penetration, or reinforced to withstand the uplift pressures. It may be more economical to protect the foundation by placing insulation below or above the footing extension or by building an earth fill around the silo to prevent excessive frost penetration.

SUMMARY AND CONCLUSIONS

The allowable gross bearing pressure of 81 kPa determined by commonly used foundation engineering methods resulted in a safe design for bearing capacity. The performance of this foundation supported the conclusion by Bozozuk (1977) that clay soil strengths measured in situ with a field vane can be used to predict the ultimate bearing capacity.

The observed total settlements were only about one third of those predicted from laboratory tests on soil samples. The difficulty in estimating small settlements resulted from the sensitivity of soil stress-strain characteristics to disturbance that normally occurs during sampling and handling.

The relatively small settlement of 21 mm indicated that the factor of safety of 3 against bearing failure, commonly used for buildings sensitive to settlement, could be reduced for farm silos. The allowable total settlement of most structures is limited to appproximately 25 mm to ensure that differential settlements will not disfigure or damage the building. The rigidity of circular tower silos should allow greater total settlements without damage; consequently it may be appropriate to reduce the factor of safety to about 2.5, which would result in smaller and more economic foundations. This may, however, increase the stresses in the foundations because of variations in soil compressibility. It would then become most important to include adequate reinforcing in the footings.

The large percentage of the total settlement which occurred in the 4 m of soil immediately below the foundation was not predictable from consideration of the soil profile or results of the laboratory tests. This zone of soil was overconsolidated by desiccation and appeared to be relatively incompressible compared to the underlying softer clay. This finding emphasizes the importance of placing footings deep enough so that the highly stressed region immediately beneath foundations does not occur in soil that has been excessively disturbed by weathering and frost action.

Foundations that may be affected by frost action should be placed below the depth of frost penetration. If this is not possible, the foundation should be protected with adequate insulation or reinforced with steel to withstand the bending moments. If frost action may cause differential movement, hence tilting of the silo, however, then frost protection rather than foundation strength should be the criterion for design.

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BOZOZUK, M. 1972. Foundation failure of the Vankleek Hill tower silo. Proc. Spec. Conf. Earth and Earth-Supported Structures, ASCE, SMFD, 1(2): 885-902.

BOZOZUK, M. 1974. Bearing capacity of clays for tower silos. Can. Agric. Eng. 16(1): 13-17.

BOZOZUK, M. 1977. Evaluating strength tests from foundation failures. Proc. 9th Int. Conf. Soil Mech. Found. Eng., Tokyo, 1: 55-59.

BOZOZUK, M. 1979. Problems with concrete tower silos. Can. Agric. Eng. 21: 69-77.
EDEN W.J. and M. BOZOZUK. 1962. Foundation failure of a silo on varved clay. Eng. J. 45(9): 54-57.
FRANSHAM, P.B. and N.R. GADD. 1977. Geological and geomorphological controls of landslides in Ottawa Valley, Ontario. Can. Geotech. J. 14(4): 531-537.
GADD, N.R. 1975. Geology of Leda clay. Proc. 4th Guelph Symp. on Geomorphology, Geological Survey of Canada Library, MGF 9S98 4th.
SOWERS, G.B. and G.F. SOWERS. 1970. Introductory soil mechanics and foundations. The Macmillan Company, London.
TURNBULL, J.E. and H.E. BELLMAN. 1977. Reinforced extended ring foundations. Canadian Society of Agricultural Engineering, Annual Meeting, 14 to 18 Aug. Guelph, Ont.

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